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MOORA FLOOD MANAGEMENT STUDY

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1 INTRODUCTION

1.01 The township of Moora, which is located some 150 km north of Perth in Western Australia, is drained by Moore River and two of its tributaries Coonderoo River and Yadgena Brook. The Moore River discharges to the Indian Ocean at Guilderton some 90 km south-west of Moora. Figure 1.1 is a locality map showing the area of interest. The catchment area of the Moore River to Guilderton is about 13,550 km^2. Of this, about 8,770 km^2 lies upstream of Moora. The catchment area to the Quinn's Ford Gauging Station is some 11,400 km^2. Yadgena Brook has a catchment area of some 132 km^2 at its confluence with Moore River.

1.02 The Moore River Catchment upstream of Moora has highly variable geology and geomorphology, with large areas of the catchment rarely contributing to Moore River discharge. The Coonderoo River, with a catchment area of some 6,700 km^2, rises in the northern section of the catchment (at an elevation of about 300 m AHD) and flows in a generally southerly direction until it joins the Moore River at Moora at an elevation of about 200 m AHD. The Coonderoo valley is characterised by flat topography, highly permeable soils and a large number of natural storage areas. The Moore River sub-catchment to the north-east of Moora, with an area of some 1,870 km^2 can be categorised into two distinct sections. The characteristics of the upper section of this sub-catchment is similar to the Coonderoo catchment. The lower section is characterised by relatively steep topography and relatively impermeable soils.

1.03 Historical reports indicate that more or less regular flooding has occurred in Moora up to 1968, with major floods occurring in 1917, 1955, 1963 and 1968 (GHD, 1991). No flooding occurred in Moora between 1968 and 1998. In 1999, three flood events of significance occurred in Moora.

1.04 During the period 18 - 20 March 1999, heavy rainfall associated with the remnants of ex-tropical cyclone 'Elaine' fell over the Moore River catchment. As a consequence, major flooding occurred in Moora over the weekend of 20 - 21 March 1999. The March 1999 flood was the highest flood on record at Moora and caused massive flood damage and social disruption in the town. This flood event was followed just two months later in May 1999 with another major flood event. In addition, another small but significant flood event occurred in Moora in August 1999.

1.05 The significant amount of flood data collected during the major flood events in 1999 provided an ideal opportunity to re-assess flooding behaviour and investigate potential flood mitigation options to alleviate flooding problems in Moora. On this basis, in November 1999, Water Studies Pty Ltd was commissioned by the Water & Rivers Commission of Western Australia (WRC) to undertake a flood management study for the township of Moora. This report is in response to this request.

1.06 This report contains a further 9 sections and is structured as follows:

- Section 2 provides a brief description of previous flood studies of the Moore River catchment.
- Section 3 describes the available topographic, rainfall and flood data for the Moore River Catchment.
- Section 4 summarises the methodology and results of the hydrologic and hydraulic analyses undertaken to estimate design flood discharges and flood levels in the study area.
- Section 5 describes the flood hazard in Moora for the existing level of development. The existing level of flood damage at Moora is also detailed.

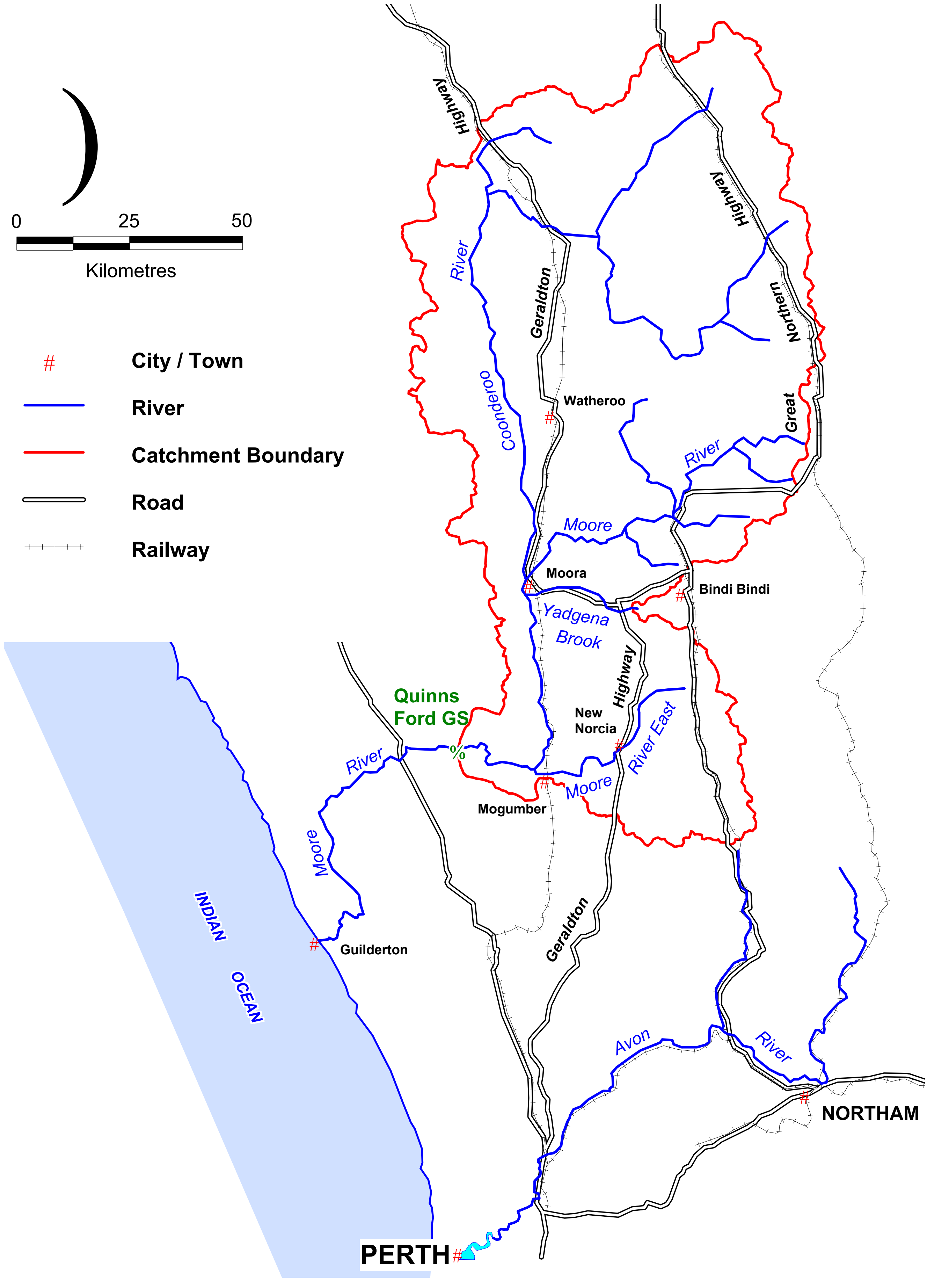

Figure 1.1 Locality Map

- Section 6 identifies the potential structural and non-structural floodplain management options which may be appropriate for Moora.

- Section 7 presents the impact of various structural mitigation options on flood levels and flood damage. The environmental and social impacts of the different structural options are also discussed.

- Section 8 provides an economic evaluation of the proposed structural mitigation options.

- Section 9 presents the conclusions of the study.

- Section 10 provides a list of references.

1.07 The report also contains six appendices.

- Appendix A provides details of the development and calibration of the hydrologic model used in the study.

- Appendix B provides details of the development and calibration of the hydraulic model used in the study.

- Appendix C details the estimation of design flood discharges in the Moore River at Moora.

- Appendix D details the estimation of design flood levels at Moora.

- Appendix E contains a report on the environmental assessment of different flood mitigation options.

- Appendix F presents detailed results on the impact of structural flood mitigation options on flood levels.

2 PREVIOUS STUDIES

2.01 Based on available information, two previous studies have been undertaken on flooding and drainage at Moora.

2.02 The first of these studies was undertaken in 1967 by Timmermans, Holt and Associates, Consulting Civil Engineers (GHD, 1991). This study investigated the causes of flooding and possible solutions to flooding problems at Moora. Based on recommendations of this study, it is understood that several drainage improvement works were undertaken in Moora in the early 1970's. These works included improvements to stormwater drainage in local catchments between the Moore River and Yadgena Brook, blocking (via an earthern levee) of an anabranch of the Moore River (Moore River No. 4 Channel) at Apex Park, and clearing and widening of sections of Moore River Main Channel through the town and downstream. The report of this study was not available for review.

2.03 The second study was undertaken in 1991 when Gutteridge Haskins and Davey Pty Ltd was commissioned by the then Water Authority of Western Australia to undertake a comprehensive flood study of the Moore River at Moora (GHD, 1991). This study estimated Moore River design flood discharges using regional flood frequency analysis methods. Flood levels through the town were estimated for 10, 25 and 100 year Average Recurrence Interval (ARI) flood events using the MIKE-11 hydraulic model. This study also assessed flood mitigation options for Moora for development conditions at the time. Data collected during the 1999 flood events and the findings of this study indicate that the 1991 study has significantly under-estimated both design discharges and design flood levels at Moora.

3 AVAILABLE DATA

3.1 OVERVIEW

3.01 Available data for the Moore River catchment consists of:

- Rainfall,
- Flood level,
- Geology and pedology, and
- Topographic information.

3.02 Rainfall data includes both continuous (pluviograph) rainfall records and daily rainfall totals. Flood level information includes continuous flood level records at stream gauges and peak flood levels throughout the township measured after a flood. A rating curve is used to convert recorded flood levels at stream gauges to flood discharges. Geology and pedology data is used to assist in the determination of runoff characteristics in the catchment. Topographic data is used to define catchment boundaries and the extent and depth of flooding along the rivers and across the floodplain. The following sections describe the available data for the Moore River Catchment.

3.2 TOPOGRAPHIC DATA

3.03 Available topographic data for the Moore River catchment consists of:

- 1:100,000, 20 m contour maps of the catchment published by the Royal Australian Survey Corps in 1981.

- 1:250,000, 50 m contour maps of the catchment published by the Royal Australian Army Corps in 1990.

- 1:250,000, 10 m contour maps published as part of the Moore Catchment Action Plan by Agriculture Western Australia in 1999.

- Half metre contour data of the study area prepared by WRC.

- Cross-section data surveyed in 1989 for the 1991 flood study.

- Detailed survey of the road bridges in Moora and cross-section surveys along some of the major roads in the town area undertaken by Fugro Pty Ltd in October 1999.

- Cross-section surveys upstream of the town area, along Moore River No. 4 branch, along Yadgena Brook and along Walebing and Mogumber Roads undertaken by Morris Heyhoe and Richards in December 1999.

3.3 GEOLOGY AND PEDOLOGY DATA

3.04 The Moore River Action Plan included 1:250,000 scale map sheets of 'geological information' and 'dominant soil groups'. These maps were used as an aid in the division of the catchment into sub-catchments to ensure that each sub-catchment encompassed soils with similar hydrologic behaviour. The maps were also used during hydrologic model calibration to assign similar model parameters (e.g. loss rates) to catchments with a similar geology and soil type.

3.4 DAILY RAINFALL DATA

3.05 Table 3.1 shows the available daily rainfall data within the area of interest. The locations of the daily rainfall stations are shown in Figure 3.1. The Commonwealth Bureau of Meteorology operates all the stations shown in Table 3.1.

Table 3.1 Daily Rainfall Stations In and Around the Moore River Catchment

Station Number	Station Name	Latitude	Longitude	Period of Record	
				From	To
008002	Ballidu Post Office	30° 36' 54"	116° 46' 11"	1/01/1911	31/12/1998
008008	Berkshire Valley 2	30° 35' 05"	116° 08' 03"	1/01/1907	31/03/1999
008009	Bindi Bindi	30° 38' 47"	116° 23' 05"	1/01/1929	30/09/1999
008013	Bowgada	29° 20' 54"	116° 08' 29"	1/01/1907	31/12/1998
008014	Dalwallinu North	30° 13' 13"	116° 47' 39"	1/02/1930	31/10/1999
008018	Buntine East	29° 56' 04"	116° 46' 08"	1/07/1929	31/08/1999
008025	Carnamah Post Office	29° 41' 24"	115° 53' 07"	1/01/1907	15/11/1999 [a]
008037	Coorow Post Office	29° 53' 60"	116° 01' 15"	1/01/1912	30/09/1999
008039	Dalwallinu Town	30° 17' 40"	116° 40' 36"	1/09/1912	15/11/1999
008070	Lake Hinds	30° 47' 02"	116° 30' 16"	1/11/1924	30/09/1999
008072	Latham Post Office	29° 46' 33"	116° 27' 33"	1/01/1934	30/09/1999
008077	Highfields	29° 36' 06"	115° 56' 19"	1/12/1956	31/10/1999
008085	Miling Post Office	30° 30' 35"	116° 22' 38"	1/10/1924	30/09/1999
008093	Morawa Post Office	29° 13' 36"	116° 01' 36"	1/05/1911	15/09/1999
008106	Perangery	29° 22' 12"	116° 24' 17"	1/09/1910	31/08/1999
008107	Perenjori Post Office	29° 26' 25"	116° 17' 60"	1/01/1918	30/09/1999
008108	Piawaning Post Office	30° 50' 24"	116° 23' 04"	1/11/1940	30/09/1999
008115	Round Hill	30° 34' 59"	116° 14' 07"	1/03/1905	31/12/1996
008121	Three Springs Post Office	29° 32' 08"	115° 46' 41"	1/01/1907	30/09/1999
008126	Minaru	29° 51' 05"	116° 14' 39"	1/01/1932	31/10/1999
008130	Watheroo Post Office	30° 18' 00"	116° 03' 29"	1/01/1907	30/09/1999
008137	Wongan Hills Post Office	30° 54' 35"	116° 43' 05"	1/01/1907	15/11/1999
008138	Wongan Hills Res.Station	30° 51' 31"	116° 44' 32"	1/01/1937	30/09/1999
008139	Wubin Post Office	30° 07' 33"	116° 38' 54"	1/01/1922	31/12/1997
008150	Newington	30° 52' 24"	116° 48' 52"	1/10/1912	31/12/1997
008151	Walebing	30° 40' 57"	116° 08' 15"	1/01/1907	30/06/1999
008225	Eneabba Post Office	29° 49' 11"	115° 16' 14"	1/05/1964	15/09/1999
008264	Wanarra	29° 31' 58"	116° 48' 02"	1/06/1973	31/12/1997
008275	Anro	30° 22' 08"	116° 16' 49"	1/01/1982	31/12/1998
008278	Warradarge	30° 04' 25"	115° 19' 42"	1/04/1980	30/09/1999
008283	Karawara	30° 09' 46"	116° 23' 58"	1/05/1986	30/09/1999
008289	Twin Hills	29° 40' 19"	115° 22' 50"	1/06/1972	31/12/1997
009006	Chelsea	30° 38' 37"	115° 47' 42"	1/10/1930	30/09/1999
009033	New Norcia Post Office	30° 58' 27"	116° 13' 37"	1/01/1907	28/02/1998
009037	Badgingarra Research Stn	30° 20' 21"	115° 32' 16"	1/04/1962	15/11/1999
009040	Wannamal	31° 09' 39"	116° 03' 07"	1/02/1906	30/09/1999
009046	Yathroo	30° 47' 19"	115° 42' 39"	1/01/1907	30/09/1999
009047	Yere Yere	30° 37' 59"	115° 43' 10"	1/08/1931	28/02/1989
009063	Badgingarra	30° 24' 30"	115° 30' 04"	1/04/1956	30/09/1999
009072	Bundidup	30° 17' 06"	115° 31' 18"	1/11/1958	30/09/1999
009167	Mogumber Farm	31° 00' 19"	115° 56' 13"	1/09/1918	31/07/1999
009218	Gillingarra	30° 56' 52"	116° 03' 30"	1/02/1991	31/05/1998
010076	Konnongorring	31° 04' 40"	116° 44' 52"	1/08/1913	30/09/1999
010156	Calingiri	31° 05' 28"	116° 27' 55"	1/01/1929	30/09/1999

[a] Three hour totals available for 01/01/1999 to 30/09/1999

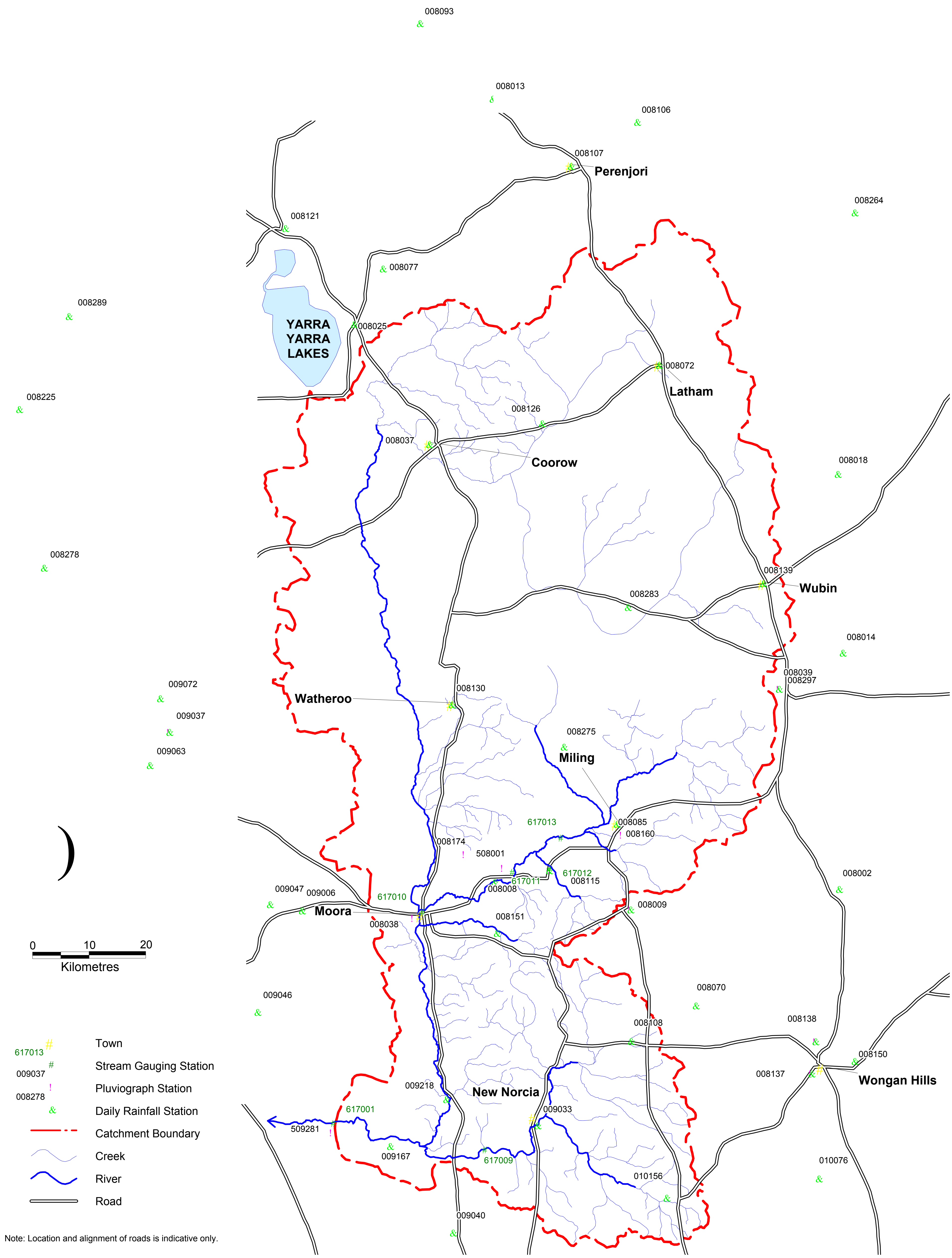

Figure 3.1 Locations of Rainfall and Stream Gauging Stations, Moore River Catchment

3.5 PLUVIOGRAPH DATA

3.06 Table 3.2 shows the available pluviograph data within the area of interest. The locations of the pluviograph stations are shown in Figure 3.1.

Table 3.2 Pluviograph Stations In and Around the Moore River Catchment

Station Number	Station Name	Latitude	Longitude	Period of Record	
				From	To
508 001	Berkshire Valley	30º 34' 43"	116º 09' 01"	1/01/1971	30/09/1999
509 281	Moore River @ Quinns Ford	30º 59' 56"	115º 50' 47"	14/11/1974	Present [a]
008 009	Bindi Bindi	30º 38' 47"	116º 23' 04"	1/07/1999	31/01/2000
008 038	Moora West	30º 38' 27"	115º 59' 03"	1/07/1999	31/01/2000
008 039	Dalwallinu Town	30º 17' 40"	116º 40' 36"	1/01/1999	Present [c]
008 115	Round Hill	30º 34' 55"	116º 14' 11"	1/07/1999	31/01/2000
008 160	Meridale	30º 31' 36"	116º 22' 11"	1/07/1999	31/01/2000
008 174	Noondine	30º 32' 23"	116º 05' 47"	1/07/1999	31/01/2000
008 137	Wongan Hills P.O.	30º 54' 35"	116º 43' 05"	26/06/1995	31/01/2000
008 138	Wongan Hills Res.Station	30º 51' 31"	116º 44' 32"	1/07/1983	30/09/1998
008 297	Dalwalanu AWS	30º 17' 40"	116º 40' 36"	23/04/1997	Present [b]
009 037	Badgingarra Research Stn	30º 20' 21"	115º 32' 16"	21/04/1997	31/01/2000

[a] Data unavailable for inclusion in the study;

[b] Data unavailable for March and May 1999 events;

[c] Three hourly rainfall data only during March and May 1999 events.

3.07 Prior to July 1999, there were only five pluviograph stations in the area of interest: Berkshire Valley (508001), Dalwallinu AWS (008297), Wongan Hills Post Office (008137), Wongan Hills Research Station (008138) and Badgingarra Research Station (009037). Of these, only Berkshire Valley and Dalwallinu AWS are inside the Moore River catchment. Unfortunately, Dalwallinu AWS failed during both the March and May 1999 flood events. Note however, the Dalwallinu Town daily rainfall station (008039) recorded 3 hourly rainfall totals during daytime hours during the above events.

3.6 STREAMFLOW DATA

3.08 Summary details of stream gauging stations in the Moore River catchment are given in Table 3.3. No streamflow data is available for the Coonderoo River Catchment.

3.09 Note that Quinn's Ford Gauge (617001) was the only gauge in operation prior to May 1999. Figure 3.1 shows the stream gauge locations.

Table 3.3 Summary of Gauging Station Details, Moore River Catchment

Station Number	Station Name	Latitude	Longitude	Period of Record		Rated
				From	To	
617001	Moore River - Quinns Ford	30º 59' 58"	115º 50' 34"	06/05/1969	present	y
617009	Moore River East - Woury Pool	31º 01' 03"	116º 07' 12"	14/05/1999	present	Y
617010	Moore River North - Moora Caravan Park	30º 38' 20"	116º 00' 11"	23/06/1999	present	Y
617011	Moore River North - Long Pool Bridge	30º 35' 41"	116º 10' 19"	17/06/1999	present	Y
617012	Dungaroo Creek - u/s Roundhill Bridge	30º 34' 04"	116º 14' 13"	17/06/1999	present	Y
617013	Moore River North - Nardy Road	30º 31' 60"	116º 16' 36"	17/06/1999	present	Y

3.10 Table 3.4 summarises recorded peak flood discharges at the six gauging stations for the four events of 1999. The peak discharge in March 1999 at Quinn's Ford (617001) is the highest recorded discharge since the gauge was established in 1969. Note that the maximum gauged discharge at Dungaroo Creek (617 012) is significantly less than the estimated flood discharges.

Table 3.4 Maximum Gauged Discharge and Recorded 1999 Event Discharges, Moore River Stream Gauges

Station Number	Station Name	Maximum Gauged Discharge (m^3/s)	Event Peak Discharge (m^3/s)			
			March 1999	May 1999	July 1999	August 1999
617001	Moore River - Quinns Ford	409	435	196	139	125
617009	Moore River East - Woury Pool	10	-	161	93	84
617010	Moore River North - Moora Caravan Park	25.4	-	-	29.1	17.2
617011	Moore River North - Long Pool Bridge	18.0	-	-	23.4	11.8
617012	Dungaroo Creek - u/s Roundhill Br	1.0	-	-	27.0	3.9
617013	Moore River North - Nardy Road	8.2	-	-	10.3	8.7

NK: Not Known
'-' Station not installed;

3.7 FLOOD LEVEL DATA

3.11 Recorded height and time of peak flood levels were available for the March 1999 and May 1999 events at the location of some of the (then) future stream gauging stations. The available data is summarised in Table 3.5. Note that the timing of flood peaks is approximate only.

Table 3.5 Summary Details of Available Flood Level Data at Gauging Station Locations

Station Number	Station Name	March 1999		May 1999	
		Aprox. Peak Height (m)	Time of Peak	Approx Peak Height (m)	Time of Peak
617 010	Moore River North - Moora Caravan Park	14.3	0700, 21/3/1999	13.8	⊄ 1600, 28/5/1999
617 011	Moore River North - Long Pool Bridge	13.21	-	12.5	⊄ 0600, 28/5/1999
617 012	Dungaroo Creek - U/S Roundhill Bridge	13.12	1800, 20/3/1999	12.1	⊄ 0730, 27/5/1999
617 013	Moore River North - Nardy Road	10.8	-	-	-

'-' Denotes data not available

3.12 Recorded flood levels were also available for the March 1999 and May 1999 events at locations other than gauging stations. This data included peak flood levels throughout the town area surveyed by WRC from debris and high water marks following the March and May 1999 floods. Flood level data used in the calibration of the hydraulic model is summarised in Table 3.6.

Table 3.6 Summary Details of Available Flood Level Data at Locations Other than Gauging Stations

Location	Description
Melbourne Street, Moora, March 1999	Flood levels taken by a resident from 0245 hrs (21/3/1999) to 2330 hrs (21/3/1999).
Tootra Street, Moora, May 1999	Flood levels taken at Tootra Street river crossing, near BP depot, Moora, by Rob Lenox from 1048 hours (27/5/1999) to 1230 hours (28/5/1999)
Yadgena Brook at Mogumber Road	Peak = 8.84 m at ⊄ 0200, 18/08/1999.
Dungaroo Creek, U/S Roundhill Bridge	Peak flood height = 13.12 m for the March 1999 event and approximately 12.1 m for the May 1999 event.
Dungaroo Creek, D/S Roundhill Bridge	Peak flood height = 12.71 m for the March 1999 event and 11.95 for the May 1999 event.
Moora Town Area	Peak flood levels measured throughout the town area for March and May 1999 event.

4 ESTIMATION OF DESIGN FLOOD DISCHARGES AND FLOOD LEVELS

4.1 METHOD OF ANALYSIS

4.01 Two numerical models were used to simulate flooding behaviour in the Moore River catchment:

- A runoff - routing model (URBS) was used to estimate flood discharges throughout the Moore River catchment, and
- An unsteady flow hydraulic model (MIKE-11) was used to estimate flood levels in the Moora township area.

4.02 The adopted configuration of the (URBS) hydrologic model of the Moore River catchment is shown in Figure 4.1. The URBS Model was calibrated against recorded flood data for four historical flood events. Full details of the Moore River URBS Model development and calibration are given in Appendix A.

4.03 The calibrated URBS model was used to estimate discharge hydrographs at the boundaries of the (MIKE-11) hydraulic model. The MIKE-11 model was then calibrated against recorded flood data for the same four historical flood events used in the URBS model calibration. The adopted configuration of the MIKE-11 model for Moora is shown in Figure 4.2. Full details of the Moora MIKE-11 model development and calibration are given in Appendix B.

4.04 The calibrated URBS model was used to estimate design flood discharges throughout the Moore River catchment. The calibrated MIKE-11 model was then used to estimate design flood levels in the area of interest.

4.05 Design flood discharges and flood levels at Moora were estimated for 2, 5, 10, 20, 50 and 100 Year ARI flood events and for the probable maximum flood event (PMF).

4.2 DESIGN FLOOD DISCHARGES

4.06 Table 4.1 shows the predicted design discharges at three key locations in the study area - Moore River at Moora Caravan Park, Moore River at Quinns Ford Gauging Station and Yadgena Brook at Walebring Road - for flood events ranging from 2 year ARI to PMF. The methodology and assumptions used to derive these results are discussed in detail in Appendix C.

4.07 Peak discharges predicted by the URBS model for March, May, July and August 1999 flood events in the Moore River catchment are also shown in Table 4.1.

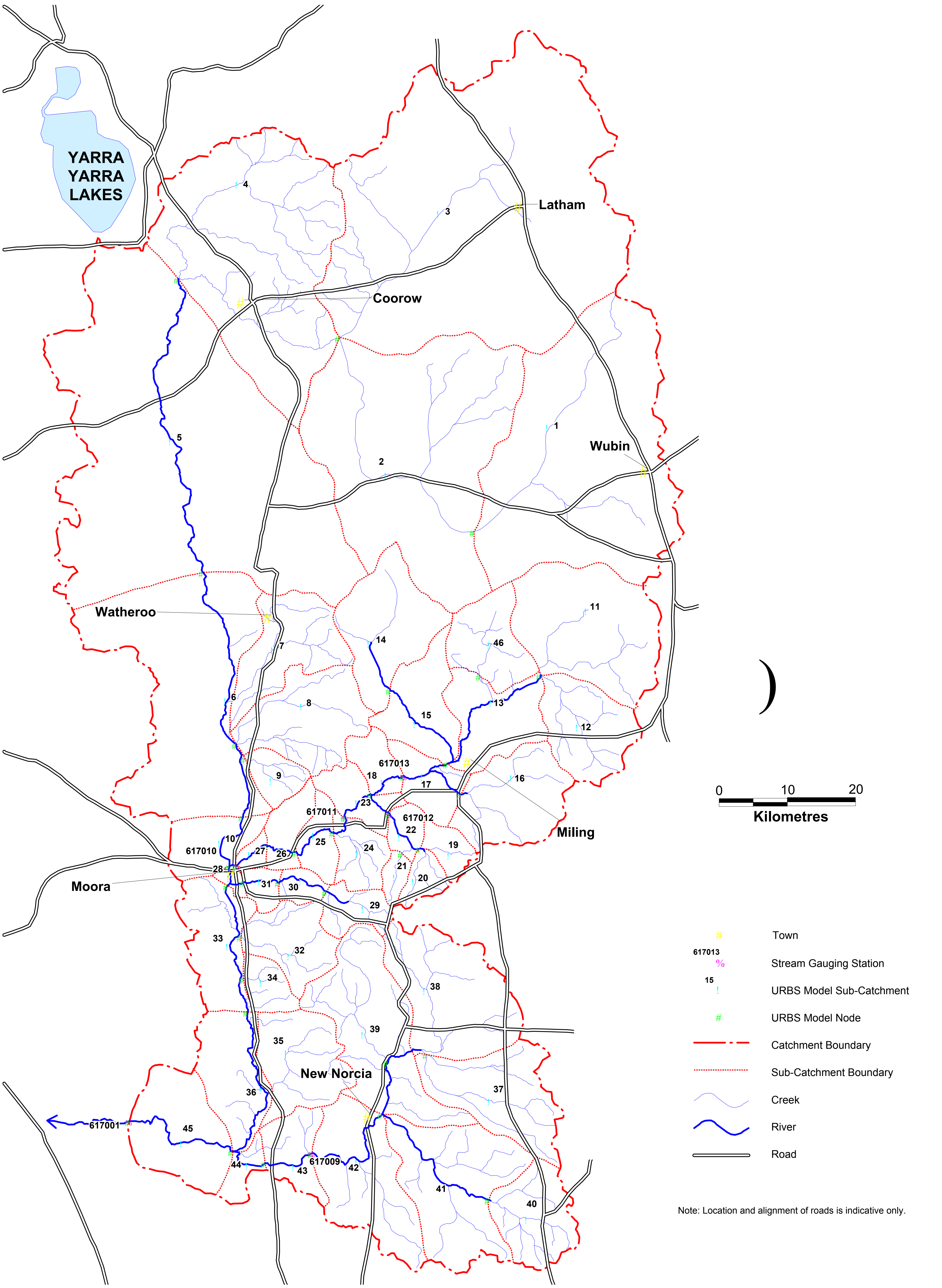

Figure 4.1 Moore River, URBS Model Configuration

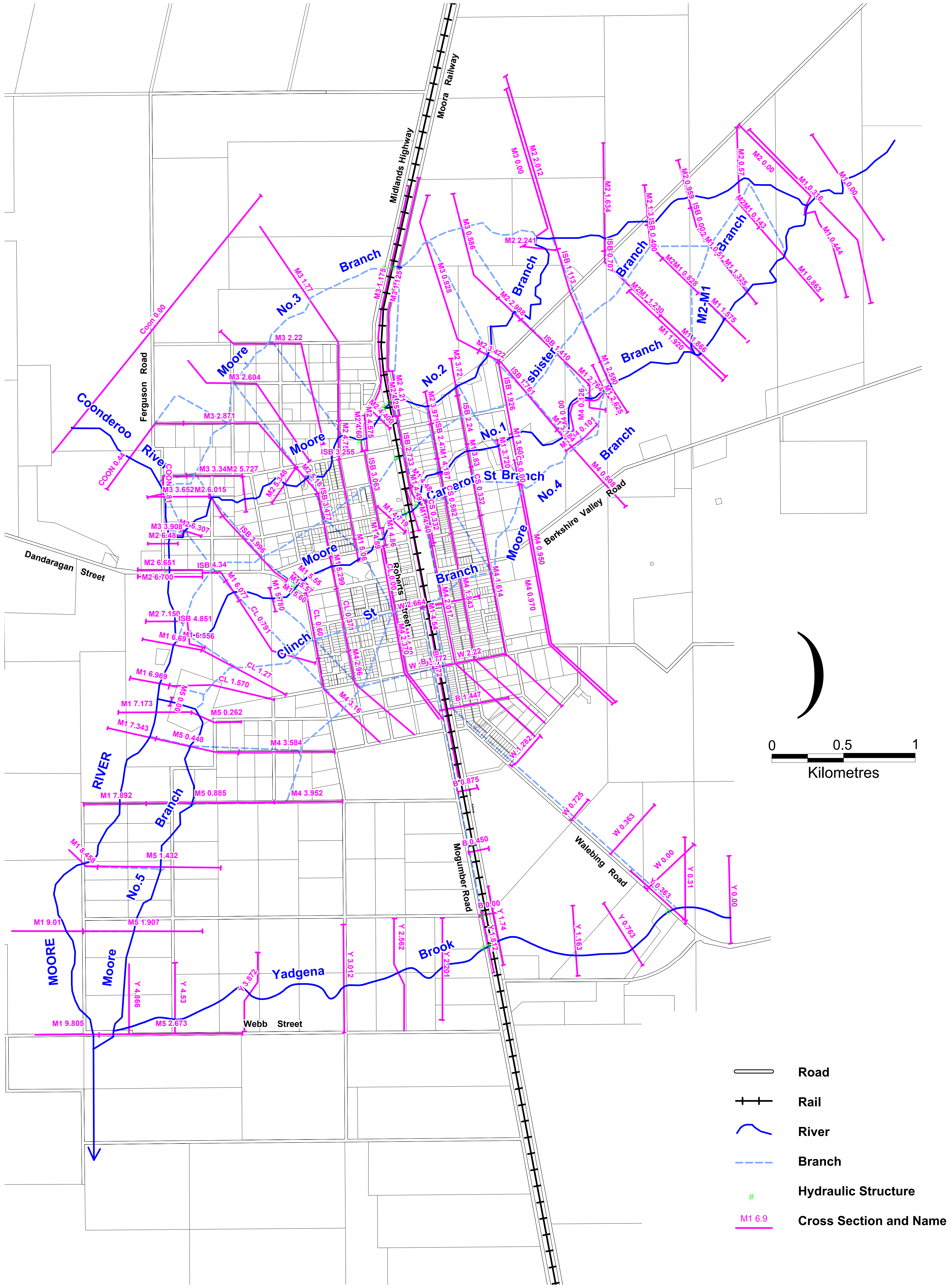

Figure 4.2 MIKE-11 Configuration of the Moora Township Area

Table 4.1 Peak Discharges for Various Design Flood Events, Moore River Catchment, URBS Model

ARI (years)	Peak Discharge (m^3/s)		
	Moore River at Moora Caravan Park	Yadgena Brook at Walebing Road	Moore River at Quinn's Ford
2	38	6.3	57
5	83	17	145
10	110	28	229
20	159	42	331
50	230	54	457
100	290	68	584
PMF	6,300	1,270	13,300
March 1999	501	97	440
May 1999	285	37	298
July 1999	33	15	138
August 1999	19	131	136

4.3 DESIGN FLOOD LEVELS

4.08 Figures 4.3, 4.4, 4.5 and 4.6 show predicted peak flood level profiles for 10, 20, 50 and 100 year ARI flood events and the PMF event along Moore River Nos. 1, 2, 3 and 4 branches respectively. The methodology and assumptions used to derive these results are detailed in Appendix D. Peak flood level estimates at all model cross-sections for the above flood events are also presented in Appendix D.

4.09 The results show that along Moore River No. 1 branch, the average difference in peak flood levels between the 10 and 20 Year ARI events and between the 20 and 50 Year ARI events is about 0.2 m. Between the 50 and 100 Year ARI events, the average difference in peak flood levels is only about 0.1 m. Between the 100 Year ARI event and the PMF, the average difference in peak flood levels is about 2.6 m. Along the other branches of the Moore River, the average difference in peak flood levels is marginally less than the equivalent values for the Moore River No. 1 Branch.

4.4 EXTENT OF FLOODING

4.10 Figure 4.7 shows the estimated extent of flooding and flood level contours for the 100 Year ARI flood event. Note that this design flood has not inundated the part of Moora between the Moore River No. 2 and 3 branches. There may be some local water from along the Midlands Highway or from the Coonderoo River that may contribute to flooding in this area, as explained in Appendix B (Hydraulic Model Development and Calibration).

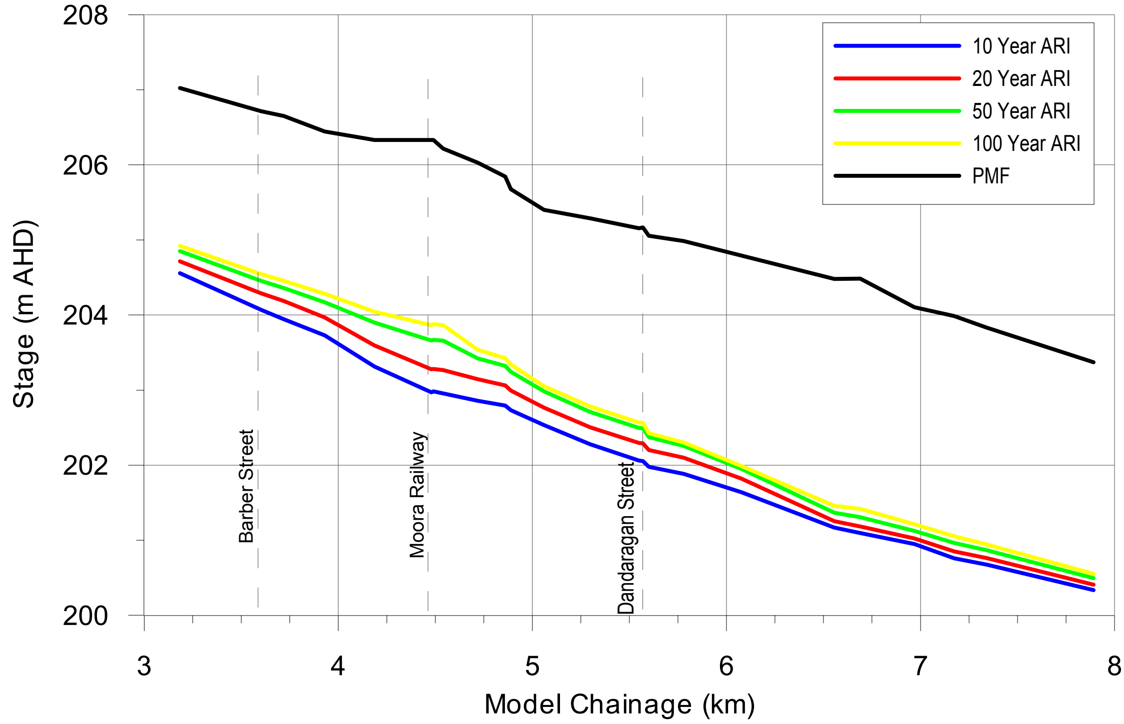

Figure 4.3 Peak Flood Level Profiles for the 10, 20, 50 and 100 Year ARI Flood Events and the PMF, Moore River No. 1 Branch

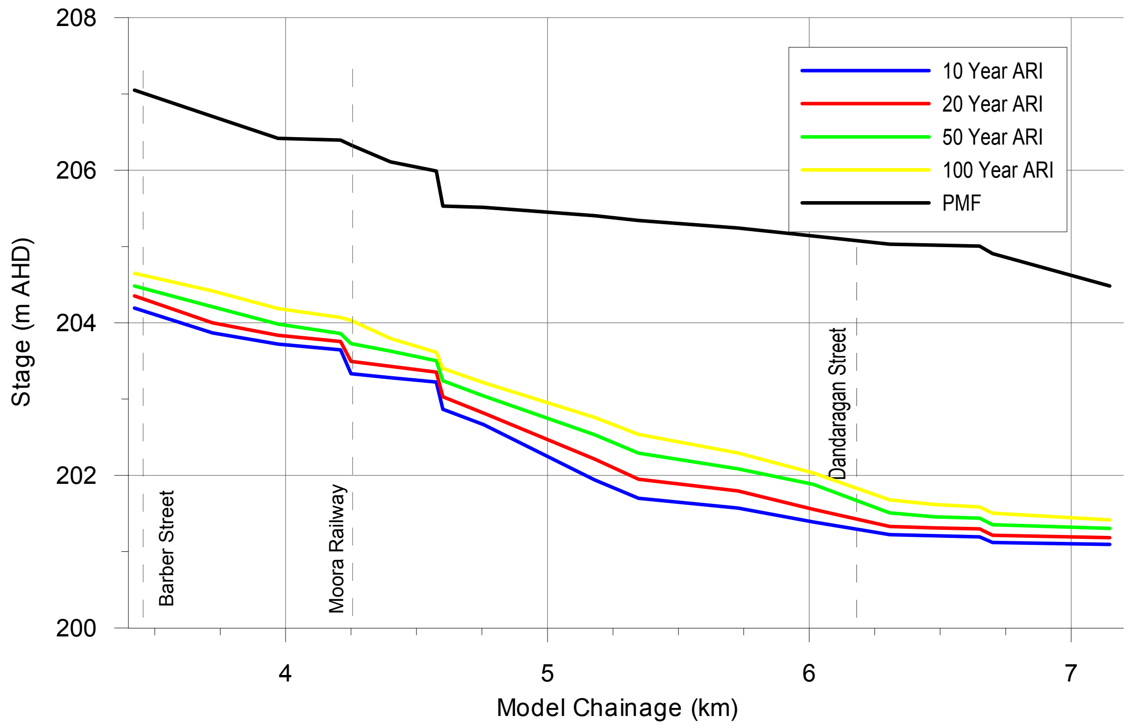

Figure 4.4 Peak Flood Level Profiles for the 10, 20, 50 and 100 Year ARI Flood Events and the PMF, Moore River No. 2 Branch

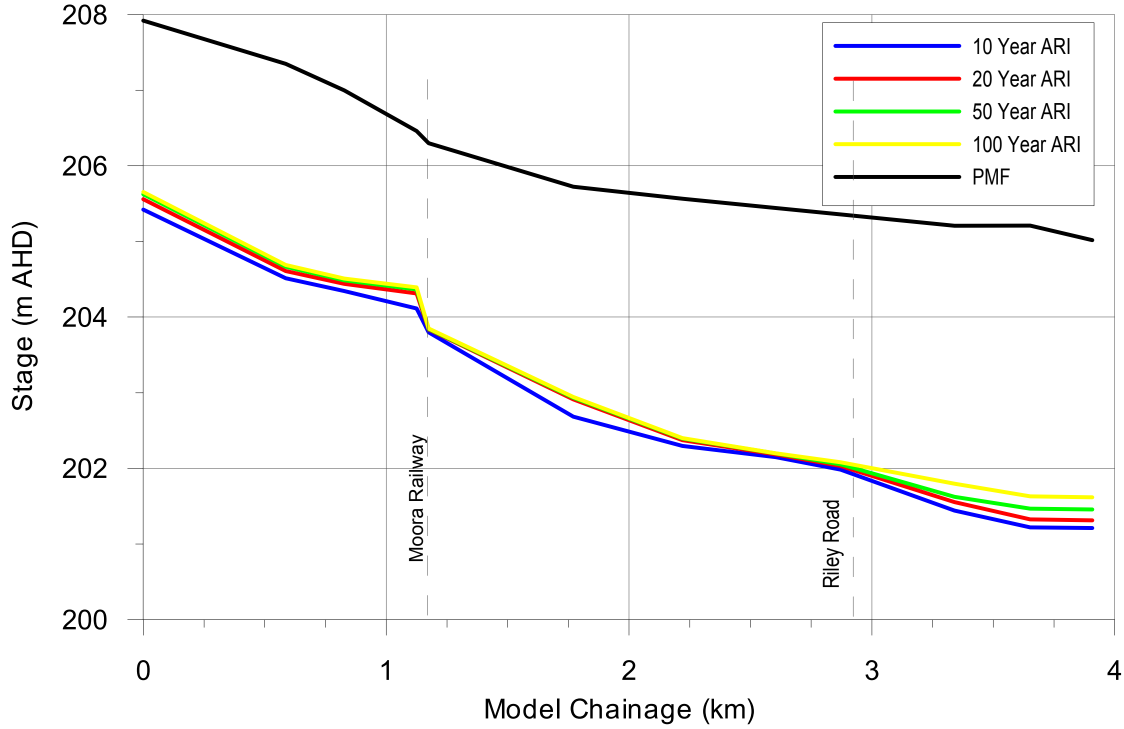

Figure 4.5 Peak Flood Level Profiles for the 10, 20, 50 and 100 Year ARI Flood Events and the PMF, Moore River No 3 Branch

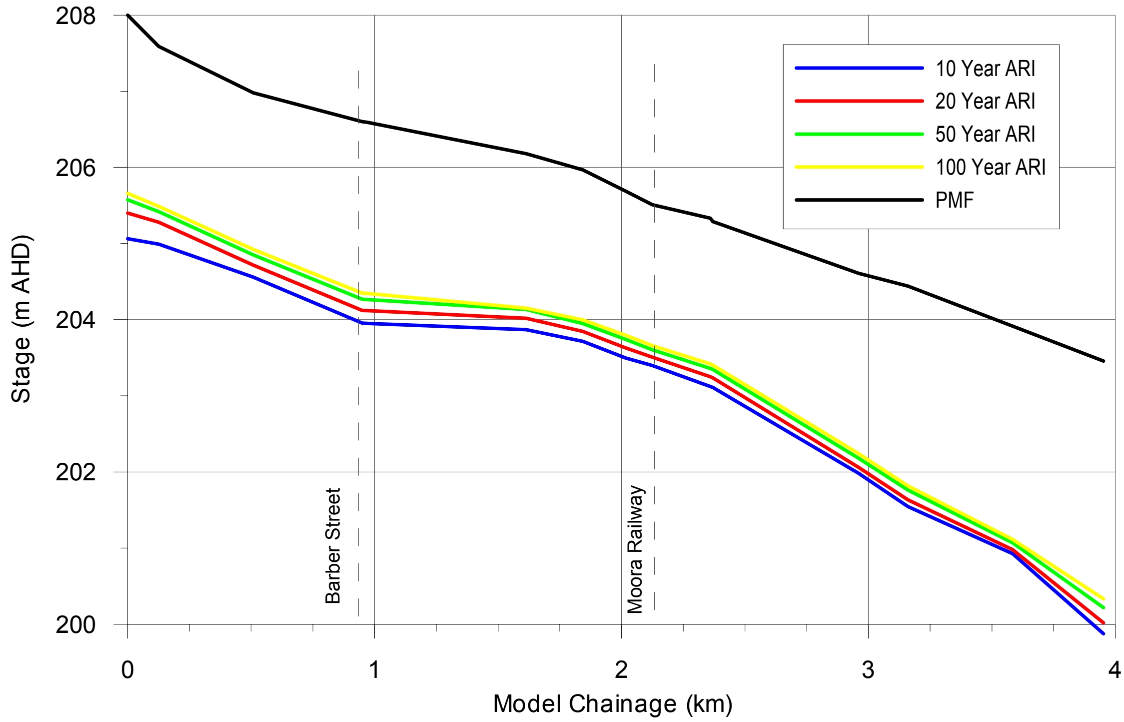

Figure 4.6 Peak Flood Level Profiles for the 10, 20, 50 and 100 Year ARI Flood Events and the PMF, Moore River No. 4 Branch

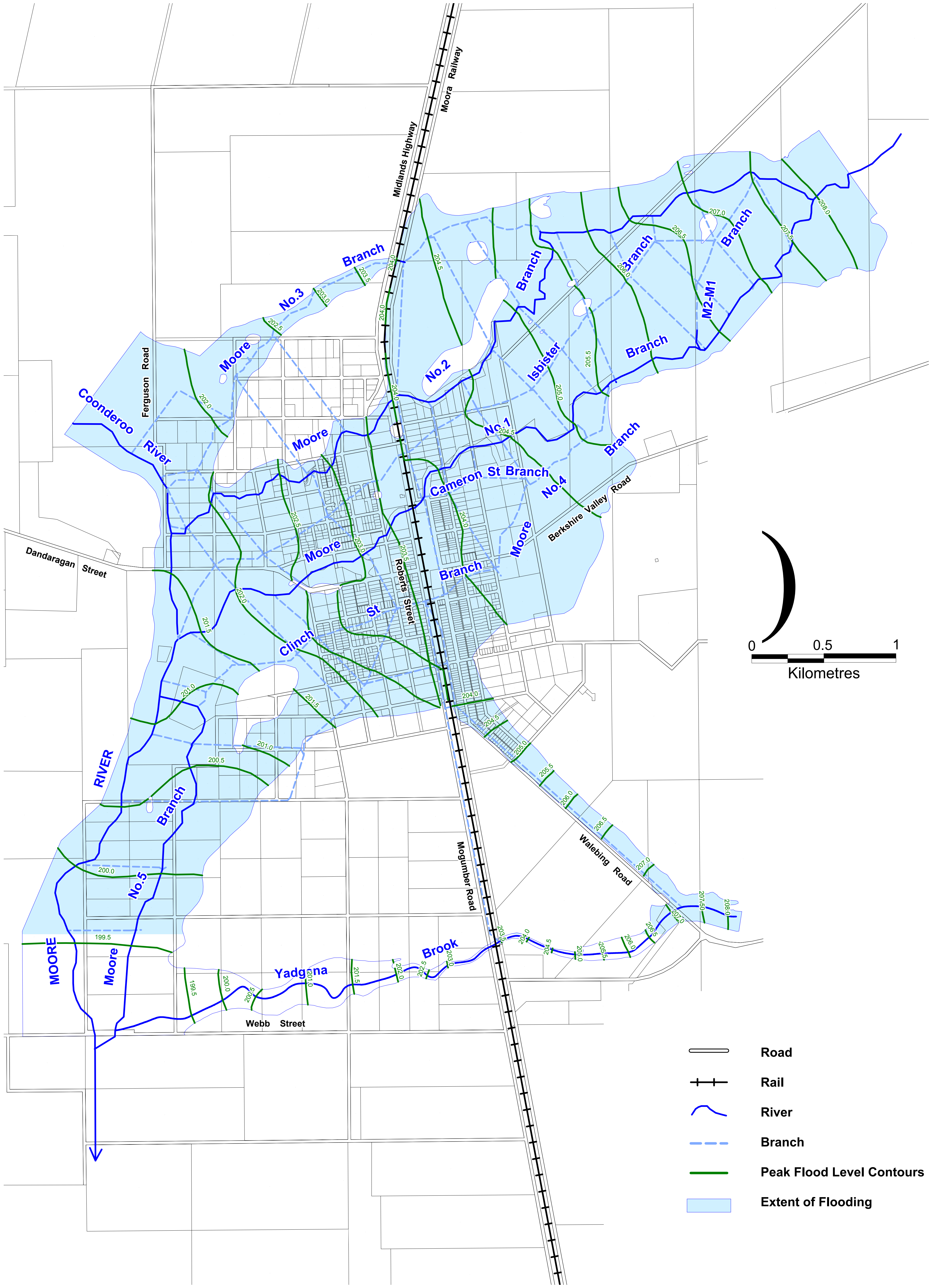

Figure 4.7 Extent of Flooding and Flood Level Contours for the 100 Year ARI Design Flood Event, Moora

4.5 SEVERITY OF MARCH AND MAY 1999 EVENTS

4.11 March and May 1999 floods at Moora were extreme events. Given the uncertainties in the design discharge estimation process (see Appendix C for details), it is not possible to assign ARI's to March and May 1999 flood events with any confidence. However, based on a subjective assessment of available information, it is estimated that the March and May 1999 events would have had ARI's of 100 - 250 years and 50 - 100 years respectively.

5 ESTIMATION OF EXISTING FLOOD HAZARD AND FLOOD DAMAGE

5.1 FLOOD HAZARD FOR EXISTING LEVEL OF DEVELOPMENT

5.01 The extent of flood liable land along the Moore River and its branches within Moora township were defined on the basis of the extent of inundation in the March 1999 flood, after consultation with WRC and Moora Shire Council. However, the existing flood hazard areas were defined on the basis of depth and velocity of floodwaters for the 100 year ARI flood event, as recommended in the Australian Floodplain Management Manual (SCARM, 2000). High and extreme hazard areas were defined as floodway areas. Low and medium hazard areas were defined as flood fringe areas. On the basis of this assessment and subsequent discussions with WRC and Moora Shire Council, Figure 5.1 shows the floodway and flood fringe area in the township of Moora.

5.2 TYPES OF FLOOD DAMAGE

5.02 Figure 5.2 shows the various types of commonly recognised flood damage. Basically, flood damages can be divided into two major categories: **Tangible** and **Intangible Damages**. Tangible Damages are the **financial costs** of flooding and can be quantified in dollar terms. Intangible Damages are the **social costs** of flooding and are reflected in increased levels of mental stress, physical illness, etc. Intangible damage is difficult to measure and impossible to meaningfully quantify in dollar terms. For this reason, only tangible damages have been considered in this investigation.

5.03 Tangible damages can be subdivided into two major sub-categories: **Direct Damages** and **Indirect Damages**. Direct Damage is the loss in value of an object or a piece of property caused by direct contact with floodwaters. Indirect damage is the loss in production or revenue, the loss of wages, additional accommodation and living expenses and any other extra outlays that occur as a consequence of the flood.

5.3 FLOOD DAMAGE MODEL

5.04 The computer program 'FLDAMAGE' was used to estimate flood damage associated with the design flood events in Moora (see Water Studies, 1992, for details of FLDAMAGE). This program estimates both direct and indirect damages. Direct damages included in the program are internal, structural and external damages. Indirect damages included in the program are cleanup costs, financial costs and opportunity costs.

5.05 In FLDAMAGE, the area of interest is divided into a number of triangular-shaped hydraulic cells (20 in the case of Moora). Estimated flood levels at cell nodes (obtained from hydraulic model results) are input to the program, as are property details throughout the area of interest.

5.06 Property details were determined by a car-based survey undertaken on 3 - 4 November 1999 and include property type, size, age and address, height of floor above ground, etc. (see Water Studies, 1992, for details). The program recognises seven basic 'types' of properties: residential,

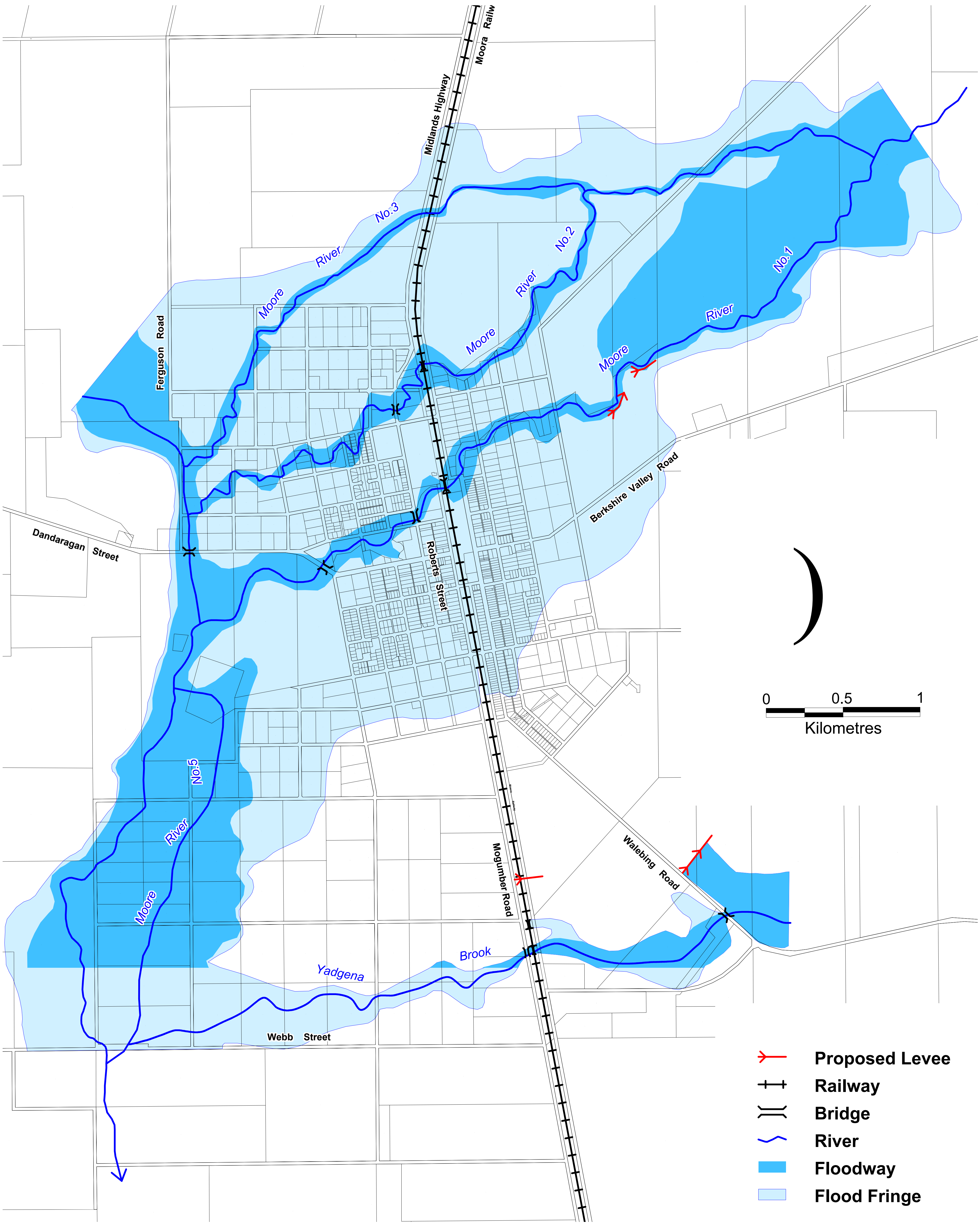

Figure 5.1 Existing Floodway and Flood Fringe Areas in Moora

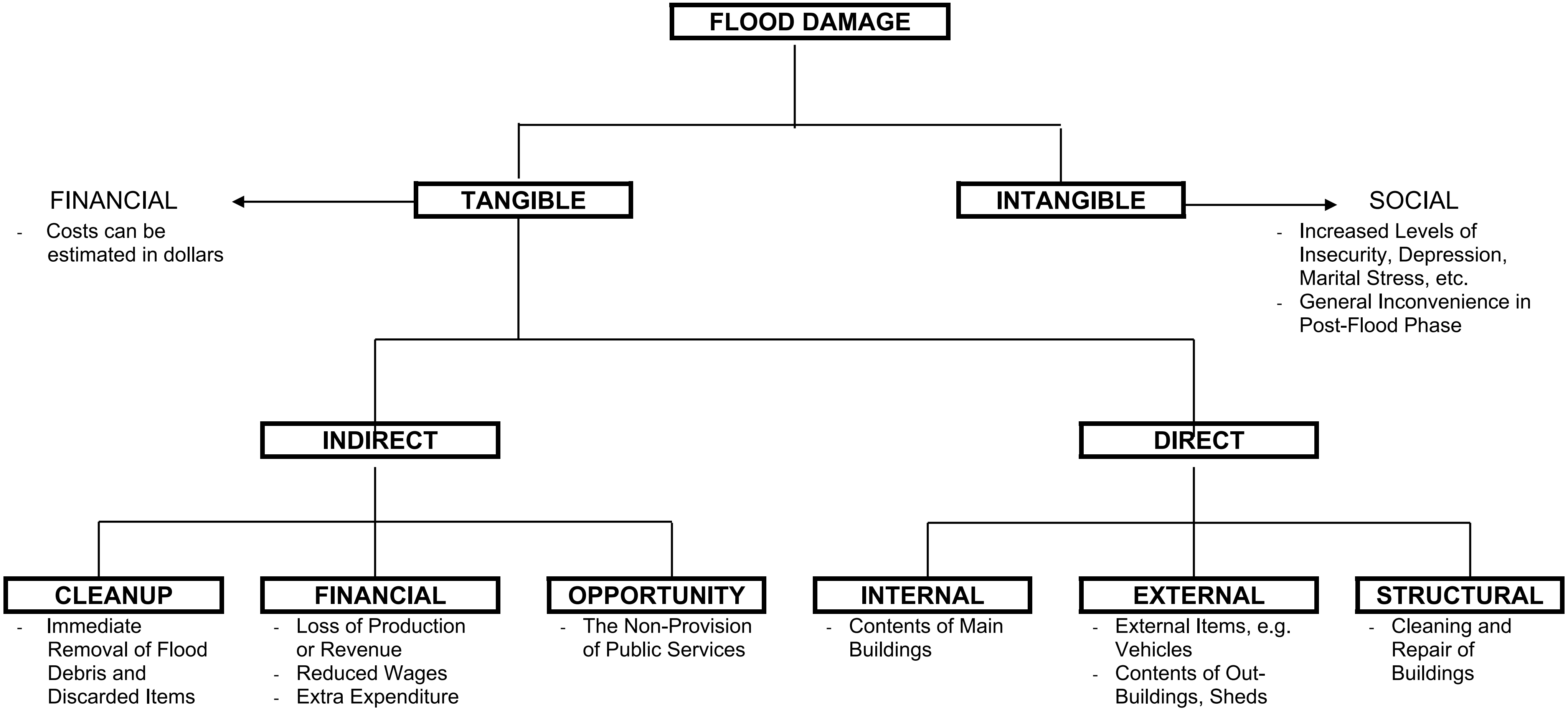

Figure 5.2 Types of Flood Damage

commercial, industrial, public authority, public institutions, recreational and other. Property types in Moora are predominantly residential, with some commercial, industrial and public. Stage-damage curves that relate potential internal flood damage to depth of flooding above floor level have been developed for each property type. A factor relating to the flood awareness of the residents converts the potential internal damage to the actual internal damage. Additional empirical relations enable external and structural damages to be estimated, together with indirect damage costs and estimates of actual damages.

5.07 Property data are coded and input to the program on a property-by-property basis, along with details of the coordinates (i.e. locations) of properties and cell nodes, and the estimated ground level at each property. Ground levels were obtained from contour maps of the area. The floor levels of properties in Moora were obtained by a car-based survey. In this survey, floor levels were estimated as ground level plus an estimated floor height above ground level.

5.08 Note that only flood damage to urban properties in Moora are estimated in this study. Flood damage to public infrastructure such as roads, railway, etc. and to rural property (farms) are not included in the damage estimates provided in this report.

5.09 The model was calibrated against available data on the number of properties flooded and the total property damage reported for the March 1999 flood event and the number of properties reported flooded for the May 1999 flood event. Having achieved a satisfactory calibration, the model was used to estimate damages for different design flood events and average annual flood damage for Moora.

5.4 MODEL CALIBRATION

5.4a March 1999 Flood Event

(i) Number of Properties Flooded

5.10 Based on available information, it appears that about 530 residential properties in Moora were flooded in March 1999 and of these, about 320 residencies were flooded above floor level. It appears that almost all commercial, industrial, public authority and public utility properties in Moora were also flooded during this event.

(ii) Reported Flood Damage

5.11 Estimates available for March 1999 flood damage in Moora are inconsistent. Furthermore, the basis of available estimates are not documented. Tables 5.1 and 5.2 show our estimates for March 1999 flood damage to property and public infrastructure respectively in Moora, on the basis of available information.

5.12 On the basis of available information the total flood damage to properties in Moora is estimated at \$11.0 m. In addition to these damages, it is reported that direct damages well in excess of \$1.0 m dollars occurred to rural (farm) properties around Moora. The total flood damage to public infrastructure such as road, rail, etc. is reported to be in excess of \$5.0 million.

Table 5.1 Estimated Property Damages, Moora, March 1999 Flood Event

Property Type	Damage	Source of Data
Residential	\$ 6.0 million	Shire of Moora
Commercial and Industrial	\$ 3.6 million	Water and Rivers Commission
Schools	\$ 0.35 million	Education Department
Public Authority and Utility	\$ 1.0 million	Shire of Moora

Table 5.2 Estimated Public Infrastructure Damages, Moora, March 1999 Flood Event

Infrastructure Type	Damage	Source of Data
Shire Roads and Footpaths	$ 2.5 - $ 4.0 million	Shire of Moora
Railway Line	$ 1.0 - $ 2.0 million	Shire of Moora
Telephone Lines and Infrastructure	$ 0.9 million[a]	Telstra

[a] Includes the exchange

(iii) Calibration

5.13 When calibrating the model against the March 1999 damages it was assumed that the degree of flood awareness in the Moora community at the time of the flood was very low. Thus, when converting potential internal damages to actual internal damages, a damage reduction factor of 0.85 was assumed for residential properties and 0.80 for commercial, industrial and public authority and utility properties. When converting potential external damages to actual external damages, a damage reduction factor of 0.5 was assumed for residential properties and 0.075 for commercial and industrial properties. For public authority and utility properties a damage reduction factor of 0.1 was assumed.

5.14 Other assumptions made in the analyses include:

- The businesses in Moora, on average, take 10 days to operate again after a major flood event.
- The flood-affected residents, on average, will require alternative accommodation for 15 days after a major flood event.
- Some houses in Moora have been lifted since the March 1999 flood. The impact of lifting these houses on flood damage estimates has been ignored.
- There are a number of floodprone churches in Moora. Available information to assess damage to church buildings is insufficient. Thus, these buildings have not been included in the flood damages analyses.

5.15 Tables 5.3 and 5.4 show details of the number of properties that were flooded in Moora and the associated flood damage in March 1999 as predicted by the FLDAMAGE model. These results are broadly consistent with reported values. The predicted number of properties flooded above floor level (AFL) is 309. (Note this excludes the number of properties raised since the flood). This compares with the reported value of 320. The total number of flooded properties (563) which includes the number of properties flooded below floor level (BFL), also compares well with the reported estimate of approximately 530 properties. The predicted damage values also agree broadly with the reported damage values. In view of the uncertainty in the reliability of available data the model is considered to reproduce the number of flooded properties and associated flood damages satisfactorily.

Table 5.3 Estimated Number of Flooded Properties, Moora, March 1999 Flood Event

Property Type	Properties Flooded			Avg. Depth of Flooding (m)	
	AFL	BFL	Total	AFL	BFL
Residential	309[a]	254	563	0.26	0.28
Commercial	52	14	66	0.30	0.26
Industrial	19	1	20	0.29	0.26
Public Authority	31	4	35	0.31	0.45
Public Utility	6	1	7	0.31	0.20
Total	417	274	691	0.30	0.28

[a] Excludes Lifted Houses

Table 5.4 Estimated Flood Damage, Moora, March 1999 Flood Event

Property Type	Actual Direct Damage ($1000)				Indirect Damage ($1000)	Total Actual Damage ($1000)
	Internal	External	Structural	Total		
Residential	1,719	1,867	877	4,463	896	5,359
Commercial	1,357	84	85	1,525	984	2,509
Industrial	700	39	44	784	508	1,291
Public Authority	294	29	71	394	241	635
Public Utility	366	37	0	403	119	522
Total	4,436	2,056	1,077	7,569	2,748	10,316

5.4b May 1999 Flood Event

5.16 Information available on damages for the May 1999 flood event is limited. Moore Shire Council has estimated that about 90 residential properties and about 16 commercial/industrial properties were flooded above floor level in May 1999. The total number of properties flooded above ground level and the damaged incurred by the flooded properties in May are not known. The only damage estimates available for May 1999 are some $ 0.3 million in damage to roads and about $ 22, 000 in damages to local schools.

5.17 In the aftermath of the March flood event, it was assumed that the Moora community had a high degree of flood awareness during the May 1999 flood event. Thus, in converting potential internal damages to actual internal damages, damage reduction factors of 0.3 and 0.2 respectively were assumed for residential and commercial properties. For industrial properties the damage reduction factor was changed to 0.70. For public authority and utility buildings a damage reduction factor of 0.8 was assumed. When converting potential external damages to actual external damages, a damage reduction of 0.2 was used for residential properties and 0.075 for commercial and industrial properties. For public authority and utility properties a damage reduction factor of 0.1 was used.

5.18 Tables 5.5 and 5.6 show details of the number of properties flooded in Moora and the associated flood damage in May 1999 as predicted by the FLDAMAGE model. The model predicts that some 317 residential and 49 commercial/industrial properties were flooded above ground level in May 1999. Of these, 92 residential and 25 commercial/industrial properties were flooded above floor level. These estimates are broadly consistent with available information. The predicted total flood damage for the May 1999 event is $1.07 million. Of this, residential properties incurred damages of $0.62 million. On this basis, the FLDAMAGE model for Moora is considered to be adequately calibrated.

Table 5.5 Estimated Number of Flooded Properties, Moora, May 1999 Flood Event

Property Type	Properties Flooded			Avg. Depth of Flooding (m)	
	AFL	BFL	Total	AFL	BFL
Residential	92	225	317	0.14	0.19
Commercial	22	18	40	0.18	0.18
Industrial	3	6	9	0.21	0.29
Public Authority	7	21	28	0.21	0.13
Public Utility	0	4	4	0.00	0.05
Total	124	274	398	0.18	0.18

Table 5.6 Estimated Flood Damage, Moora, May 1999 Flood Event

Property Type	Actual Direct Damage ($1000)				Indirect Damage ($1000)	Total Actual Damage ($1000)
	Internal	External	Structural	Total		
Residential	127	268	116	511	151	662
Commercial	57	4	14	75	149	224
Industrial	49	3	3	55	40	95
Public Authority	32	2	6	40	48	89
Public Utility	0	0	0	0	0	0
Total	265	277	139	682	388	1,070

5.5 EXISTING FLOOD DAMAGE

5.5a Average Annual Damage

5.19 Major flood events and consequent flood damage occur infrequently. For this reason, it is common practice to represent flood damage as an **average annual damage** which incorporates the relative probability and damage of a range of flood events. (Large floods with high damage costs occur less frequently than small floods with low damage costs). Average annual flood damage is determined by summing the product of annual flood probability and flood damage cost. When calculating average annual flood damages, the March 1999 flood event was assumed to have an average recurrence interval of 250 years.

5.20 The estimated average annual actual flood damage for properties in Moora under existing conditions is **$105,400**. It was assumed that the Moora community now has a high degree of flood awareness. Thus, in converting potential internal damages to actual internal damages, damage reduction factors of 0.3 and 0.2 respectively were assumed for residential and commercial properties. For industrial properties the damage reduction factor was changed to 0.70. For public authority and utility properties a damage reduction factor of 0.8 was assumed. When converting potential external damages to actual external damages, a damage reduction of 0.2 was used for residential properties and 0.075 for commercial and industrial properties. For public authority and utility properties a damage reduction factor of 0.1 was used.

5.5b 100 Year ARI Flood Event

5.21 Table 5.7 shows the number of residential, commercial and industrial properties flooded above floor level (AFL) and below floor level (BFL) by the 100 year ARI flood event (290 m^3/s) under 'existing conditions'.

- A total of 114 residential properties are flooded AFL and a further 245 properties are flooded BFL by the 100 year ARI flood event.
- Twenty-five commercial properties are flooded AFL.
- Three industrial properties are flooded AFL.
- Thirteen public authority and three public utility properties are flooded AFL.
- Most of the flooded properties are residential dwellings (72% of properties flooded AFL, 85% of properties flooded BFL).
- The average depths of flooding AFL are about 0.16 m for residential, 0.21 m for industrial and 0.22 m for commercial properties.
- The greatest depths of flooding AFL experienced by individual properties in Moora for the 100 year ARI flood event is 0.6 - 0.8 m.

Table 5.7 Details of Flooded Properties, Moora, 100 Year ARI Flood Event

Property Type	Properties Flooded			Avg. Depth of Flooding (m)	
	AFL	BFL	Total	AFL	BFL
Residential	114	245	359	0.16	0.18
Commercial	25	21	46	0.22	0.20
Industrial	3	6	9	0.21	0.32
Public Authority	13	17	30	0.21	0.18
Public Utility	3	1	4	0.05	0.20
Total	158	290	448	0.19	0.19

5.22 Table 5.8 shows the estimated damage associated with the 100 Year ARI flood event for existing conditions.

- The estimated total actual property damage is $1.532 M.
- About 56% of the total damage arises from residential properties. The average actual damage per flooded residential property is $2,400.
- About 30% of the total damage occurs to the commercial and industrial properties. Average actual commercial and industrial damages are $7,700 and $11,000 respectively per flooded property.
- Public authority and utility buildings suffer 14% of the total damage.

Table 5.8 Details of Estimated Flood Damage, Moora, 100 Year ARI Flood Event

Property Type	Actual Direct Damage ($1000)				Indirect Damage ($1000)	Total Actual Damage ($1000)
	Internal	External	Structural	Total		
Residential	175	295	190	661	202	863
Commercial	90	6	23	119	235	354
Industrial	50	4	4	58	41	99
Public Authority	55	5	12	72	89	161
Public Utility	30	0	0	30	25	55
Total	400	310	229	939	592	1,532

5.5c 50 Year ARI Flood Event

5.23 Table 5.9 shows the number of properties flooded and the estimated flood damage for the 50 Year ARI flood event (230 m^3/s) under existing conditions.

- A total of 59 residential properties are flooded AFL and a further 191 properties are flooded BFL by the 50 Year ARI flood event.
- Fourteen commercial properties are flooded AFL.
- Two industrial properties were flooded AFL.
- Five public authority properties are flooded above AFL.
- No public utility properties are flooded AFL.
- The estimated total actual property damage for this event is some $538,000.

Table 5.9 Details of Flooded Properties and Total Flood Damage, Moora, 50 Year ARI Flood Event

Property Type	Properties Flooded			Total Actual Damage ($1,000)
	AFL	BFL	Total	
Residential	59	191	250	362
Commercial	14	19	33	105
Industrial	2	4	6	12
Public Authority	5	15	20	59
Public Utility	0	1	1	0
Total	80	230	310	538

5.5d 20 Year ARI Flood Event

5.24 Table 5.10 shows the number of properties flooded and the estimated flood damage for the 20 Year ARI flood event (160 m^3/s) under existing conditions.

- A total of 9 residential properties are flooded AFL and a further 125 residential properties are flooded BFL by the 20 year ARI flood event.
- Nine commercial properties are flooded AFL for this event. No industrial properties are flooded AFL.
- Three public authority properties are flooded AFL for this event. No public utility properties are flooded AFL.
- The estimated total actual property damage for this event is $109,000.

Table 5.10 Details of Flooded Properties and Total Flood Damage, Moora, 20 Year ARI Flood Event

Property Type	Properties Flooded			Total Actual Damage ($1,000)
	AFL	BFL	Total	
Residential	9	125	134	59
Commercial	9	10	19	26
Industrial	0	3	3	0
Public Authority	3	7	10	24
Public Utility	0	1	1	0
Total	21	146	167	109

5.5e 10 Year ARI Flood Event

5.25 Table 5.11 shows the number of properties flooded and the estimated flood damage for the 10 Year ARI flood event (110 m^3/s) under existing conditions.

- No residential properties are flooded AFL by the 10 year ARI flood event. However, forty-six residential properties are flooded BFL for this event.
- No commercial, industrial, public authority and public liability properties are flooded AFL for this event.
- The estimated total actual property damage for this event is $12,000.

Table 5.11 Details of Flooded Properties and Total Flood Damage, Moora, 10 Year ARI Flood Event

Property Type	Properties Flooded			Total Actual Damage ($1,000)
	AFL	BFL	Total	
Residential	0	46	46	12
Commercial	0	10	10	0
Industrial	0	0	0	0
Public Authority	0	2	2	0
Public Utility	0	0	0	0
Total	0	58	58	12

5.5f 5 Year ARI Flood Event

5.26 No flood damage is suffered in Moora for flood events less than or equal to 5 year ARI (83 m^3/s). No properties are flooded AFL for this event. However, 11 residential and 8 commercial properties are flooded BFL by the 5 year ARI flood event.

5.5g Probable Maximum Flood Event

5.27 To estimate average annual damage for Moora an estimate was made of flood damage associated with the probable maximum flood event (6300 m^3/s).

5.28 All properties in Moora (approximately 750) will be flooded above floor level for the probable maximum flood event. This includes 604 residential, 69 commercial, 21 industrial properties, and 35 public authority and 7 utility properties. The total actual property damage associated with this event is estimated at some $22.133 M.

6 FLOODPLAIN MANAGEMENT OPTIONS

6.1 BACKGROUND

6.01 Modern floodplain management practice recognises three separate 'flood problems': the Existing Problem, the Future Problem and the Residual Problem.

- The **Existing Problem** refers to existing properties which are liable to flooding and flood damage.

- The **Future Problem** refers to those properties which upon development or redevelopment become flood-liable and susceptible to significantly higher levels of flood damage.

- The **Residual Problem** refers to the risk of flooding and flood damage that remains when all adopted floodplain management measures have been put in place. The residual flood risk – and the associated damage – can only be eliminated by designing for the probable maximum flood (PMF) event. In general, design for the PMF event is either economically or practically infeasible.

6.02 Different management measures are appropriate to each flood problem.

- **Structural measures**, e.g. retention basins, levees, channel widening and house raising, are a common and effective way of reducing damage, hazard and disruption associated with the Existing Problem.

- **Planning measures**, such as zoning and building regulations (e.g. minimum floor levels) are an effective means of reducing damage, hazard and disruption associated with the Future Problem.

- **Emergency response measures**, such as a flood warning, evacuation and recovery, are the only way of reducing damage, hazard and disruption associated with the Residual Problem.

6.03 It should be noted that the three types of management measures described above differ in cost-effectiveness.

- Structural measures **tend to be costly, but quite effective** in delivering protection up to the design flood event. However, when the structural measures are overtopped by a larger flood – and this is not a question of 'if', but of 'when' – the resulting damage and social disruption can be massive, especially when an appropriate contingency plan for dealing with this situation is not in place. The resulting economic and social disruption inflicted on the residents of Nyngan in New South Wales by the April 1990 Flood is evidence of this.

- Planning measures are the most **cost-effective** of all floodplain management measures, i.e. it is better to prevent or limit the problem to acceptable levels than to attempt to correct the problem after it occurs. Land use planning is a key floodplain management measure, the importance of which is increasingly recognised by all levels of Government.

- Emergency response measures are **superficially attractive**: they are less expensive than structural measures, but they are also less effective than structural or planning measures in reducing damage. The effectiveness of emergency response measures relies on the flood-liable population being effectively warned and knowing how to respond in an effective fashion to limit hazard to themselves and damage to their goods and possessions.

6.2 STRUCTURAL OPTIONS

6.04 After an initial appraisal of viable structural flood mitigation options for Moora, and subsequent discussions with the study advisory committee, five different structural options to reduce the flood risk in Moora were identified for detailed investigation:

- Detention basins in the upper catchment.
- Levee across the Moore River No. 4 Channel Bifurcation.
- Levees across the Yadgena Brook Bifurcation.
- Levees and diversion channels around the northern side of the town and/or around the southern side of the town.
- Widening of the Moore River No. 1 and 2 Channels.

Each of these structural options is discussed in detail below.

6.2a Detention Basins

6.05 Figure 6.1 shows the location of the six detention basins used in an assessment of the impact of detention basins on flooding at Moora. Table 6.1 provides details of the detention basins investigated.

Table 6.1 Summary Details of Detention Basins Investigated

Basin No.	Dam Location	Spillway Height Above Valley Floor (m)	Wall Length at Spillway Height (m)	Wall Length 2m Above Spillway (m)	Spillway Length (m)	Area Inundated at FSL (km^2)	Storage Volume at FSL (ML)
1	4.5 km D/S Cattady Road	7	1,200	1,660	50	7.2	15,110
2	1 Km U/S of Kitchin Bridge	8	1,400	1,550	50	5.8	16,200
3	Longpool - 2km U/S of Bridge	7	1,020	1,370	50	8.5	20,800
4	Roundhill - 6 km U/S Of Gauging Station	5	770	930	50	1.9	4,330
5	Roundhill Gauging Station	7	1,210	1,360	50	1.6	7,410
6	Nardy Road	3.5	2,460	3,100	50	4.3	6,280

6.06 Stage-area relationships for each detention basin were derived from 0.5 m contour maps of the Moore River catchment (supplied by WRC). In the analyses undertaken, each basin was assumed to have an overflow spillway and a concrete pipe low level outlet. Note that the type of low-level outlet was selected for design purposes only: final dam design could include any suitable form of low-level outlet.

6.07 Each detention basin was modelled independently. That is, it was assumed only one basin will be built (therefore, the effect of two basins acting simultaneously was not modelled). The URBS model was first run only for the March 1999 event with each of the detention basins in place. The spillway length and low-level outlet size for the basin were iteratively adjusted for each basin until an optimum basin size/flood attenuation combination was achieved. The URBS model was then run for the 100 year ARI event and other design events for each of the detention basins, using the optimised spillway length, wall height and low level outlet capacity.

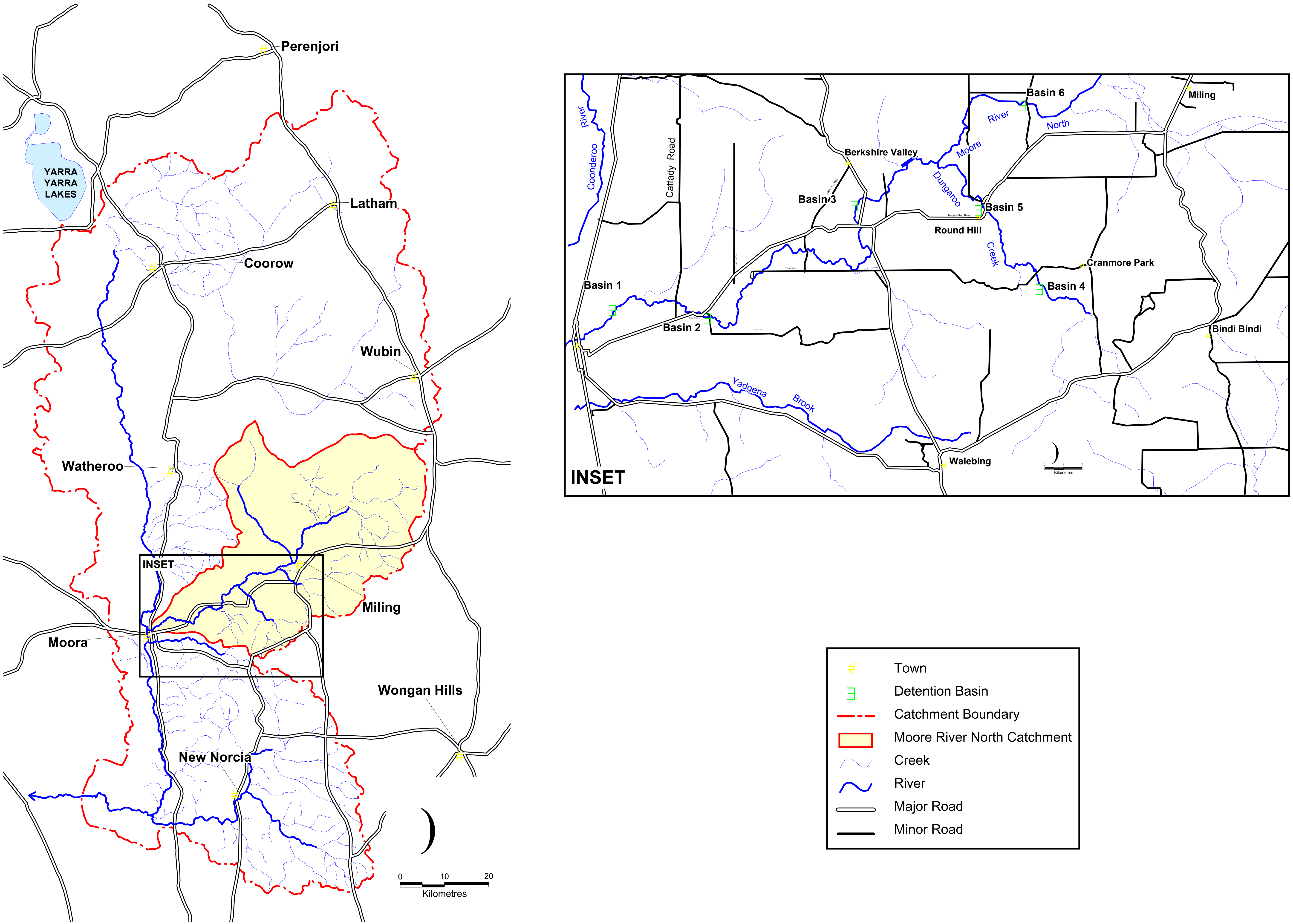

Figure 6.1 Potential Detention Basin Locations, Moore River Catchment

6.08 Table 6.2 shows the impact of each of the detention basins on the March 1999 and design flood discharges in the Moore River at Moora. The analyses showed that the most effective location for a detention basin is at either Site 1 (4.5 km downstream Cattady Road), Site 2 (1 Km upstream of Kitchin Bridge), or Site 3 (2 km upstream of Longpool Bridge). The other three basin locations were found to be ineffective.

Table 6.2 Impact of Detention Basins on Peak Flood Discharges at Moora Caravan Park

Flood Event	Peak Discharge at Moora (m^3/s)						
	Existing Conditions (With No Basins)	Basin 1 (4.5 km D/S Cattady Rd)	Basin 2 (1 km U/S Kitchin Bridge)	Basin 3 (Longpool 2 km U/S of Bridge)	Basin 4 (Roundhill, 6 km U/S of Gauging Station)	Basin 5 (Roundhill Gauging Station)	Basin 6 (Nardy Road)
PMF	6,300	-	6,300	6,300	-	-	-
March 1999	500	170	155	150	410	380	400
100 Year ARI	290	110	120	110	250	230	250
50 Year ARI	230	-	108	107	-	-	-
20 Year ARI	159	-	95	86	-	-	-
10 Year ARI	110	-	82	71	-	-	-
5 Year ARI	83	-	70	60	-	-	-
2 Year ARI	38	-	37	34	-	-	-

"-" denotes not analysed

6.09 Hydraulic analyses of the impact of detention basins on flood levels in Moora were only undertaken for basins located at Site 2 (1 km U/S of Kitchin Bridge) and Site 3 (2 km upstream of Longpool Bridge). The basin at Site 1 (4.5 km downstream Cattady Road) was omitted from further analysis since it had a similar impact on peak discharges at Moora as the basin at Site 2.

6.2b Moore River No. 4 Channel Bifurcation Levee

6.10 An old flood channel (Moore River No. 4 Channel) of the Moore River flows through the town to the south of the Moore River No 1 channel, as shown in Figure 4.2. Remnants of this channel are visible between Barber, Ranfurley and Melbourne Streets and along Dandaragan Street to the east of Lefroy Street. Filling and development have occurred along most of the length of this channel downstream of Ranfurly Street. In the late 1960's, a levee had been built across the Moore River No. 4 Channel to prevent overflows from Moore River No. 1 Channel flowing into this channel.

6.11 During the March 1999 flood, the levee across the Moore River No. 4 channel bifurcation was both outflanked and overtopped. The damaged levee has not been restored to date. As a result, even small floods can now flow into this channel to flood several houses along its length.

6.12 Figure 6.2 shows the surveyed cross section at the Moore River No. 4 channel bifurcation levee site. As shown there are two locations where the Moore River floodwaters enter the Moore River No. 4 channel. Hydraulic modelling indicates that the Moore River No. 4 channel will start to flow at discharges of about 20 to 30 m^3/s in the Moore River. In comparison, the Moore River No. 2 channel starts to flow at discharges of about 16 m^3/s in the Moore River.

6.13 It is recommended that the levee across the Moore River No. 4 bifurcation be re-constructed to about the existing ground level as shown in Figure 6.2. This levee will prevent nuisance flooding in the Berkshire Valley Road area. Scour protection, such as coarse rock rip rap should be provided along the downstream face of the levee to prevent it from eroding should it be overtopped again. The impact of this levee on flood levels and flood damage is presented in Section 7.

6.2c Yadgena Brook Levees

6.14 During March and August 1999 flood events, floodwaters from Yadgena Brook inundated several houses along Gardiner Street in the southern side of the Moora township. Site inspections undertaken by representatives from Water Studies Pty Ltd, Water and Rivers Commission and Moora Shire Council following the March and August flood events revealed that both Walebing and Mogumber Road crossings of Yadgena Brook had constricted flows causing flood water to be diverted along the respective roads towards the town.

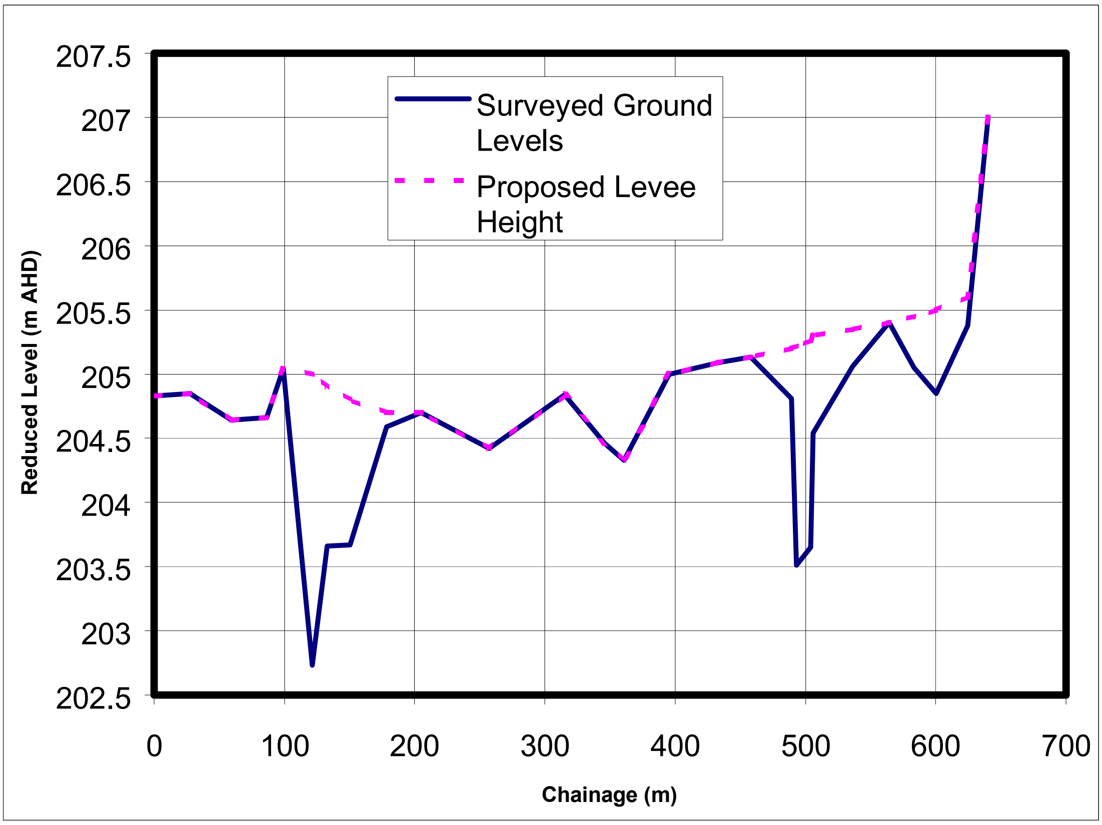

Figure 6.2 Surveyed Cross-Section of the Moore River No. 4 Branch Bifurcation Levee

(i) Proposed Levee Design

6.15 Figure 6.3 shows the location and length of the proposed levees to reduce the flood risk from Yadgena Brook. These locations were selected from 0.5 m contour data provided by WRC. The levee heights and lengths were based on hydraulic modelling results which indicate that Yadgena Brook is confined to a single channel upstream of Walebing Road for the design flood event (August 1999 flood event - see Appendix B for details). The results also indicate that floodwater only overflows Yadgena Brook along a length of about 200 m upstream of Walebing Road.

6.16 The hydraulic modelling results indicate that Yadgena Brook floodwaters do not flow along Mogumber Road towards Moora. However, because anecdotal evidence suggested that some floodwater did flow along Mogumber Road during the March flood, it is recommended that a levee be constructed at this location also to ensure problems do not occur in the future.

6.17 The two levees have been designed such that water will overtop the respective roads before it overtops the levee to flood the town. Based on hydraulic model results, the Walebing Road levee should be constructed to a level of 208.1 m AHD. The Mogumber Road levee should be constructed to a level of 205 m AHD. Surveyed ground levels indicate that the height of the Walebing Levee will be about 1.0 m at the road and will reduce to ground level about 300 m to the north east. The Mogumber Road levee varies in height from about 1.0 m near the railway to ground level about 200 m to the east.

(ii) Hydraulic Assessment of the Proposed Levees

6.18 Table 6.3 shows a comparison between the existing and the predicted peak flood levels along Yadgena Brook and at Hamilton Road when the proposed levees are in place for the design flood event. The results show that peak flood levels along Yadgena Brook will be increased by up to 0.33 m, and that no flooding of existing properties will occur at Moora from Yadgena Brook.

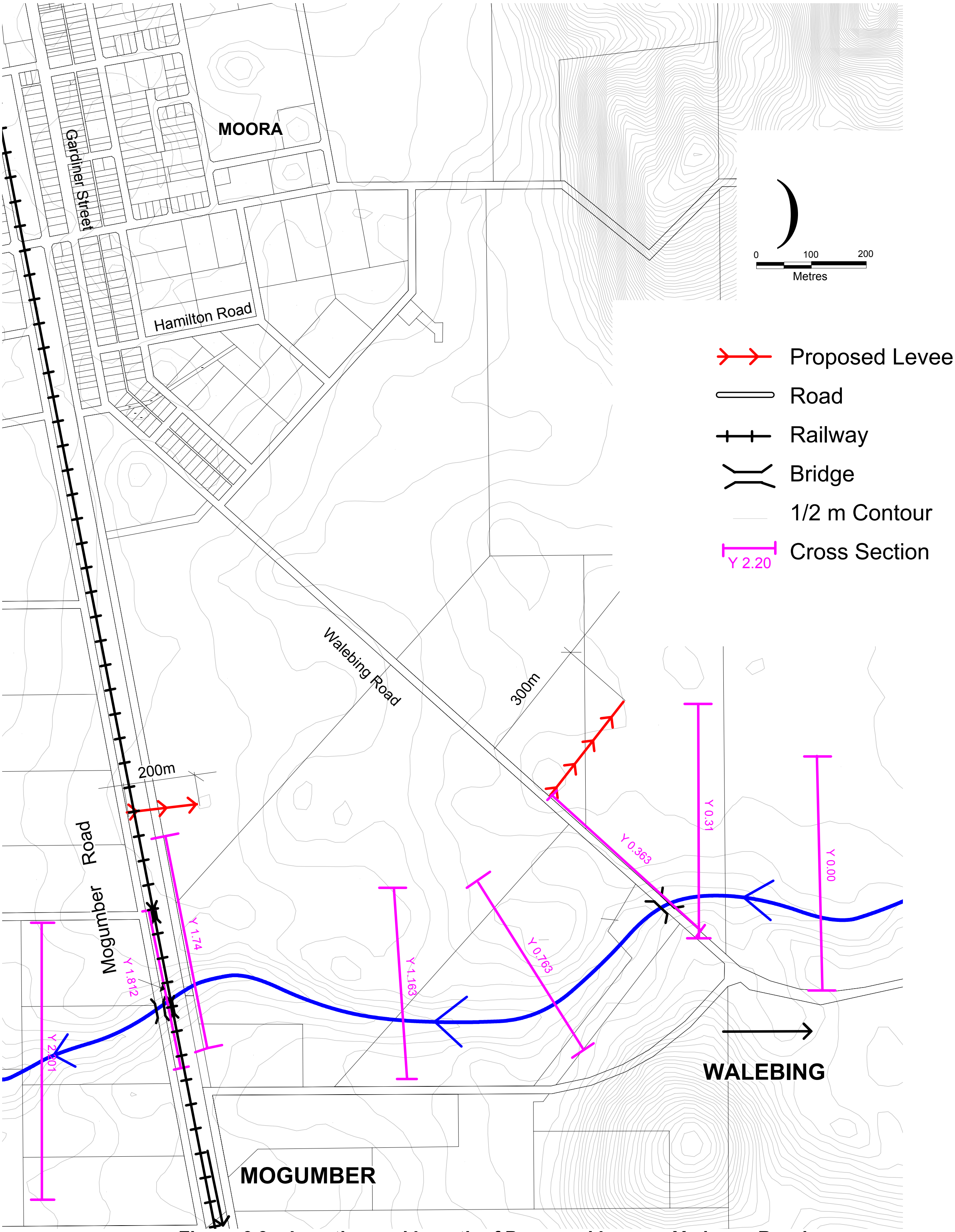

Figure 6.3 Location and Length of Proposed Levees, Yadgena Brook

Table 6.3 Estimated Peak Flood Levels along Yadgena Brook and at Hamilton Road with and Without the Proposed Levees, Design Flood Event

Location	Cross-Section No.[a]	Peak Flood Level (m AHD)		Increase in Peak Flood Level (m)
		Existing	With Levees	
Yadgena Brook U/S Walebing Road	Y0.363	207.47	207.8	0.33
Yadgena Brook D/S Walebing Road	Y0.763	206.52	206.66	0.14
Yadgena Brook U/S Mogumber Road	Y1.812	203.76	204.05	0.29
Moora at Hamilton Road	Y1.906	204.27	Not Flooded	-

[a] See Figure 6.4 for location of Cross-Sections

6.19 The model results also indicate that Walebing Road will be overtopped during the design event by about 0.05 m. The predicted peak flood level at Mogumber Road for the design event is below the road level. However, floodwater may flow along Mogumber Road towards the town at this level if a levee is not constructed.

6.20 Moora Shire Council provided surveyed ground levels and floor levels of 9 properties adjacent to Yadgena Brook. Hydraulic model results indicate that none of these properties will be flooded above floor level from the design event if the levees are constructed.

6.2d Flood Diversion Channels and Associated Levees

6.21 Two options for flood diversion drains and associated levees were considered to reduce the flood risk in Moora. The first option involves the diversion of upstream floodwaters to the north of Moora into the Coonderoo Lakes System. The second option involves the diversion of upstream floodwaters to the south to rejoin the Moore River downstream of the town.

6.22 For both levee options, it was assumed that pipe culverts would be constructed where the levees cross the Moore River No. 1 and 2 branches. The pipe culverts were designed such that the flow regime in the Moore River No. 1 and 2 branches are not altered up to discharges equivalent to about 10 Years ARI, at which level the flood damage to properties commences.

(i) Northern Diversion Drain and Levee

6.23 Figure 6.4 shows the location and alignment of the proposed Northern Diversion drain and levee and the expected extent of flooding from the March 1999 flood, if it had occurred after the construction of the proposed drain and levee. The extent of flooding was estimated by conservatively assuming that Coonderoo Lakes storage effects are negligible. The main features of the Northern Diversion drain and levee are as follows:

- The length of the proposed levee and drain is about 4,800 m.
- The size of the drain was assumed to be equivalent to the material required to construct the levee at that location.
- The crest of the levee was assumed to be 0.3 m above the March 1999 flood level at 206.6 m AHD near the Moore River No. 1 branch. The maximum height of the levee between the Moore River No. 1 and No. 2 branches is about 2.1 m above ground level.
- The approximate volume of excavated material required for the levee is 60,000 m^3.
- Five 1.8 m diameter culverts are required (through the levee) at the Moore River No. 1 Branch crossing.
- One 1.8 m diameter culvert is required (through the levee) at the Moore River No. 2 Branch crossing.

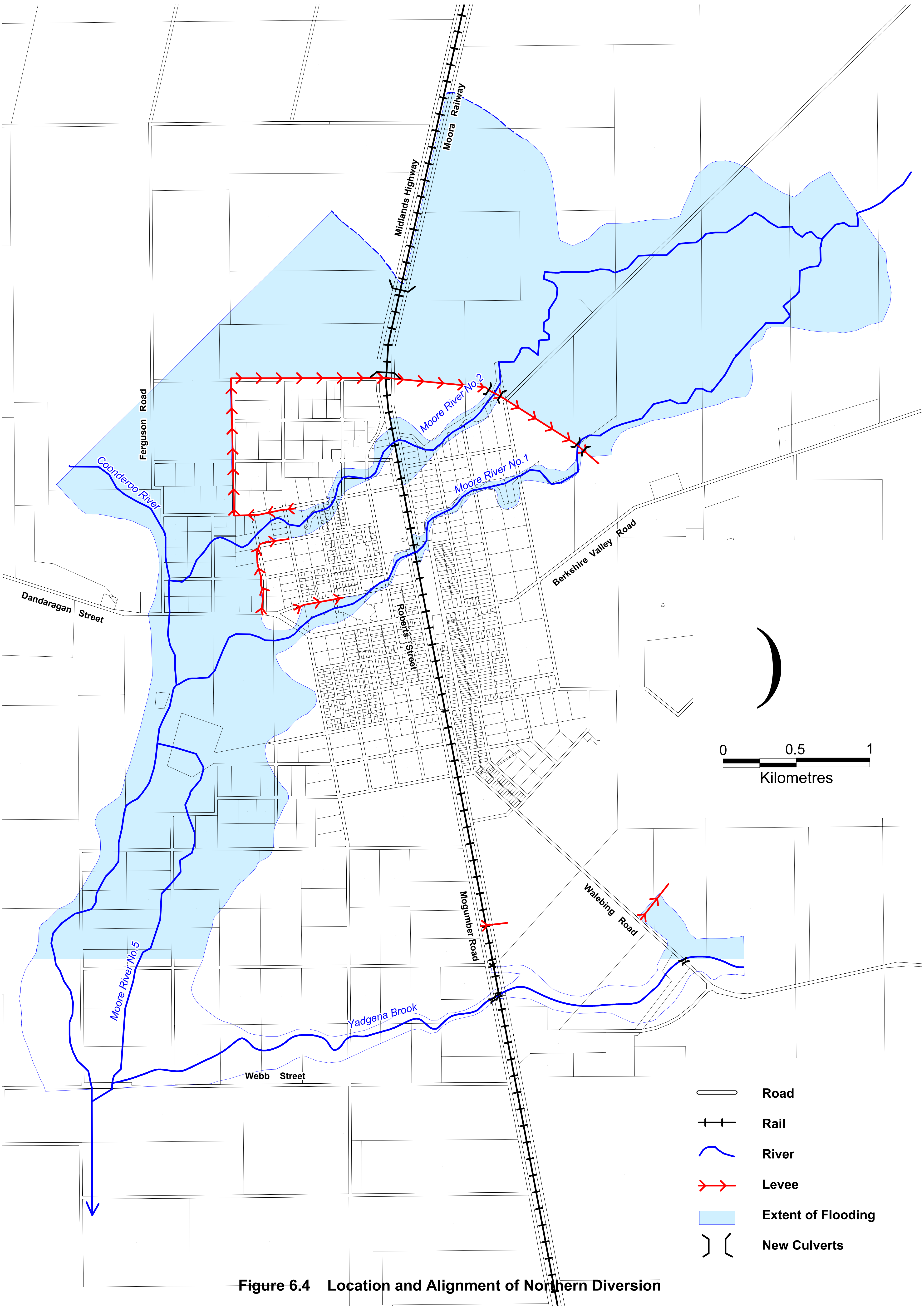

Figure 6.4 Location and Alignment of Northern Diversion

- Fifty 1.2 m x 1.5 m box culverts are required at the Moore No. 3 Branch crossing at the railway line. In addition, the rail level needs to be lifted by approximately 1.5 m to prevent it being overtopped.
- The Midlands Highway across the Moore River No. 3 Branch needs to be lowered to ground level to act as a causeway during floods.
- A spillway should be incorporated in the levee over the Moore River No. 1 Branch to facilitate controlled overtopping during more extreme floods than the March 1999 event.

6.24 Figure 6.5 shows that most properties in Moora are flood free for the March 1999 flood event after the construction of the proposed diversion drain and levee. Hydraulic modelling indicates that about 85% of the peak discharge in the March 1999 flood would have been diverted to the north of Moora had this levee been in place. Some properties to the east of Long and Bishop Streets in North Moora remain at risk to flooding. Some properties in this area that were not flooded during March 1999 will be flooded if this levee is constructed. This area is required to convey the additional flood flows resulting from the levees.

6.25 An important consideration of this option is that little to no warning would be provided to the residents of Moora should the levee breach during a large flood. If the levee breached at the peak of the March 1999 flood, a 2.0 metre high wall of water could potentially flow into Moora. The consequences of this occurring would be severe and catastrophic. Regular maintenance of the levee would be required to maintain its structural integrity.

(ii) Southern Drain and Diversion Levee

6.25 Figure 6.5 shows the location and alignment of the proposed southern drain and diversion levee and the expected extent of flooding from the March 1999 flood, if it had occurred after the construction of the proposed drain and levee. The main features of the proposed levee are as follows:

- The proposed levee and drain is about 7,000 m long.
- The drain between the high school and Mogumber Road is up to 4 m deep and 30 m wide at the base.
- The approximate volume of excavated material required to construct the drain and levee is 400,000 m^3.
- The crest level of the levee near the Moore River No. 1 Branch crossing is 206.4 m AHD. This is about 2.5 m above natural surface level at this location.
- Five 1.8 m diameter culverts are required (through the levee) at the Moore River No. 1 Branch crossing.
- One 1.8 m diameter culvert is required (through the levee) at the Moore River No. 2 Branch crossing.
- Berkshire Valley Road will have to be raised 2.5 m to get over the proposed levee.
- Vehicle access for the residents along Atbara and Saleeba Streets is provided to Berkshire Valley Road.
- Forty metre long railway and road bridges will be required across the proposed drain at the Moora railway line and Mogumber Road crossings. It was assumed that Walebing Road will be closed and traffic will be diverted to Mogumber Road to reduce costs.
- A spillway should be incorporated in the levee over the Moore River No. 1 Branch to facilitate controlled overtopping during more extreme floods than the March 199 event.

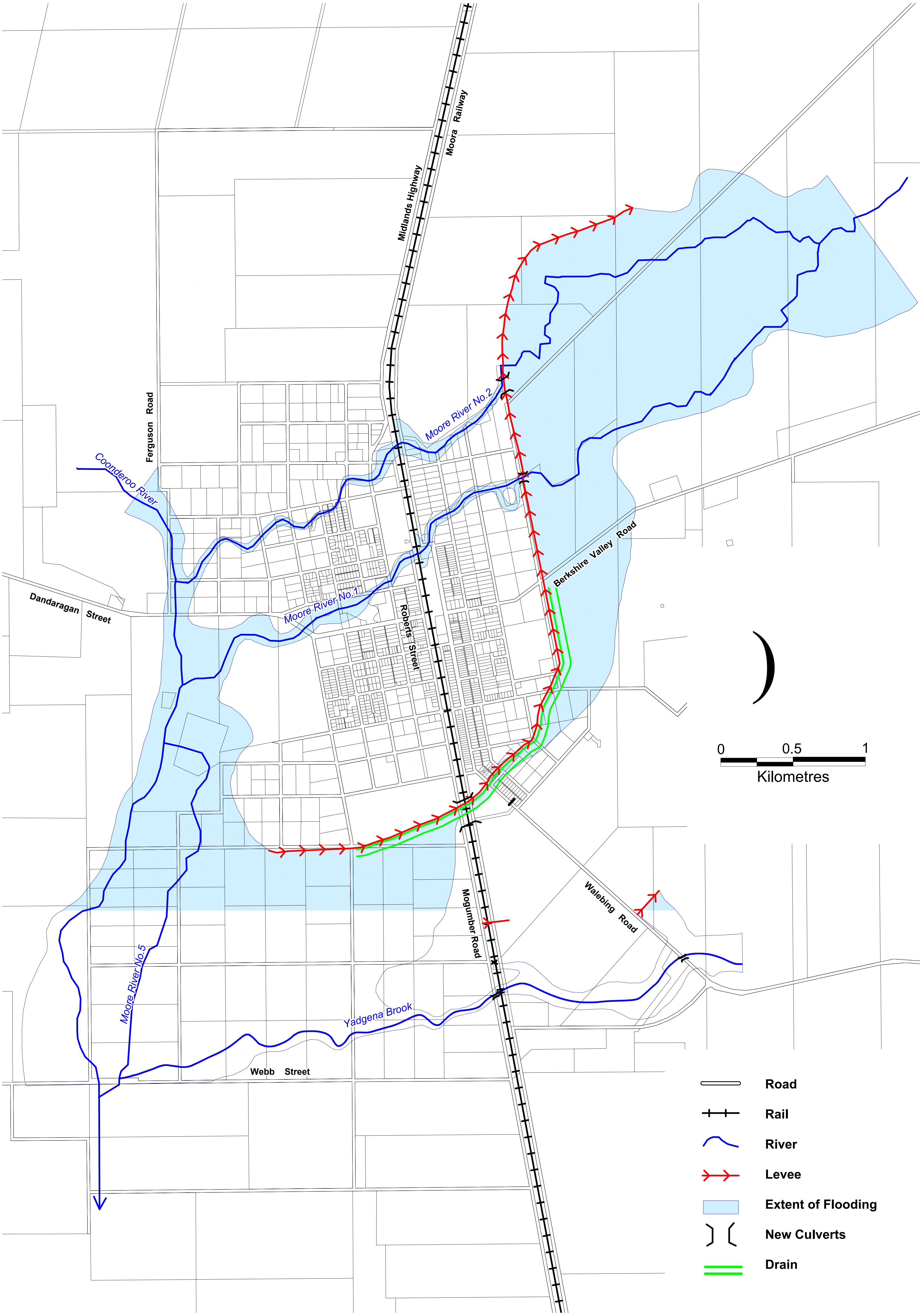

Figure 6.5 Location and Alignment of Southern Diversion Drain and Levee

6.26 Figure 6.5 shows that no houses would have flooded in Moora by the March 1999 flood, if it had occurred after the construction of this drain and levee. However, some of the existing houses along Atbara and Saleeba Roads may need to be removed to make way for the diversion drain and levee.

6.28 As explained in the previous section on the northern levee option, the consequences of a breach of this levee would be severe.

6.2e Channel Widening

6.27 An assessment of the impact on flood levels of channel widening was undertaken by increasing the width of the main channels of the Moore River No. 1 and 2 branches by 20 m and 10 m respectively. It was assumed that none of the existing hydraulic structures will be upgraded as part of this widening.

6.28 Approximately 280,000 m^3 of material will be excavated from the existing channel banks for this option.

6.3 NON-STRUCTURAL OPTIONS

6.29 Non-structural or soft options for managing flooding involve a community response to reduce flood damages through land and building controls (eg. appropriate zoning) or reducing the potential for flood damages (eg. flood warning). Non-structural options involve managing the known flood impacts, and can generally be effectively implemented at a local level using existing Council powers and procedures.

6.30 The following non-structural options were considered for flood management in Moora:

- Land use zoning controls;
- Building and development controls;
- Floodproofing of buildings, including house raising;
- Public awareness education;
- Flood forecasting, warning and evacuation;
- Voluntary purchase.

6.31 A proposed strategy for each non-structural option has been developed assuming that only the Yadgena Brook and Moore River No. 4 bifurcation levee structural options are implemented. Some of the strategies outlined may not be relevant should other structural options be implemented.

6.32 Based on consultations with WRC and Moora Shire Council (MSC), the March 1999 flood event has been adopted as the 'Defined Flood Event' for the assessment of non-structural options in this study. Figure 5.1 shows the extent of inundation from the March 1999 flood if the Yadgena Brook and Moore River No 4 bifurcation levees had been constructed. The area flooded by the March 1999 flood has been classified as 'flood liable' for the purpose of developing non-structural options for Moora. Larger floods may flood surrounding areas. However, stringent controls have not been proposed for these surrounding areas on the premise that the flood risk in these areas is low.

6.3a Land Use Zoning Controls

(i) Purpose

6.33 The application of land use zoning is an effective and long-term means of controlling development in flood affected areas. Future flood damages can be minimised by restricting or preventing incompatible development on flood-liable land.

(ii) Considerations

6.34 Land use zoning over flood liable land should be based on an objective assessment of hazard, environmental and other factors, including:

- The objectives of the floodplain management plan;
- Whether the land is in a high hazard or floodway category;
- Potential for future development to have an adverse impact on flood behaviour and thereby on existing development;
- Whether adequate access is available during floods;
- Whether certain activities should be excluded because of additional or special risk to their users, eg. accommodation for aged people, hospitals and the like;
- Existing planning controls.

(iii) Proposed Strategy

6.35 The potential for an increase in future flood damage in Moora is high if the existing land use zonings are maintained. It is recommended that future residential, commercial, industrial or public utility development in Moora be encouraged to locate in a flood free area, ie outside of the flood liable zone. New residential, commercial, and industrial zones should be created in flood free areas to plan and manage the growth in these areas. Infrastructure such as roads, water and sewerage should be provided to these areas to facilitate the growth.

6.36 It is likely that the community will want some development to continue within the flood liable land. To minimise the potential flood hazard and damage resulting from development in these areas, it is recommended that zones of high (floodway) and low (flood fringe) flood hazard be defined as shown in Figure 5.1. Stringent building and development controls should be imposed upon development in both areas as outlined in Section 6.3b.

6.37 The following land use zoning changes are recommended for the flood fringe areas:

- Limit the size of the existing industrial zone to the existing level of development.
- Prevent the subdivision of larger residential allotments.
- Relocate areas zoned for future public infrastructure, such as hospitals, to a flood free area.

6.3b Building and Development Controls

(i) Purpose

6.38 At the development consent and building consent stages of a proposal, appropriate conditions can be imposed to ensure the development is compatible with the prevailing flood situation and that the overall level of potential flood damages is not significantly increased.

(ii) Considerations

6.39 The components of relevance to flooding in Moora Shire Council's existing town planning scheme is based on recommendations made in the previous flood study of Moora (GHD, 1991). The Council has adhered to these controls since 1991. The current controls include:

- Prevention of development in an area designated as a floodway.
- Restriction of development in flood fringe areas up to the extent of the 100 Year ARI flooding.
- Imposition of a minimum finished floor level of 0.5 m above the 100 Year ARI flood level for all new residential developments.

6.40 To date, there has been a preference for new residential developments to be constructed on earth pads. If this practice is continued, the cumulative impact of these earthen pads will result in higher peak flood levels in Moora.

6.41 The findings of this study should supersede the findings of the previous study. It is also noted that the March 1999 flood event has been adopted as the new 'flood standard' for Moora. The March 1999 flood levels in Moora on average are about 0.5 m higher than the previous 100 Year ARI flood level.

(iii) Proposed Strategy

6.42 The changes proposed below to MSC's Planning Scheme have been developed in consultation with the Council on the basis of guidelines given in SCARM (2000).

- Any proposed development within the high hazard (floodway) areas should have a hydraulic assessment to determine its impact on flood flows and flood levels. Any development proposal found to have an adverse impact on peak flood levels at neighbouring properties should not be accepted. This assessment should be made by a suitably qualified neutral person such as a representative of WRC.
- The minimum finished floor level (FFL) of new habitable buildings should be set at 0.5 m above the March 1999 flood level.
- For non habitable buildings such as sheds, industrial and commercial sites;
  - The minimum FFL should be 0.15 m above the March 1999 flood level.
  - Power points, electrical or data connection outlets should be installed 1.0 m above floor level.
  - Windows should be installed no lower than 0.5 m above the March 1999 flood level.
  - Septic tank disposal of waste should not be allowed when a connection to the sewer is available.
  - Chemical storage areas should have a minimum FFL of 0.5 m above the March 1999 flood level.
  - Breather inlets to underground storage tanks should be 0.5 m above the March 1999 flood level.
- All new development on allotments smaller than 2,000 m^2 must be constructed using a high base foundation structure (not on earth pads). Development underneath these structures should not be approved.
- Solid fences should be discouraged on existing developments and not approved on new developments.
- For new developments on allotments greater than 2,000 m^2, an earth pad foundation may be permitted, provided less than one quarter of the lot is being filled.

6.3c Floodproofing of Buildings, Including House Raising

(i) Purpose

6.43 Floodproofing involves raising the floor levels of existing dwellings above the flood level of the flood standard ('house raising') and/or using flood compatible materials and appropriate structural designs to reduce structural flood damage.

(ii) Considerations - House Raising

6.44 The raising of houses after construction as a floodproofing option is usually only feasible for timber-framed buildings with timber floors. Masonry or masonry veneer structures cannot be raised without major structural alterations.

6.45 Freeboard for floor levels is required to allow for uncertainties in the hydrological and hydraulic modelling procedures, differences in water level across the floodplain, effects of wave action and the effect of any subsequent infill development. Freeboard should not be considered as protection against larger floods, although to some extent it may serve this purpose.

6.46 Table 6.4 shows the number of floodprone houses suitable for raising in Moora in areas flooded by various flood events. Note that this analysis does not include the houses raised between the March 1999 flood event and the time of the flood damage survey in November 1999. Table 6.4 also shows the potential reduction in flood damage (i.e. savings) to these houses for different floods if they are raised. Note that if these houses are raised, they would not suffer internal and structural damages, but the properties would still suffer external damages. Also shown in Table 6.4 is the estimated cost of raising these houses, based on a value of $10,000 per dwelling.

Table 6.4 Details of Numbers, Savings and Costs of House Raising in Moora

Flood Event	Reduction in No. of Houses Flooded AFL	Potential Reduction in Flood Damage from House Raising ($)	Cost of House Raising ($)
March 1999	87	$792,000[a]	$870,000
May 1999	21	$78,000	$210,000
100 Year ARI	25	$105,000	$250,000

[a] Assumes low flood awareness as described in Section 2.3a.

(iii) Considerations - Materials and Design

6.47 Much of the flood damage that occurs to the structure of a residential, commercial or industrial building can be reduced by various floodproofing measures aimed at minimising the damaging effects of floodwaters on the materials of the building and to the structure. Particular methods of construction and certain types of materials are better able to withstand the effects of immersion. For example, plasterboard and chipboard, common materials in the internal linings and built-in cupboards and fittings of a house, can be badly damaged on immersion and may have to be replaced. In contrast, double brick construction can withstand immersion and may only require a hose and scrub down when the flood subsides.

6.48 To prevent gross structural damage from flooding, developments should be designed to withstand water pressure, forces from debris and flotation.

(iv) Proposed Strategy

6.49 Table 6.4 indicates that the cost of house raising is higher than the potential savings in flood damage associated with house raising. Thus community funding for raising all flood prone houses is not recommended. However, funding for an individual property may be appropriate if it can be proved that it is in a high risk area (floodway) and has a high potential internal damage cost.

6.50 MSC currently has no control over the selection of building materials for new developments or renovations. It is recommended that Council's publication entitled 'Guide to Building in Shire of Moora' include a section on appropriate building materials suitable for buildings in flood liable areas. Such a brochure providing information on appropriate building materials will also be useful for residents planning renovations to existing homes and buildings.

6.3d Flood Forecasting, Warning and Emergency Response

(i) Purpose

6.51 If sufficient notice is given prior to major flooding, the timely evacuation and removal or raising of contents from properties prior to inundation can be achieved in many cases. The purpose of a flood forecasting system is to predict the likely size and extent of a flood before it occurs to improve warning and evacuation lead times.

6.52 The purpose of a flood warning system is to warn a community of an impending flood. The purpose of evacuation planning is to make people aware of when and how they should evacuate themselves and their possessions, and where they should go when a flood eventuates.

(ii) Considerations

6.53 The effectiveness of this management option depends upon a number of factors:

- The effective flood warning time must be adequate so that people who are likely to be flood-affected can initiate evacuation procedures;
- The level of public awareness must be such that people will accept and act on flood warnings and advice to evacuate; and
- The availability of a flood free location to receive evacuated people and possessions.

6.54 An effective flood forecasting and warning system can reduce risk to life and property damage. It is generally effective when used in conjunction with other options. Although implementation costs are modest, additional resources and equipment have to be committed to develop an effective system.

6.55 The effectiveness of a flood warning system depends on how well public awareness has been maintained. Public awareness is difficult to maintain when moderate to major flooding does not occur for a long period of time or when there is significant population turnover. It is noted that the effectiveness of flood warning decreases if floodwaters rise quickly and predictions are unreliable.

(iii) Proposed Strategy

6.56 WRC has established a flood warning system in the Moora catchment in the aftermath of the 1999 floods. A network of 5 rainfall stations have been established across the Moore River catchment. These stations automatically alert both the Commonwealth Bureau of Meteorology (CBM) and WRC if more than 10 mm of rainfall is recorded in a 24 hour period. Four river gauging stations have also been established in the catchment. These stations record water levels continuously in the Moore River at various locations. Recorded water levels are posted daily on the WRC's Internet site (http://www.wrc.wa.gov.au/waterinf/telem/moore.htm). Key steps of the WRC flood warning system are as follows:

1. When weather forecasts are adverse or once a rainfall alert has been issued by one of the rainfall stations, the CBM system will automatically call up the rainfall stations and forward this information to WRC.
2. WRC will call up the river gauging stations to determine the current river levels.
3. Rainfall and river flow will be continually monitored (at 1 – 4 hourly intervals) to review and update the forecast of peak flood levels in Moora.
4. If flooding is expected in Moora, WRC will provide advice as appropriate to MSC, SES, Police and CBM.
5. CBM will provide local and general flood warnings to the general public and other stakeholders throughout the event.

6.57 MSC has developed a preliminary simple and concise three step flood emergency plan for local residents. The plan has been distributed to all residents in the form of a fridge magnet. The three step plan is as follows:

1. Activate fire, football and police sirens to provide a first warning to local residents. Upon receiving this warning, residents are advised to prepare for the flood by shifting furniture and pack.

2. Activate a second siren to prompt residents to turn off the gas and evacuate to the Central Midlands Senior High School. In addition, residents are urged to register at the evacuation centre if they are leaving Moora.

3. Activate a final warning. All residents are expected to be evacuated by this time.

6.58 To ensure correct implementation of the above flood emergency plan, it is recommended that the role of Council and SES representatives be explicitly defined and documented. It is also recommended that the plan contain information on:

- The WRC contact names and numbers.
- The person responsible to activate each of the sirens.
- The people responsible for the evacuation.
- The location of keys to the evacuation centre.
- The person responsible for the operation of evacuation centre.
- The people assigned to assist in the evacuation of hospital and nursing home residents.
- The names of backup people to undertake the above roles should assigned people be absent.

6.59 The plan should be reviewed and practiced on an annual basis and updated as required.

6.3e Public Awareness and Education

(i) Purpose

6.60 Appropriate and timely public response at times of flood is related to the level of understanding in the general community of the nature, frequency and extent of flooding, the rate of rise of flood waters and the degree of risk. Therefore, public awareness and education programmes with respect to the risk, hazard, response, cleanup and recovery of floods should be an integral and ongoing part of managing flood affected areas. These are all issues that need to be addressed in the flood emergency plan.

(ii) Considerations

6.61 Following the 1999 flood events the local community should have a high level of flood awareness at present. However, a continuing public education programme is recommended on the basis that a well prepared community will suffer less damage and other flood related problems during a significant flood event.

6.62 A public education process should:

- Improve and maintain general flood awareness in the community;
- Outline flood warning and evacuation plans, i.e. flood emergency management;
- Outline the basis of adopted floodplain management plans.

6.63 Public education is relatively inexpensive and has the potential to reduce the risk to life and property. Significant flood events are infrequent. Thus, a programme of public information must be ongoing and sustained if it is to be effective. Council must regard public education as an ongoing 'maintenance cost' of proper floodplain management.

6.64 The education process can be undertaken by various means including flood signs and markers indicating historical flood levels or predicted flood levels, media releases, special commemoration services on the anniversary of the March 1999 flood, information packages via mail drops or with rate notices and public displays.

(iii) Proposed Strategy

6.65 In addition to measures described above, it is recommended that the disaster alert fridge magnet be sent out to the community annually, perhaps on the anniversary of the March 1999 flood.

6.3f Voluntary Purchase

(i) Purpose

6.66 In certain high hazard areas of the floodplain it may be impractical or uneconomic to mitigate the flood hazard. In such circumstances it may be appropriate to cease occupation of the properties at risk, in order to free both residents and potential rescuers from the danger and cost of future floods. This is achieved by the purchase of the properties and removal or demolition of the improvements as part of a flood mitigation scheme

(ii) Considerations

6.67 Where voluntary purchase is implemented, the property should be purchased at an equitable price and only where voluntarily offered. Development should be removed and such areas should ultimately be rezoned for a flood compatible use.

6.68 On one hand, voluntary purchase eliminates losses and threat to life associated with occupation of affected buildings. In addition, it allows change in land use to a more flood compatible usage, providing a community benefit. eg. playing fields, parkland, open space. On the other hand, costs associated with voluntary purchasing could be high, although the government generally provides subsidies where the voluntary property purchase is part of an approved scheme. In addition, such schemes are slow to implement as they are dependant upon a property being voluntarily offered. Funds for such schemes are usually limited due to other statewide priorities.

(iii) Proposed Strategy

6.69 There are currently several residential buildings located within the designated floodway. None of these properties are flooded above floor level for floods up to the 20 Year ARI flood severity. Noting the difficulties outlined above, and the fact that floodwaters in these areas move relatively slowly and adequate time is now provided by improved flood forecasting and warning to evacuate residents, the acquisition of these properties is not considered to be a high priority.

7 IMPACT OF STRUCTURAL FLOOD MITIGATION OPTIONS

7.1 IMPACT ON FLOOD LEVELS

7.01 The Moora Mike 11 model was used to investigate the impact of the six structural mitigation options given below on peak flood levels throughout Moora for the 5, 10, 20, 50 and 100 Year ARI design events and the March 1999 event:

- Option 1: Moore River No. 4 Bifurcation Levee
- Option 2: Detention Basin at Longpool
- Option 3: Detention Basin at Kitchin Bridge
- Option 4: Northern Diversion Drain and Levee
- Option 5: Southern Diversion Drain and Levee
- Option 6: Channel Widening

7.02 The Yadgena Brook levees (see Section 6.2c) were assumed to be included in all of the above six options. In addition, for all options, except the Diversion Drain and Levee options (Options 4 and 5), the Moore River No. 4 Bifurcation Levee was assumed to be included.

7.03 Figure 7.1 shows peak March 1999 flood levels along the Moore River No. 1 Branch for the various options. Table 7.1 shows the reduction in peak flood level at Moora Caravan Park (GS 617010) for each option for the different flood events. Appendix F provides details of the changes in peak flood levels at each cross-section for each option and for the various design events.

Table 7.1 Reduction in Peak Flood Level at the Moora Caravan Park (GS 617010) For Different Structural Mitigation Options

Option No.	Description	Reduction in Peak Flood Level (m)					
		5 Yr ARI	10 Yr ARI	20 Yr ARI	50 Yr ARI	100 Yr ARI	March 1999
1	Moore River No. 4 Bifurcation Levee	-0.07	-0.07	-0.05	0	0	0
2	Longpool Detention Basin	0.18	0.25	0.32	0.38	0.33	0.43
3	Kitchin Bridge Detention Basin	0.07	0.12	0.24	0.37	0.38	0.40
4	Northern Drain and Levee	0.16	0.34	0.53	0.69	0.71	0.69
5	Southern Drain and Levee	0.20	0.34	0.52	0.68	0.71	0.78
6	Channel Widening	0.53	0.46	0.39	0.2	0.08	0.07

7.04 Conclusions drawn from results presented in Table 7.1, Figure 7.1 and Appendix F are summarised below.

7.1a Option 1 - Moore River No. 4 Bifurcation Levee

7.05 The construction of the Moore River No 4 Bifurcation Levee marginally increases peak flood levels at some locations in Moora for the smaller flood events. At the nearest property upstream of the levee (Isbister's homestead), the peak March 1999 flood level will be increased by about 0.02 m. However, the construction of the levee reduces the extent of flooding along the Moore River No. 4 branch markedly for the small floods. For the larger events, the levee has an insignificant impact on flood levels at Moora.

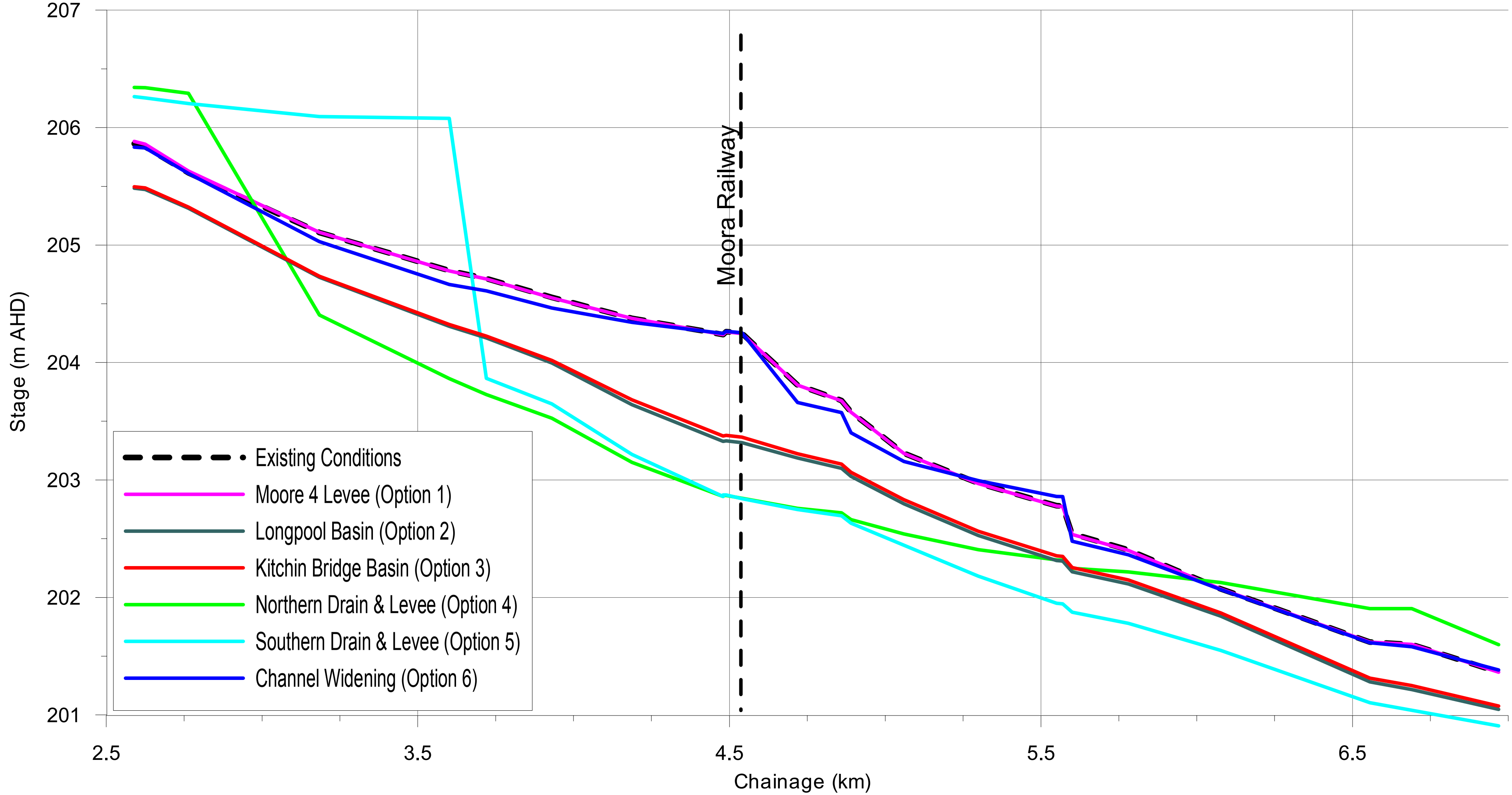

Figure 7.1 Impact of Proposed Structural Options on Peak Flood Levels along Moore River No. 1 Branch, March 1999 Flood

7.1b Option 2 - Detention Basin at Longpool

7.06 The Longpool detention basin reduces peak March 1999 flood levels along the Moore River No. 1 Branch by up to 0.93 m upstream of the Moora Railway and up to 0.47 m downstream of the railway. The extent of flooding will also reduce markedly. For smaller events, this option has a smaller but still significant impact on peak flood levels (see Appendix F).

7.1c Option 3 - Detention Basin at Kithcin Bridge

7.07 The Kitchin Bridge detention basin has a similar impact on peak flood levels to the Longpool basin (Option 2). For smaller events such as the 5 and 10 Year ARI events the impact is smaller than for Option 2 (see Appendix F).

7.1d Options 4 and 5 - Northern and Southern Diversion Drains and Levees

7.08 The northern and southern drain and levee options provide the best level of flood immunity for the town (up to the level of the March 1999 event). However, the peak March 1999 flood level at the nearest upstream property of the levees (Isbister's homestead) will increase by about 0.75 m for both diversion options.

7.1e Option 6 - Channel Widening

7.09 Channel widening will significantly reduce peak flood levels for small floods. However, the reduction in flood level becomes progressively smaller with increasing flood magnitude. For floods larger than 50 year ARI, the reduction in peak flood levels are small. The results also show that upstream of the Moora Railway and Dandaragan Street bridges, peak flood levels will increase unless these crossings are upgraded.

7.2 IMPACT ON FLOOD DAMAGE

7.10 The effect of the various structural mitigation options on the number of properties flooded AFL for the defined flood event (March 1999 flood) is shown in Table 7.2. Table 7.3 shows the effect of the various structural options on flood damages for the defined flood event. Table 7.4 shows the effect of the various options on Average Annual Actual Flood Damage (AAAFD).

Table 7.2 Effect of Structural Options on Number of Properties Flooded AFL, March 1999 Flood Event, Moora

Property Type	Properties Flooded AFL						
	Existing Condition	Option 1 (Moore No. 4 Levee)	Option 2 (Longpool Basin)	Option 3 (Kitchin Basin)	Option 4 (Northern Diversion)	Option 5 (Southern Diversion)	Option 6 (Channel Widening)
Residential	309	298	4	15	46	0	41
Commercial	52	55	8	8	0	0	44
Industrial	19	19	2	2	0	0	17
Public Authority	31	31	3	3	0	0	26
Public Utility	6	6	0	0	0	0	5
Total	417	409	17	28	46	0	333

Table 7.3 Effect of Structural Options on Flood Damage, March 1999 Event, Moora

Property Type	Flood Damage ($1,000)						
	Existing Condition	Option 1 (Moore No. 4 Levee)	Option 2 (Longpool Basin)	Option 3 (Kitchin Basin)	Option 4 (Northern Diversion)	Option 5 (Southern Diversion)	Option 6 (Channel Widening)
Residential	2,896	2,884	38	89	459	0	2,261
Commercial	1,331	1,387	24	26	0	0	924
Industrial	1,992	1,214	9	9	0	0	911
Public Authority	635	679	22	23	0	0	402
Public Utility	522	653	0	0	0	0	282
Total	6,576	6,817	93	147	459	0	4,780

Table 7.4 Effect of Structural Mitigation Options on Average Annual Actual Flood Damage, Moora

Situation	AAAFD ($1,000)
Existing Conditions	$ 105,400
Option 1: Moore River No. 4 Branch Bifurcation Levee	$ 102,500
Option 2: Longpool Detention Basin	$ 45,900
Option 3: Kitchin Bridge Detention Basin	$ 46,400
Option 4: Northern Diversion	$ 46,800
Option 5: Southern Diversion	$ 42,400
Option 6: Channel Widening	$ 74,000

7.11 All of the flood mitigation options have a measurable effect on average annual flood damage in Moora. The Moore River No. 4 Bifurcation Levee coupled with either of the detention basin options or either of the diversion options are the most effective measures in reducing both the number of properties flooded above floor level and the total actual damage for the defined flood. For the detention basin options, the March 1999 flood will overtop the main channels of the Moore River No. 1 and 2 Branches.

7.12 For the Northern Diversion Drain and Levee option, the properties affected by flooding are located in unprotected areas along the western end of Riley, Clarke and Dandaragan Streets and Stack and Cooper Streets. For the Southern Diversion Drain and Levee option, the houses at the eastern end of Atbara Street and Seymour Street have not been included in the flood damage model. It was assumed that the houses affected will be removed. The cost of the house removal has been included as a capital cost in the economic analysis.

7.13 The channel widening option (in conjunction with the Moore River No. 4 Bifurcation Levee) significantly reduces flood damage costs. However, there is still a considerable ‘residual’ flood problem if this option is adopted in isolation. Other measures will be required to lower this residual flood risk.

7.3 ENVIRONMENTAL IMPACTS

7.14 Likely environmental impacts associated with the implementation of each of the six structural mitigation options were assessed. The assessment primarily focussed on the biological components most likely to be affected (i.e. flora and fauna). A full report on the environmental impact assessment is given in Appendix E. Only the findings are presented below.

7.3a Option 1 - Moore River No. 4 Bifurcation Levee

7.15 Moore River No. 4 Channel is an old flood channel of the Moore River which flows through Moora to the south of Moore River No. 1 channel. An old levee across Moore River No. 4 channel had previously been constructed to prevent overflows from Moore River No. 1 into this channel. During the March 1999 flood event this levee was both overtopped and outflanked and as a consequence was severely damaged. This option proposes that the levee be reconstructed at the same level as the original to prevent nuisance flooding in the Berkshire Valley Road area. Scour protection would be provided to prevent the levee from eroding if overtopped again.

7.16 Moore River No. 4 Channel is fringed by York Gum (*Eucalyptus loxophleba* ssp. *loxophleba*) dominated open woodland with scattered Wandoo (*Eucalyptus wandoo*). Given the existence of the old levee prior to the 1999 floods, there is unlikely to be any major impact on vegetation as a result of the construction of a new levee.

7.3b Option 2 - Detention Basin at Longpool

7.17 This option in the upper catchment involves the construction of a 4 m thick wall, 1370 metres in length (including a spillway length of 50 m, which is 7 m above the height of the stream), with a low flow culvert. At full supply capacity, the basin will have a stored volume of approximately 20,800 ML. The proposed basin is dominated by scattered Swamp Sheoak (*Casuarina obesa*), with occasional York Gum, Jam (*Acacia acuminata*), *Hakea preissii* and Paperbark (*Melaleuca rhaphiophylla*). There is little understorey remaining due to heavy grazing pressures on the area from livestock, although intermittent Bluebush (*Maireana* sp.) occurs on the fringes of the area. The highly saline drainage line is fringed by Swamp Sheoak.

7.18 The construction of a detention basin wall at the Longpool site will require the clearing of several Swamp Sheoak trees, and inundation, at peak flow, of an area of about 8.5 km^2. Associated with the loss of these trees would be the subsequent removal of avifauna nesting habitats as well as the loss other vertebrate or invertebrate fauna which use the trees as habitats. Inundation will affect some significant York Gum trees and other species intolerant to waterlogging. The drainage line in the vicinity of the basin had, as a result of the 1999 flood events, become severely eroded, and the roots of fringing Sheoaks were exposed and undermined to an extent where they may topple into the drainage line with further erosion. Additionally, the detention of water in the basin may result in the silting of an area upstream of the dam wall as well as possible erosion downstream.

7.19 During a flood event similar to the March 1999 event, it is predicted that water would remain within the basin for a period of up to 14 days. Positive short-term benefits of this retention include the creation of a temporary freshwater wetland and associated ecosystem that is likely to attract a variety of waterfowl etc to the area.

7.3c Option 3 - Detention Basin at Kitchin Bridge

7.20 This option in the upper catchment involves the construction of a 4 m thick wall, 1550 metres in length (including a spillway length of 50 m, which is 8 m above the height of the stream), with a low flow culvert. At full supply capacity, the basin will have a stored volume of approximately 16,200 ML. Vegetation in this location is predominantly of an open woodland of York Gum and Swamp Sheoak with scattered Jam, Wandoo, Flooded Gum and *Hakea preissii*, with a degraded, weed infested understorey. The drainage line (which is relatively saline) is fringed with *Halosarcia* sp. dominated samphire. A sizeable samphire flat occurs approximately a hundred metres to the northeast of the drainage line.

7.21 The construction of a detention basin at this site upstream of Kitchin Bridge will necessitate the clearing of several York Gum trees. Associated with the loss of these trees would be the subsequent loss of nesting habitats for birds and habitats for vertebrate or invertebrate fauna. Should a storm of similar intensity to the 1999 flood events occur, an area of about 5.8 km^2 of water will be detained in the basin. Detention of water in the basin will result in a period of inundation, with associated waterlogging, probably resulting in the deaths of all waterlogging intolerant trees to the east of the basin wall. Subsequently, there will be a deleterious impact on vertebrate and invertebrate fauna that utilise the vegetation in this area as habitat. Additionally, there will be some erosion of the drainage line resulting is the exposure of tree roots and probable subsequent deaths. Other than noise result generated during the construction of the wall, avifauna noted from the area (which included Port Lincoln Parrots, Magpies, Magpie Larks and Pink and Grey Galah's) are unlikely to be adversely impacted as a result of this option. The detention of water in the basin may result in the silting of an area upstream of the dam wall as well as possible erosion downstream.

7.22 During a flood event similar to the 1999 event, it is estimated that water would remain within the basin for a period of 10-14 days. Positive short-term benefits of this retention of water include the potential for creation of a temporary freshwater wetland and associated ecosystem that is likely to attract a variety of waterfowl etc to the area.

7.3d Option 4 – Northern Diversion Drain and Levee

7.23 This option involves the diversion of upstream floodwaters to the north of Moora into the Coonderoo Lakes System. The construction of a levee system has also been designed to reduce flood risk in Moora. The length of the proposed drain and levee is some 4,500 m and has been designed to cater for a flood of 10-15 year ARI. Several low flow culverts through the levee are also proposed (see Section 6.2d).

7.24 The diversion drain and levee construction will potentially result in the disturbance to and removal of a substantial stand of York Gum trees within the Carrick Street and Ralston Road road reserves. Associated with the loss of these trees would be the subsequent loss of avifauna nesting habitats and other vertebrate or invertebrate fauna which use them as habitats. Additionally vegetation occurring to the west and south of the proposed levee and drain system may suffer periods of inundation and potential waterlogging and death during a flood event, while increased flow velocity along the bottom reach of the Coonderoo River may result in bank erosion and subsequent loss of riparian vegetation and associated fauna habitat.

7.3e Option 5 – Southern Diversion Drain and Levee

7.25 The southern drain and diversion levee option involves the diversion of upstream floodwaters to the south to rejoin the Moore River downstream of Moora. It is a considerably more extensive drain and levee system than the northern diversion option (6,200 m in length).

7.26 Between the high school and Mogumber Road, the proposed diversion drain will be up to 30 metres wide and 3 metres in depth. Along with the removal of existing houses along Saleeba and Atbara Roads to allow this option to be constructed, a remnant of high quality Wandoo dominate low woodland with an understorey of *Allocasuarina campestris* heath will be negatively impacted upon. This area is potentially an important refuge for vertebrate fauna and will be detrimentally affected as a result of any clearing of vegetation. Additionally, this option is likely to result in the removal of good quality York Gum woodland in the Barber Street road reserve and several York and Salmon Gum trees where the drain intersects Cooper Road.

7.3f Option 6 – Widening of the Moore River No. 1 and No. 2 Channels

7.27 This option proposes the widening of Moore River No. 1 Channel by 20 m and Moore River No.2 channel by 10m commencing at Barber Road through to where both channels join (at Coonderoo River).

7.28 Moore River No. 1 Channel is fringed by a moderately diverse array of flora, particularly in the area adjoining the Causeway crossing on Gardiner Street. This area is vegetated with a tall open woodland dominated by York Gum and Flooded Gum scattered with the occasional Swamp Sheoak, with an understorey consisting of *Melaleuca viminea*, *Acacia saligna,* Jam, *Mesomelaena* sp., *Halosarcia* sp. The fringing vegetation further upstream is less diverse and more degraded as the River runs through land that has been cleared for agriculture, and consists of York and Flooded Gum along with numerous Flooded Gum seedlings. An area on the southern side of the channel near the channel crossing at Barber Road has been used as a refuse site for old car bodies, agricultural machinery and hydrocarbon drums. The vegetation fringing Moore River No. 2 channel is comprised predominantly of York Gum with occasional Wandoo and no native species in the understorey.

7.29 Widening Moore River No. 1 channel by 20 metres and the Moore River No. 2 channel by 10 metres from Barber St through to their junction with the Coonderoo River will result in the removal of the majority of the fringing riverine trees. Disturbance to the refuse/dump site near the Barber St crossing has the potential to release hydrocarbons and other contaminants into the river system.

7.3g Ranking Of Environmental Impacts

7.30 In order to determine the most favourable option, each of the six options were ranked on the basis of the likely extent of their environmental impact. The ranking (1 to 10) assigned to each option is on the basis of impacts on flora and vegetation; fauna and other factors (including increased salinity and erosion) are given in Table 7.5. A ranking of 10 implies the highest environmental impact and a ranking of 1 implies the lowest environmental impact. Note that the rankings assigned have taken into account any possible environmental impacts as well.

Table 7.5 Ranking of Environmental Impacts, Moora Structural Mitigation Options

Item Scored	Score/Rank					
	Option 1 (Moore No. 4 Levee)	Option 2 (Longpool Basin)	Option 3 (Kitchin Bridge Basin)	Option 4 (Northern Diversion)	Option 5 (Southern Diversion)	Option 6 (Channel Widening)
Impact on Flora and Vegetation	2	6	8	7	8	9
Impacts on Fauna	2	3	7	6	7	8
Other Impacts (including increased salinity, erosion	2	6	6	5	6	6

7.3h Summary Of Findings

7.31 Option 6 (Widening of Moore River No. 1 and No. 2 Channels) is likely to have the most significant detrimental environmental impact of any of the options, while Option 1 is likely to have the least potential for adverse impact on the environment.

7.32 There may be some short term positive impacts resulting from the implementation of either of Options 2 or 3, including the creation of a temporary freshwater wetland for waterbirds as a result of detention of water within the basins for approximately 10-14 days.

7.33 Findings of this study are considered sufficient only for the selection of the most favourable option. Once the most favourable option is determined, any further detailed studies that may be required relating to that option should be identified.

7.4 SOCIAL IMPACTS

7.4a Social Benefits

7.34 Social benefits from flood mitigation works are generally intangible but should still be considered in assessing the viable options. These intangible benefits include:

- Reduction in frequency of social disruption to the local community.
- Increase in value of properties in the area due to a reduction in flood problems.
- Increase in potential for development of the area.
- Reduction in health problems resulting from flood inundation.
- Reduction in stress and anxiety during severe rainfall events.
- Reduction in damage to public infrastructure such as roads.

7.35 For the Longpool detention basin option (Option 2), additional social benefits are likely to be achieved due to a reduction in flood damage to the Moora-Miling Road. This road provides the local community their only access to Moora during smaller floods. A reduction in the frequency of occurrence of larger floods will reduce the time the road is impassable. The Longpool detention basin will also reduce rural property damage along this reach.

7.4b Social Constraints

7.36 The social constraints associated with each of the flood mitigation options are outlined below.

(i) Option 1 - Moore River No. 4 Bifurcation Levee

7.37 There are no major social constraints associated with Option 1. There may be some disruption to the landholder during construction of the levee but this is likely to be only for a period of one or two days.

(ii) Option 2 - Detention Basin at Longpool

7.38 Up to 8.5 km^2 of land will be inundated upstream of the detention basin during a March 1999 type flood event. Although the frequency of inundation is low, some compensation for any potential loss of income may be necessary. Note that should the detention basin wall breach during an event similar to the March 1999 flood, a five to six metre wall of water could potentially flow into Moora. The consequences of this will be severe and catastrophic.

(iii) Option 3 - Detention Basin at Kitchin Bridge

7.39 Up to 5.8 km^2 of land will be inundated upstream of the Kitchin Bridge detention basin during a March 1999 type flood event. As per the Longpool detention basin, some compensation for any potential loss of income may be necessary. In addition, as per Longpool detention basin, consequences of a detention wall breach will be severe and catastrophic.

(iv) Option 4 - Northern Diversion Drain and Levee

7.40 The Northern diversion option will increase peak March 1999 flood levels at the nearest upstream property by about 0.2 m. The resultant property damage associated with this increase is not known.

7.41 There are properties along the western end of Riley, Clarke and Dandaragan Streets that were not flooded above floor level during the March 1999 flood event. Some of these properties would have flooded above floor level had this option been adopted. Compensation for these residents for the increase in flood risk may be necessary.

7.42 As for the detention basins, breaching of this levee at the peak of a flood event similar to the March 1999 flood will cause a wall of water one to two metres high flowing into Moora. The consequences of this will be severe and catastrophic.

(v) Option 5 - Southern Diversion Drain and Levee

7.43 Had the southern diversion drain and levee option been constructed prior to the March 1999 flood, peak flood levels at the nearest upstream property would have been increased by about 0.4 m. Further, some houses at the eastern end of Atbara and Seymour Streets will have to be removed should this option be adopted. Compensation for acquiring these houses will be necessary. Similar to the northern diversion option, breaching of this levee during a March 1999 type flood will be catastrophic.

(vi) Option 6 - Channel Widening

7.44 Some privately owned land along the Moore River No. 1 and No. 2 Branches may be affected by the channel widening option. Compensation for any loss of land may be necessary.

8 ECONOMIC EVALUATION OF STRUCTURAL MITIGATION OPTIONS

8.01 To assess the costs and benefits of the different structural flood mitigation options, economic analysis of the various options have been undertaken. Estimated costs, financial benefits and benefit/cost ratios for the different options are presented below.

8.1 ESTIMATED COSTS

8.02 The costs of structural flood mitigation options for Moora identified in Section 6 have been estimated using the Australian Construction Handbook (Rawlinsons, 2000). The costs are based on January 2000 unit rates for Perth, Western Australia. The estimated design and documentation costs and a 10% contingency allowance are included in the cost estimates. The total cost, as well as a breakdown of the estimated cost for major items of each option are given below.

8.1a Moore 4 Branch Bifurcation Levee

8.03 The estimated cost to construct the Moore 4 Bifurcation Levee is about $6,000. This estimate includes the cost of constructing the earth embankment from local materials, the placement of rip rap rock bank protection on the downstream face of the levee, and revegetation. It was assumed that there would be no land acquisition costs for this levee. The annual maintenance cost was assumed to be $500.

8.1b Yadgena Brook Levees

8.04 The estimated cost to construct the Yadgena Brook levees is about $2,000. This estimate covers the cost of constructing the earthern embankments from local materials. Note that land acquisition and maintenance costs associated with these levees have been assumed insignificant.

8.1c Longpool Detention Basin

8.05 The estimated cost to construct the Longpool detention basin is shown in Table 8.1. The costing assumes that there is sufficient clay material within 5 km of the site to construct the dam wall and sufficient rock material within the same distance to construct the spillway.

Table 8.1 Estimated Construction Cost for the Longpool Detention Basin

Item	Estimated Cost
Dam Wall	$ 2,200,000
Low Flow Pipes	$ 200,000
Spillway	$ 285,000
Land Acquisition Cost	$ 500,000
Contingencies @ 10%	$ 318,000
Total	$ 3,503,000

8.06 Note that $500,000 has been allocated to compensate the land owner for the siting of the detention basin on private property on the basis of the area expected to be inundated during a March 1999 type flood event. The operation and maintenance cost for the Longpool detention basin has been assumed at $5,000 per annum.

8.1d Kitchin Bridge Detention Basin

8.07 The estimated cost to construct the Kitchin Bridge detention basin is shown in Table 8.2. As per the Longpool detention basin option, it has been assumed that there is sufficient material available within 5 km to construct the dam wall and spillway.

Table 8.2 Estimated Construction Cost for the Kitchin Bridge Detention Basin

Item	Estimated Cost
Dam Wall	$ 2,965,000
Low Flow Pipes	$ 200,000
Spillway	$ 285,000
Land Acquisition Cost	$ 350,000
Contingencies @ 10%	$ 380,000
Total	$ 4,180,000

8.08 The Kitchin Bridge detention basin is slightly smaller than the Longpool detention basin. This is reflected by the smaller allocation of land acquisition/compensation costs for the basin when compared to the Longpool detention basin. The dam wall for this option is marginally higher and longer than for Longpool. Thus the total construction cost is slightly higher than the Longpool option. The operation and maintenance for this basin was assumed to be the same as for the Longpool basin ($5,000 per annum).

8.1e Northern Diversion Drain and Levee

8.09 Table 8.3 shows the estimated cost to construct the northern diversion drain and levee. The cost estimate for the levee earthworks assumes that material can be sourced at the site without additional cartage. The cost estimate to construct the railway culvert was provided by the Acting Project Manager, Westrail, Narngulu.

Table 8.3 Estimated Construction Costs for the Northern Drain and Levee

Item	Estimated Cost
Levee Earthworks (inc. Topsoil removal & fencing)	$ 565,000
Moore River No. 1 Pipe Culverts	$ 70,000
Moore River No. 2 Pipe Culverts	$ 25,000
Lower Midlands Highway	$ 70,000
Railway Culvert	$ 1,000,000
Land Acquisition Cost	$ 20,000
Contingencies @ 10%	$ 175,000
Total	$ 1,925,000

8.10 The operation and maintenance cost for this option has been assumed at $5,000 per annum.

8.1f Southern Diversion Drain and Levee

8.11 Table 8.4 shows the estimated cost to construct the southern diversion drain and levee. It has been assumed that the material for the levee can be sourced locally and that excess material from the diversion drain can be dumped within 1 km.

8.12 The southern diversion option is significantly more expensive than the northern diversion option. This is mainly due to the higher volume of material required to be excavated between the High School and Mogumber Road. A cost of $230,000 has also been allocated to purchase any properties that may be affected by the alignment of the proposed drain and levee.

Table 8.4 Estimated Construction Cost for the Southern Diversion Drain and Levee

Item	Estimated Cost
Levee Earthworks (inc. Topsoil removal & fencing)	$ 3,285,000
Moore River No. 1 Pipe Culverts	$ 70,000
Moore River No. 2 Pipe Culverts	$ 25,000
Bindoon Road and Rail Bridges	$ 1,150,000
Land & Property Acquisition Cost	$ 230,000
Road to Berkshire Valley	$ 240,000
Contingencies @ 10%	$ 500,000
Total	$ 5,500,000

8.1g Channel Widening

8.13 Table 8.5 shows the estimated cost to widen the Moore River No. 1 and No. 2 Channels. It was assumed that these channels would be widened only between Barber Street and the Moore River No. 1 and No. 2 confluence. A small levee (0.3 m high) will be required to divert any overbank floodwater back into these two channels at Barber Street.

Table 8.5 Estimated Construction Cost for the Channel Widening

Item	Estimated Cost
Site Clearing/Sediment Control	$ 60,000
Bulk Excavation	$ 1,360,000
Revegetation	$ 70,000
Contingencies	$ 159,000
Total	$ 1,640,000

8.14 It has been assumed that there would be no land acquisition costs involved for this option and no upgrade of the bridges and culverts. The operation and maintenance cost for this option was assumed at $5,000 per annum.

8.1h Summary of Structural Mitigation Option Costs

8.15 Table 8.6 shows a summary of the costs to construct the six structural mitigation options identified in this study. Note that the costs presented here should be considered preliminary and indicative only at this stage. Detailed investigations are necessary to obtain more accurate cost estimates.

Table 8.6 Summary of Estimated Capital Costs for Structural Mitigation Options, Moora

Option	Description	Estimated Cost
1	Moore River No. 4 Bifurcation Levee	$ 6,000
2	Longpool Detention Basin	$ 3,503,000
3	Kitchin Bridge Detention Basin	$ 4,180,000
4	Northern Diversion and Levee	$ 1,925,000
5	Southern Diversion and Levee	$ 5,500,000
6	Channel Widening	$ 1,640,000

8.16 Note that Yadgena Brook levees (see Section 8.1b) were assumed to be included in all of the above six options. In addition, for all options, except the diversion drain and levee options (options 4 and 5), the Moore River No. 4 Bifurcation Levee was assumed to be included.

8.2 FINANCIAL BENEFITS

8.17 The financial benefits for the identified flood mitigation options include:

- Reduced property damages for a particular flood event.
- Reduced average annual damages.

8.18 The estimated property damages for the defined flood event (March 1999 flood) for the different flood mitigation options investigated are provided in Table 8.7.

Table 8.7 Estimated March 1999 Flood Property Damages for Different Structural Flood Mitigation Options

Option	Estimated 100 Year ARI Flood Damage ($1,000)
Existing Conditions	6,576
Option 1 - Moore River No. 4 Bifurcation Levee	6,817
Option 2 - Longpool Detention Basin	93
Option 3 - Kitchin Bridge Detention Basin	147
Option 4 - Northern Diversion & Levee	459
Option 5 - Southern Diversion & Levee	0
Option 6 - Channel Widening	4,780

8.19 The estimated average annual actual flood property damages (AAAFD) for the different flood mitigation options are given in Table 8.8

Table 8.8 Estimated AAAFD for Different Structural Flood Mitigation Options

Option	Estimated AAAFD
Existing Conditions	$ 105,400
Option 1 - Moore River No. 4 Bifurcation Levee	$ 102,500
Option 2 - Longpool Detention Basin	$ 45,900
Option 3 - Kitchin Bridge Detention Basin	$ 46,400
Option 4 - Northern Diversion & Levee	$ 46,800
Option 5 - Southern Diversion & Levee	$ 42,400
Option 6 - Channel Widening	$ 74,000

8.3 BENEFIT/COST RATIOS

8.20 The benefit/cost ratios (BCR) for the different structural flood mitigation options investigated are shown in Table 8.9. A design life of 50 years for the works has been assumed and discount rates of 4%, 6% and 8% were applied to determine benefit/cost ratios.

Table 8.9 Benefit/Cost Ratios for Different Structural Flood Mitigation Options

Option	Adopted Discount Rate		
	4%	6%	8%
Option 1 - Moore River No. 4 Bifurcation Levee	6.44	4.73	3.67
Option 2 - Longpool Detention Basin	0.33	0.25	0.19
Option 3 - Kitchin Bridge Detention Basin	0.28	0.20	0.16
Option 4 - Northern Diversion & Levee	0.60	0.44	0.34
Option 5 - Southern Diversion & Levee	0.23	0.17	0.13
Option 6 - Channel Widening	0.34	0.25	0.20

8.21 The analyses of costs and benefits show that the Moore River No. 4 Bifurcation Levee (Option 1) yields a very high BCR, although the reduction in AAAFD compared to the existing situation is not significant. All other options have BCR's significantly less than 1, with the possible exception of the Northern Diversion and Levee Option (Option 4). For a 4% discount rate the BCR for Option 4 is 0.60.

8.22 It is noted that the above analysis has not taken into account the flood damage to public infrastructure (roads, railway, footpaths, etc.) and the potential reduction in damage to public infrastructure due to the different flood mitigation options. Flood damage to rural properties (farms) has also not been taken into account. Furthermore, the potential social costs and benefits arising from the different flood mitigation options have also not been included. Incorporation of the above factors is likely to make some of the above BCR's more attractive.

9 CONCLUSIONS

9.01 Two numerical models were developed and calibrated to simulate flooding behaviour in the Moore River catchment:

- A runoff - routing model (URBS) was used to estimate flood discharges throughout the Moore River catchment.

- An unsteady flow hydraulic model (Mike 11) was used to estimate flood levels in the Moora township area.

9.02 Hydrologic and hydraulic models were first calibrated against four recent flood events that occurred in March, May, July and August 1999. The calibrated models were then used to estimate design flood discharges and flood levels at Moora for 2, 5, 10, 20, 50 and 100 year ARI flood events and for the probable maximum flood event.

9.03 March and May 1999 floods at Moora were extreme events. Given the uncertainties in the data available for design flood estimation at Moora, it is not possible to assign ARI's to March and May 1999 flood events with any confidence. However, based on a subjective assessment of available information, it is estimated that the March and May 1999 events would have had ARI's of 100 - 250 years and 50 - 100 years respectively.

9.04 It is estimated that some 158 properties will be flooded above floor level for the 100 year ARI flood event. A further 290 properties will be flooded below floor level. The total actual flood damage to urban properties associated with this event is some $1.53 million. The estimated average annual actual flood damage for properties in Moora under existing conditions is $105,400.

9.05 A range of structural flood mitigation options for Moora was investigated. The structural measures investigated in detail include:

- Detention Basins
- Levees
- River Diversions, and
- Channel Widening.

9.06 It was found that the Moore River No. 4 Bifurcation Levee Option (Option 1) and the Northern Diversion and Levee Option (Option 4) were the most viable structural options for reducing flood damages in Moora.

- Moore River No. 4 Bifurcation Levee by itself yields a very high benefit/cost ratio (greater than 3) although the reductions in the number of properties flooded and the flood damage are relatively small.

- The Northern Diversion and Levee Option yields a benefit/cost ratio of between 0.34 and 0.6 (depending on the adopted discount rate), but the reductions in the number of floodprone properties and associated flood damages are significant. It is noted that the benefit/cost ratio estimates of this study are considered to be under estimated because the analyses have not taken into account the potential reduction in damage to public infrastructure and the potential social costs/benefits arising from the different flood mitigation options.

9.07 A range of non-structural options was also investigated. The non-structural measures investigated in detail included:

- Land Use Zoning Controls,
- Building and Development Controls,
- Floodproofing of Buildings, including House Raising,
- Public Awareness Education,
- Flood Forecasting, Warning and Evacuation, and
- Voluntary Purchase.

Merits of each of these options have been assessed and strategies to implement each option have been recommended.

9.08 In terms of environmental impacts, the Moore River No. 4 Bifurcation Levee Option (Option 1) will have the least potential for adverse impact on the environment. The Moore River No. 1 and No. 2 Channel widening option (Option 6) is likely to have the most significant adverse environmental impact of all the options considered. The Northern Diversion and Levee Option (Option 4) is likely to have only a moderate impact on the environment. Note that, if any of the identified options are considered for implementation, an appropriate level of detailed investigation should be undertaken prior to works being approved.

9.09 The social impacts of all of the flood mitigation options are generally positive. The benefits include a reduction in the frequency of social disruption to the local community, a reduction in potential health problems, an increase in the value of properties in Moora and an increase in the potential for additional development to occur. There will also be a significant reduction in stress and anxiety associated with flooding.

10 REFERENCES

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A Appendix HYDROLOGIC MODEL DEVELOPMENT AND CALIBRATION

A1 INTRODUCTION

A.01 This Appendix, which describes the development and calibration of the hydrologic (URBS) model, is structured as follows:

- Section A2 describes the selection of calibration events.
- Section A3 describes the development and calibration of the hydrologic model.
- Section A4 presents conclusions on model development and calibration.
- Section A5 is a list of references.

A.02 In addition, this Appendix contains seven Addenda:

- Addendum A lists the adopted pluviograph station and event rainfall data used for each sub-catchment in the URBS model for the calibration events.
- Addendum B lists URBS model parameters.
- Addendum C presents graphs of recorded and predicted discharges for the March 1999 event.
- Addendum D presents graphs of recorded and predicted discharges for the May 1999 event.
- Addendum E presents graphs of recorded and predicted discharges for the July 1999 event.
- Addendum F presents graphs of recorded and predicted discharges for the August 1999 event.
- Addendum G presents rating curves estimated from the URBS model output for the Nardy Road and Roundhill Bridge gauging stations.

A2 SELECTION OF CALIBRATION EVENTS

A.03 Data available for this flood study is described in Section 3 of the Main Report. It is recalled that there is a good coverage of daily rainfall stations in the Moore River catchment. However, the coverage of pluviograph stations and stream gauging stations prior to July 1999 was poor:

- Prior to July 1999, five pluviograph stations were within the area of interest. However, only Berkshire Valley (508 001), was located within the catchment with available data for the March and May events (see Section 3.5 and Table 3.2).
- Prior to June 1999 only two stream gauging stations were located in the catchment: Quinns Ford (617 001) and Woury Pool (617 009). Quinn's Ford gauging station is located 70 km downstream of Moora. Woury Pool stream gauging station is located on Moore River East and was installed in May 1999.

A.04 No significant flood events have been recorded in the Moore River Catchment, upstream of Moora, prior to 1999. Two significant events (March and May) and two small events (July and August) were recorded in 1999. Due to the limited quantity of available data, all four of these events were used for model calibration. Table A2.1 shows the details of the adopted calibration events.

Table A2.1 Summary of Adopted Calibration Events

Event	Calibration Period				
	Start		Finish		Duration (Days)
March 1999	0900	17/03/1999	0900	31/03/1999	14
May 1999	0900	24/05/1999	0900	08/06/1999	15
July 1999	0900	03/07/1999	0900	25/07/1999	22
August 1999	0900	15/08/1999	0900	23/08/1999	8

A.05 For the hydrologic model calibration, recorded streamflow data for the March and May 1999 events was supplemented with flood levels recorded in the vicinity of Moora Township (see Table 3.5). These flood levels provided information on the timing and shape of the flood hydrograph at Moora. Estimates of peak flood discharge at Moora for the March and May 1999 events were made on the basis of a joint calibration of hydrologic and hydraulic models.

A.06 Figures A2.1 to A2.4 show recorded streamflows at Quinn's Ford (617001) along with recorded rainfalls at Berkshire Valley (508001) for each calibration event. Figures A2.3 and A2.4 also show the recorded streamflows at Moora Caravan Park (617010) for the July and August 1999 events respectively.

A.07 The July and August 1999 events provide some indication of the runoff response in the Moore River North catchment upstream of the Moora township. Note that both events are quite small and display distinct double peaks. These events were used primarily to assess the different rainfall - runoff behaviour (and loss characteristics) of the various geological and soil groups within the catchment. The March and May 1999 events were used to calibrate the hydrologic and hydraulic models to reproduce overall catchment response during large rainfall events.

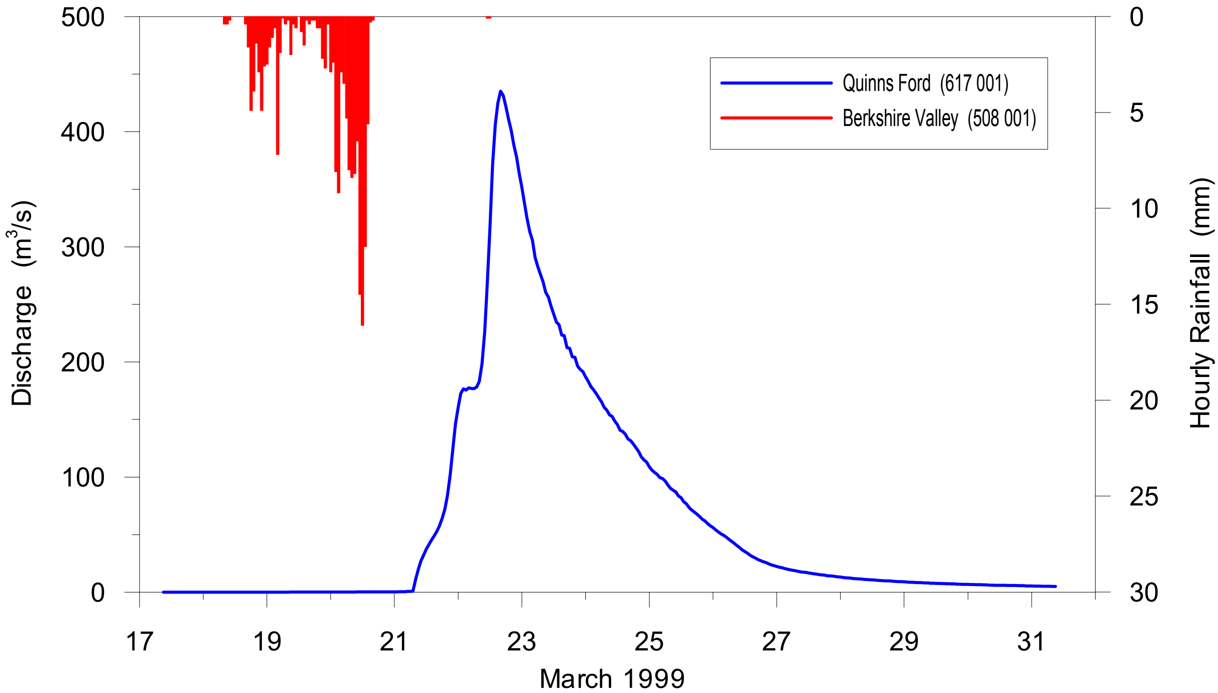

Figure A2.1 Representative Rainfall and Discharge Hydrograph, March 1999 Event

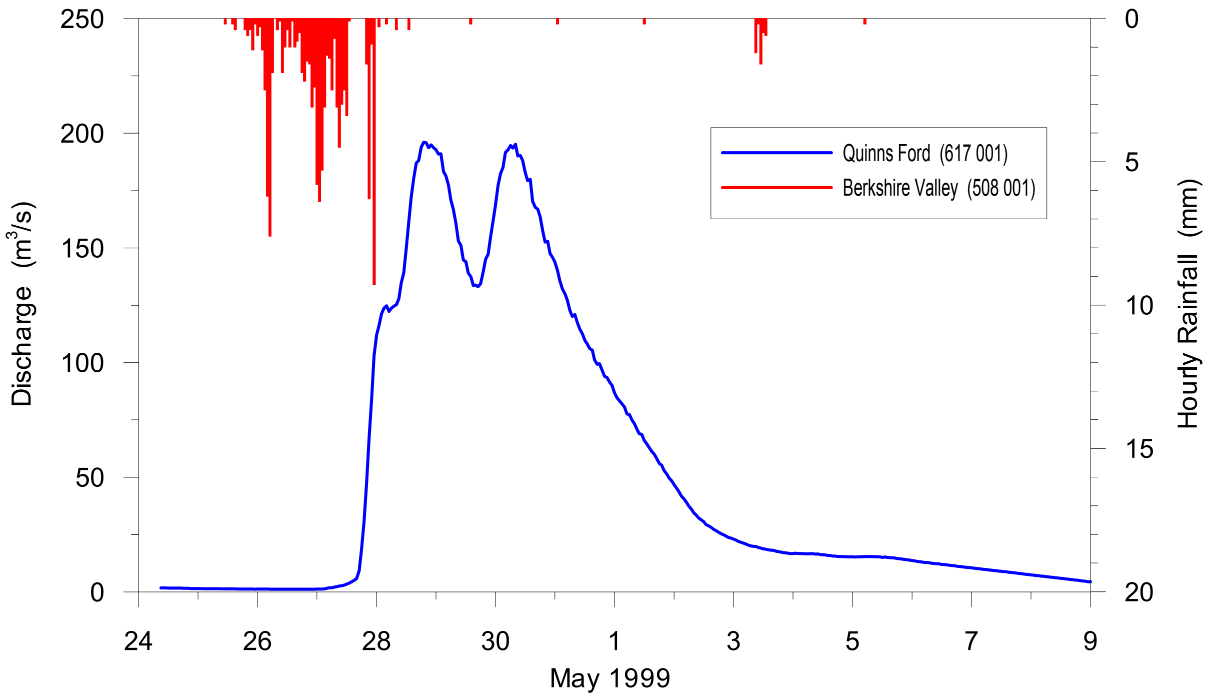

Figure A2.2 Representative Rainfall and Discharge Hydrograph, May 1999 Event

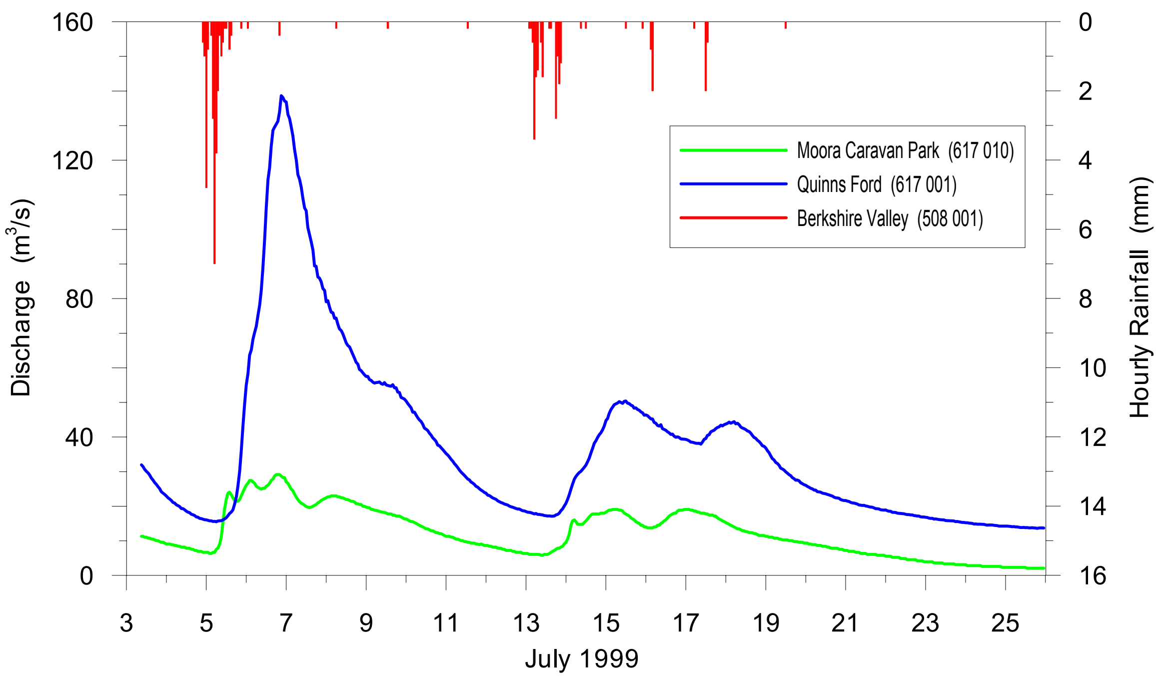

Figure A2.3 Representative Rainfall and Discharge Hydrographs, July 1999 Event

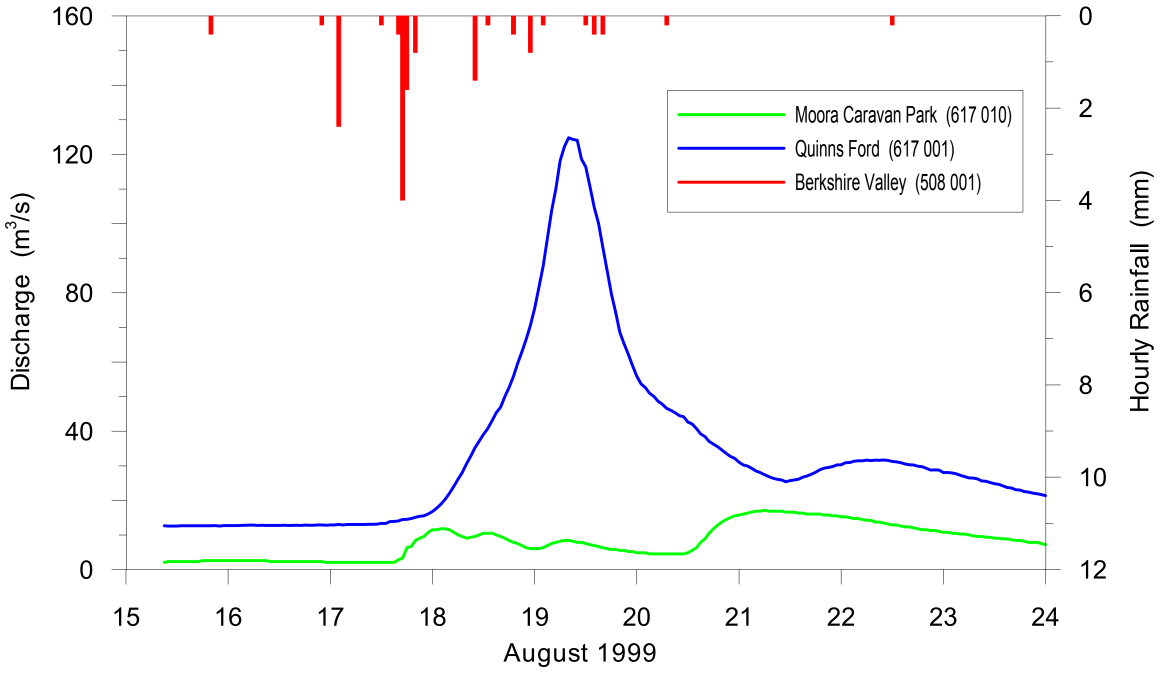

Figure A2.4 Representative Rainfall and Discharge Hydrographs, August 1999 Event

A3 HYDROLOGIC MODELLING

A3.1 Model Description

A.08 The Urban Runoff and Basin Systems (URBS) model (Carroll, 1998) is a sub-catchment based runoff-routing model used to estimate flood discharges throughout a catchment. The model provides a number of options for conceptualising the rainfall-runoff process. For the Moore River catchment, the 'Split' model (in which sub-catchment runoff and channel flows are routed separately), was adopted.

A.09 In the Split model the rainfall excess for each sub-catchment is first determined by subtracting losses from the rainfall hyetograph. For the Moore River catchment the 'continuing loss' model was adopted. This model assumes that there is an initial loss of **'il'** mm before any rainfall becomes effective. After this, a continuing loss rate of **'cl'** mm per hour is applied to the rainfall, subject to the limit of the soil infiltration capacity.

A.10 Soil infiltration capacity is assumed to follow a simple linear relationship with volume infiltrated. This is done using the equation:

$$f_{eff} = f_u + \frac{F_t}{F_{max}} \qquad (f_{eff} \leq 1) \qquad \text{(A.1)}$$

where f_{eff} is effective impervious area (%),
f_u is existing impervious area (%),
F_t is cumulative infiltration into the impervious area after time 't' (mm),
F_{max} is the maximum infiltration capacity (mm).

A.11 Recovery of infiltration capacity (drying of the soil profile) is modelled by reducing the infiltrated volume after each time step:

$$F_{t+\Delta t} = k\ F_t \tag{A.2}$$

where <t is the model time step increment,
k is the infiltration capacity recovery factor.

A.12 For the Moore River catchment model, F_{max} and k are global parameters.

A.13 The rainfall excess is then routed through a conceptual catchment storage to determine the local runoff hydrograph for the sub-catchment. The storage - discharge relationship for catchment routing is:

$$S_{catch} = \left\{ \frac{\beta \sqrt{A}\,(1 + F)^2}{(1 + U)^2} \right\} Q^m \tag{A.3}$$

where S_{catch} is the catchment storage (m^3 h/s),
β is the catchment lag parameter,
A is the area of sub-catchment (km^2),
U is the fraction urbanisation of sub-catchment,
F is the fraction of sub-catchment forested, and
m is the catchment non-linearity parameter.

Note that in Equation A.3, β is determined during model calibration, and is a global parameter.

A.14 The local runoff hydrograph is then combined with runoff from the upstream catchment and routed through a channel storage to obtain the outflow hydrograph at the outlet of the sub-catchment. The channel routing storage - discharge relationship is given by:

$$S_{chnl} = \alpha f \frac{n L}{\sqrt{S_c}} \left(x Q_u + (1 - x) Q_d \right)^{n^*} \tag{A.4}$$

Channel routing is based on non-linear Muskingum Model and is given as:

where S_{chnl} is the channel storage (m^3 h/s),
α is the channel lag parameter,
f is the reach length factor,
L is the length of reach (km),
S_c is the channel slope (m/m),
Q_u is the inflow at upstream end of reach (includes catchment inflow) (m^3/s),
Q_d is the outflow at downstream end of the channel reach (m^3/s),
x is the Muskingum translation parameter,
n* is the Muskingum non-linearity parameter (exponent), and
n is the Manning's 'n' or channel roughness factor.

Note that in Equation A.4, α and f are the principal calibration parameters. Note also that α is a global parameter, whereas f can be varied for each channel reach.

A.15 URBS also allows for variation in the Manning's 'n' value for 'in-channel' (n_c) and 'overbank' (n_o) flow. The channel capacity (Q_c) defines the upper limit of 'in channel' flow. Mathematically this is written:

$$n = n_c \qquad (Q \leq Q_c) \tag{A.5a}$$

$$n = n_o \qquad (Q > Q_c) \tag{A.5b}$$

A.16 Note that while the 'channel capacity' and Manning's 'n' values for 'in-channel' and 'overbank' flow parameters have a conceptual meaning, in terms of model calibration these parameters merely provide a means of modifying hydrograph shape. For the Moore River catchment model n_c, n_o and Q_c vary for each channel reach.

A.17 Note also that equation A.4 shows that the reach length factor (f) and the Manning's channel roughness factor (n) are inter-dependent. That is, if one parameter is increased and the other decreased by the same proportion, model output will be unaffected. Hence, for practical purposes, the following conventions were adopted:

- n_c = 1 for all sub-catchments.
- Channel roughness was modelled using the reach length factor (f).
- The difference between channel roughness for in-channel and overbank flow was modelled with the n_o parameter.

A.18 Full details of the URBS model are given in the URBS User Manual (Carroll, 1998).

A3.2 Model Configuration

A.19 The configuration of the Moore River URBS model is shown in Figure A3.1. The model covers the entire catchment upstream of Quinn's Ford and consists of 46 sub-catchments, ranging from 13 km^2 to 1,510 km^2. Details of the sub-catchment areas are given in Table A3.1. The variation in sub-catchment areas reflects the resolution of both the available data and the desired model output. For example, sub-catchment areas in the northern portion of the catchment are large since there is limited available streamflow or pluviograph data. Further, observed flooding behaviour indicates that this portion of the catchment has little impact on the southern portion of the catchment. Conversely, the Moore River North catchment is divided into much smaller sub-catchments since there is more streamflow and pluviograph data and accurate modelling of the catchment is critical for assessment of flooding behaviour in the Moora township.

Table A3.1 Moore River URBS Sub-Catchment Areas

Sub-Catchment	Area (km^2)	Sub-Catchment	Area (km^2)	Sub-Catchment	Area (km^2)
1	896	21	24	41	298
2	876	22	36	42	242
3	1,510	23	69	43	85
4	769	24	71	44	36
5	1,498	25	55	45	196
6	618	26	59	46	128
7	177	27	17		
8	208	28	13		
9	51	29	60		
10	99	30	30		
11	338	31	29		
12	163	32	122		
13	154	33	153		
14	215	34	51		
15	140	35	169		
16	192	36	159		
17	82	37	212		
18	50	38	277		
19	41	39	310		
20	37	40	142		

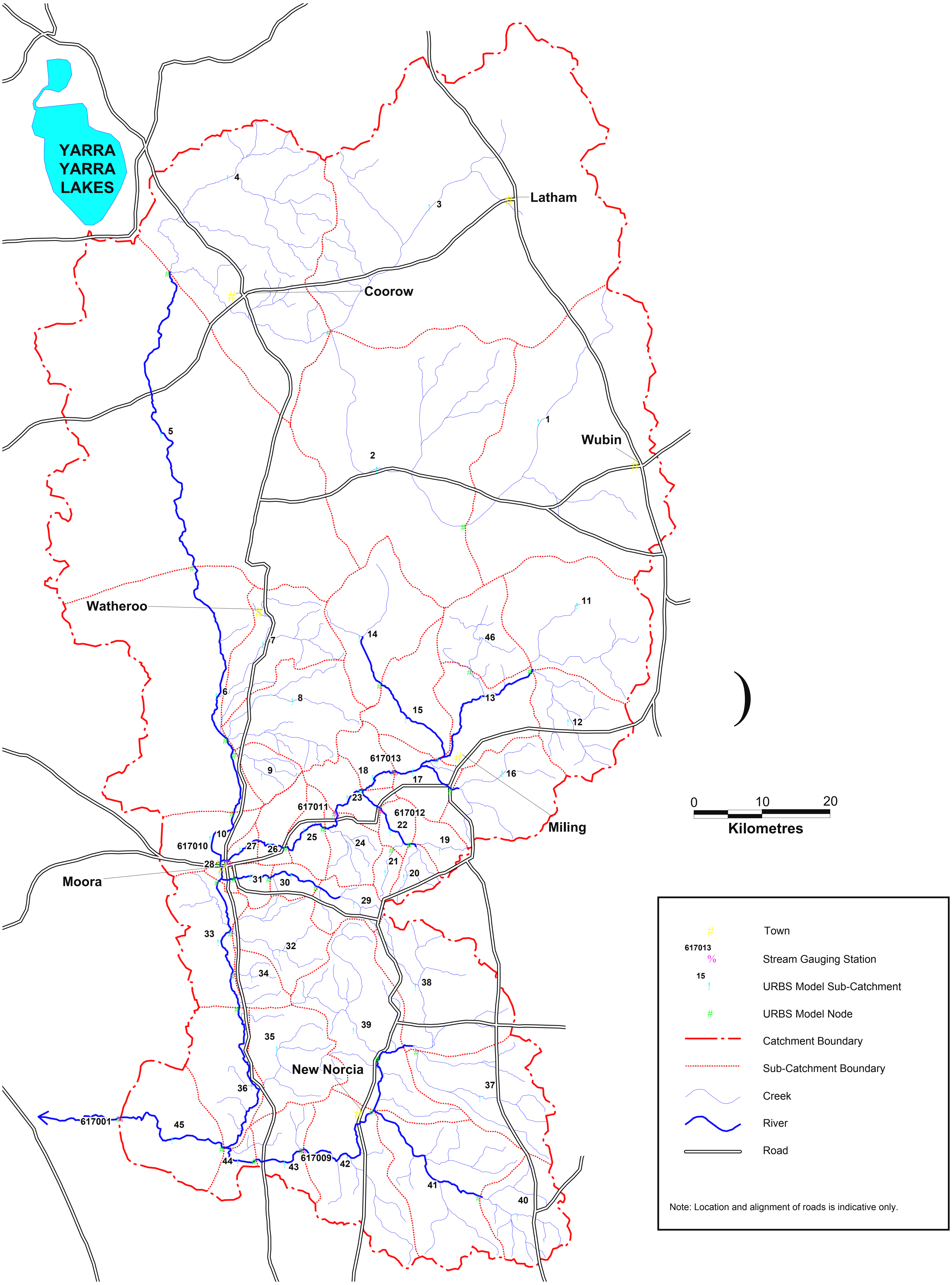

Figure A3.1 Moore River Catchment, URBS Model Configuration

A3.3 Model Calibration

(i) Calibration Methodology

A.20 The URBS model was calibrated to achieve the best possible fit between recorded and predicted discharge hydrographs at the various gauging stations within the catchment for the following events:

- March 1999,
- May 1999,
- July 1999, and
- August 1999.

A.21 The underlying philosophy applied to the hydrologic model calibration was to use the minimum number of parameters necessary to obtain reasonable model fits for the four events. The geographic diversity of the Moore River catchment precluded the use of a single set of model parameters covering the entire catchment. Hence, the catchment was divided into five regions of similar hydrologic behaviour. These are shown in Figure A3.2.

- Region 1 is the Coonderoo River catchment upstream of the confluence with Moore River North.

- Region 2 contains the headwaters of the Moore River North catchment, known locally as the 'Miling' area. Region 2 contains most of the area upstream of the Nardy Road gauging station (617 013).

- Region 3 is the remainder of the Moore River North catchment upstream of the Moora Caravan Park gauging station (617 010). This region includes the Dungaroo Creek catchment upstream of the Roundhill Bridge gauging station (617 012), which is known locally as the 'Bindi Bindi' catchment.

- Region 4 consists of the sub-catchments containing the main Moore River North and Moore River channel between the Moora Caravan Park and Quinn's Ford gauging stations (617 010 and 617 001 respectively).

- Region 5 contains the Yagdena Brook catchment, the Moore River East catchment and tributaries of Moore River North between Moora and Quinn's Ford.

A.22 Note that the aggregation of sub-catchments into Regions 4 and 5 reflects the different topography, geology and soil characteristics of the two regions, controlled by the presence of the Darling Fault. Sub-catchments in Region 4 have highly permeable soils and a flat topography, therefore producing very little runoff. In contrast, sub-catchments in Region 5 are generally steeper and less permeable than those in Region 4, and consequently generate more runoff.

A.23 Calibration of the URBS model was achieved by:

- Adjusting the various global parameters (such as α and β),
- Adjusting the initial and continuing rainfall losses for the five model regions
- Adjusting the reach length factor (f) for various channel reaches, and

to achieve the best fit between recorded and predicted discharge hydrographs

A.24 The model fit was assessed through both visual inspection of the recorded and predicted discharge hydrographs and analysis of the magnitude and timing of the recorded and predicted peak discharge. Attempts were made to use a non-linear parameter estimation model (PEST, Watermark Computing, 1998) during calibration. However, this technique proved unsuccessful due to insufficient streamflow data.

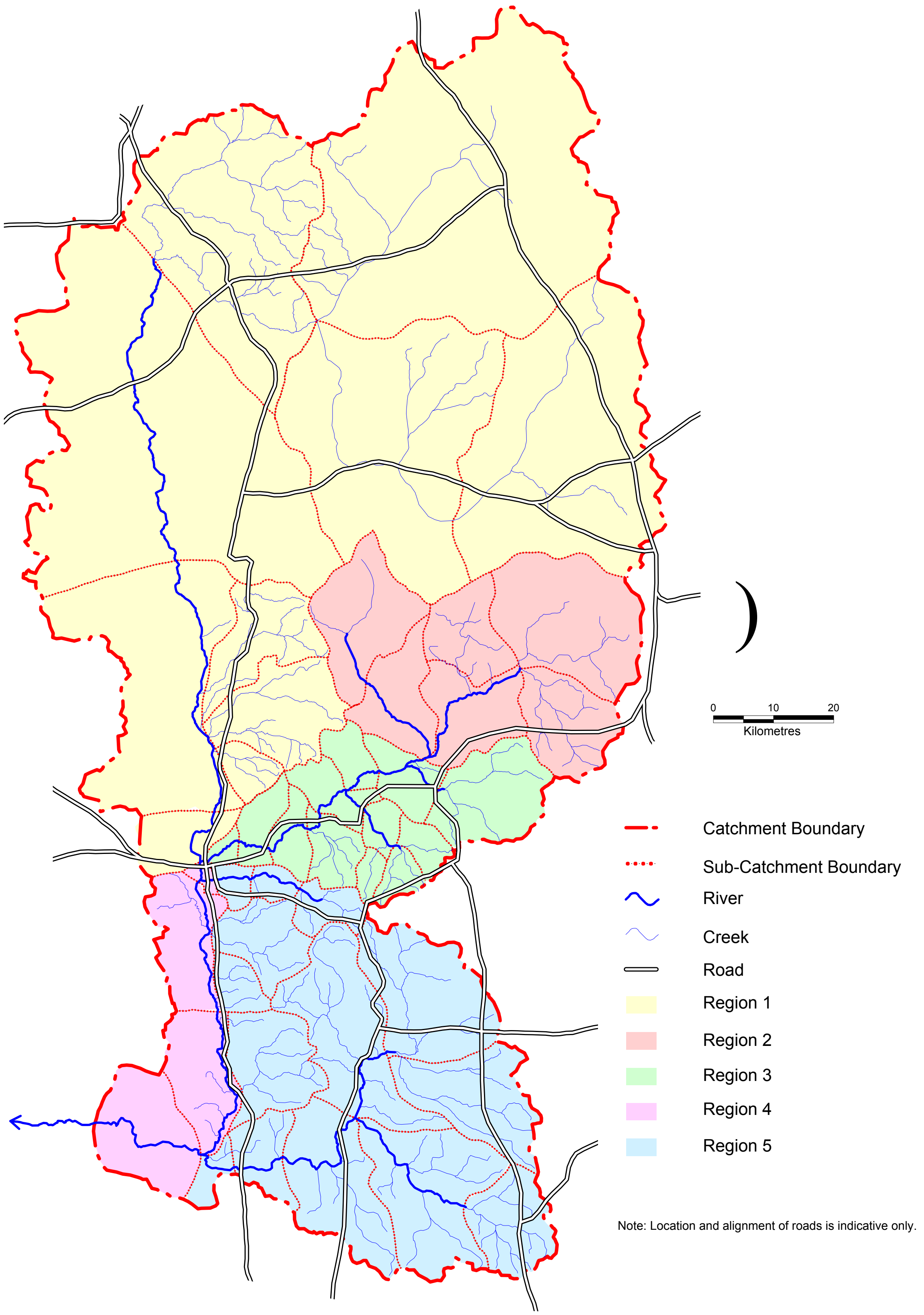

Figure A3.2 Catchment Regions, Moore River URBS Model

A.25 Initially, attempts were made to derive a single set of model parameters for each region for all events (with the exception of initial loss, which varied between events). However, it was found that no set of parameters adequately described all events. Hence, different sets of initial and continuing losses were used for each region for each event (although continuing losses were kept 'reasonably' similar between events).

A.26 It was also found that while the hydrological behaviour of sub-catchments within a region were generally similar, some sub-catchments within a region displayed atypical behaviour. This results in some variation between parameters within regions. Consider, for example, sub-catchment No. 27 which encompasses Moora township. Widening and clearing, resulting in a hydraulically efficient channel, have extensively modified the stream through the town. This is reflected in a reach length factor (f) of 0.5 for sub-catchment No. 27, compared to a value of f = 0.8 to 0.9 for the remainder of Region 3 (see Table B1, Addendum B). Further, while in-channel flow may be hydraulically efficient, overbank flow is relatively inefficient due to obstruction by urban development on the floodplain. Hence the value of Manning's 'n' for overbank flow (n_o) for sub-catchment No. 27 is 1.1, compared with n_o = 0.6 for the remainder of Region 3 (see Table B4, Addendum B).

A.27 To avoid difficulties associated with estimating baseflow for design events, the URBS model parameters were calibrated to match the full event hydrographs, including baseflow.

(ii) Hydraulic Model

A.28 A hydraulic model of the Moora township was developed as a component of the Moora Flood Management study. Section 3 of the main report showed that there was a paucity of discharge data for the catchment at locations other than Quinns Ford. Further, while there was some flood level information in Moora for the March and May events, there was no corresponding discharge data. Hence, calibration of both the hydrologic and hydraulic models was an iterative process. Discharge estimates from the hydrologic model were used as input to the hydraulic model, with flood level predictions from the hydraulic model then checked against recorded flood levels. This proved to be an effective method of estimating discharges in Moora for ungauged events, and checking the accuracy of recorded stream discharges.

(iii) Assignment of Total Rainfalls and Temporal Patterns

A.29 The program URBSRAIN, developed by the Bureau of Meteorology, was used to calculate the depth of rainfall of each sub-catchment of the model and also to determine the appropriate pluviograph to be applied to each sub-catchment rainfall.

A.30 The total event rainfall for each sub-catchment is determined by a distance-weighted average of the four nearest daily rainfall stations. The weighting for each station is inversely proportional to the square of the distance from the daily rainfall station to the centroid of the sub-catchment. The temporal pattern of the nearest pluviograph rainfall station is assigned to each sub-catchment in the model. The adopted pluviograph station and event rainfall for URBS model sub-catchments for each calibration event are provided in Addendum A.

A.31 For the May event only, the Dalwallinu Town Rainfall Station (008 039) temporal pattern was assigned to URBS sub-catchments Nos. 11, 12, 13, 16 and 46. Based on anecdotal information from WRC, a six hour time lag was applied to rainfall at these five sites (i.e. rainfall was adjusted to occur six hours later than recorded at Dalwalanu).

A.32 For the August event, the Moora West pluviograph (008 038) temporal pattern was assigned to URBS model sub-catchments 29, 30 and 31. Based on anecdotal information from WRC, a 10 minute time lag was applied to rainfall at these three sites (i.e. rainfall was adjusted to occur 10 minutes later than recorded at Moora West).

(iv) Adopted Model Parameters

A.33 Based on the results of the model calibration, Table A3.2 lists adopted 'global' model parameters that were applied to all regions for all calibration events.

Table A3.2 URBS Global Model Parameters

Parameter	Value
Catchment Routing Parameter (β)	2.0
Channel Routing Parameter (α)	1.0
Catchment Routing Exponent (M)	0.8
Fraction Urbanised Parameter (U)	0
Fraction Forested (F)	0
Muskingum Translation Parameter (x)	0.45
Muskingum Exponent (n*)	1
Infiltration Capacity (F_{max}) (mm)	150
Infiltration Capacity Recovery Factor (k)	0.9

A.34 Note that the urbanization and forestation parameters, U and F respectively, are only applicable when investigating the effect of changes in catchment land use on streamflow. For the Moore River catchment model, these parameters were set to zero ('0').

A.35 A summary of the adopted 'regional' parameters for the five model regions is given in Table A3.3. Note that these parameters varied slightly for some sub-catchments within a region.

Table A3.3 URBS Regional Model Parameters [a]

	Region 1	Region 2	Region 3	Region 4	Region 5
Reach length factor (f)	2.3	2.3	0.8	0.6	0.8
Roughness Factor (in-channel flow) [b] (n_c)	1	1	1	1	1
Roughness Factor (overbank flow) [b] (n_o)	1	0.6	0.6	1.1	1
Channel Capacity [c] (Q_c) (m^3/s)	20-100	30-40	20-110	30-100	90-150
Continuing Loss (cl) (mm/hr)					
- March	3	3	1.2	2.1	2.1
- May	3	0	0	2.5 - 4.0	3
- July	3	3.1	3.1	2.3	3
- August	3	1	0	1.7	1.7
Initial Loss (il) (mm)					
- March	190	160	100	105	190
- May	120	80	30	10	120
- July	70	2	12	0.5	70
- August	50	15	16	16	50

a. Some parameters varied slightly for individual sub-catchments within a region. See Addendum B for full parameter list.
b. See Section A3.1 for a full explanation of the model significance of the Manning's 'n' parameters.
c. Parameter range given. Actual capacity depended on catchment area upstream of each sub-catchment. See Addendum B for full parameter list.

A.36 Transmission loss was extracted from Moore River North streamflows between Moora and the confluence with Moore River East, to account for overbank flows that does not return to the main channel. For all calibration events, discharges at Node 36 that were greater than 150 m^3/s were reduced by 20%. A full list of URBS model parameters is provided in Addendum B.

(v) Calibration Results

Overview

A.37 Plots of recorded and predicted discharge hydrographs at the various gauging stations and for Yadgena Brook at Walebing Road, for the four calibration events, are shown in Addenda C to F.

A.38 In general, the fit between recorded and predicted hydrographs is reasonable. The calibration process was hampered by limited streamflow data for the catchment for locations other than Quinns Ford. For the calibration events of March and May 1999, discharge data was only available at Quinn's Ford (with limited stream height data available at other locations). This meant that a good model fit could be obtained at Quinn's Ford with any number of different discharge scenarios upstream. For example, it could be assumed that all streamflow at Quinn's Ford is derived from the Moore River East catchment with no contribution from the Moore River North catchment. Alternatively, discharge at Quinn's Ford could be generated entirely in the Moore River North catchment with no contribution from Moore River East. Both of these scenarios would produce a good model fit at Quinn's Ford, but would give inaccurate results for elsewhere in the catchment.

A.39 The model calibration relied heavily on the July and March events. The July event data provided a good representation of the spatial distribution of streamflow within the catchment, since five new gauging stations were installed in May/June 1999 (see Table 3.1). The March event provided data for calibration of the model against high streamflows.

A.40 The results of the hydraulic model indicated that the Coonderoo River had no significant impact on streamflows during any of the four calibration events. Hence, in the absence of any other information on Coonderoo catchment discharges, loss parameters in the Coonderoo catchment were adjusted so that the streamflow generated from the catchment during any of the calibration events was insignificant.

A.41 Note that the 'recorded' discharge at Woury Pool gauging station for the May, July and August events is based on a preliminary rating curve. Thus, during calibration, the recorded discharge at Woury Pool was used to determine the timing of the peak discharge and provide an indication only of the magnitude of the peak discharge.

March 1999 Event

A.42 A good fit was obtained for the March 1999 event at Quinn's Ford. The hydraulic model estimated the peak discharge at Moora Caravan Park to be about 500 m^3/s at 7.00 am on 21 March 1999. URBS model parameters were adjusted to match the timing and magnitude of the flood peak at Moora Caravan Park.

May 1999 Event

A.43 Calibration results for the May 1999 event are poor. Note that there is almost no streamflow information for this event in Moore River North upstream of Moora.

A.44 Based on an iterative joint calibration of the hydrologic and hydraulic models the estimated peak discharge at Moora for the May event was 285 m^3/s. Note that the March event in Moora peaked at 500 m^3/s and the difference in flood levels between the March and May events was only about 300 mm. Hence, the estimated peak discharge of 285 m^3/s at Moora for the May event appears reasonable. However, to achieve a discharge of this magnitude at Moora using the hydrologic model whilst matching the recorded times of flood peaks at Moora Caravan Park, Long Pool and Roundhill, it was necessary to 'sacrifice' the model calibration at Quinn's Ford. The estimated discharge at Quinn's Ford was 380 m^3/s compared to a recorded value of about 200 m^3/s.

A.45 Based on the estimated discharge hydrograph for Woury Pool, it is possible that the stream gauging station at Quinn's Ford may have malfunctioned during the May event. This is because the peak discharge for Woury Pool in May is 161 m^3/s and the peak discharge for the first recorded peak at Quinn's Ford (which would have come from Moore River East) is only 194 m^3/s. Given the super-position effect of the flows from Moore River North and Moore River East, the peak discharge for the first recorded peak at Quinn's Ford would be expected to be much higher than 194 m^3/s.

A.46 The model prediction of the discharge hydrograph at Woury Pool is good. Note that the difference between recorded and predicted peak discharge was not improved upon because:

- The rating curve for Woury Pool is preliminary only,
- There is a good fit between the general shape of the recorded and predicted hydrographs, and
- Peak discharge at Quinn's Ford is greatly over-predicted and an increase in discharge from Moore River East would only exacerbate the problem.

A.47 The estimated peak discharge at Quinn's Ford is significantly greater than the peak recorded discharge. If URBS model parameters were adjusted so that the model produced the recorded peak discharge of 200 m^3/s at Quinn's Ford, then peak discharge at Moora Caravan Park would be under-predicted by some 170 m^3/s. Given that the emphasis of this study is on the Moora township, it was decided to parameterise the model to provide 'good' discharge estimates at Moora, and 'sacrifice' the discharge estimates at Quinns Ford. It is likely that there are either:

- Unidentified physical processes occurring in the catchment that are not adequately represented in the model, or
- Errors in the recorded data.

A.48 Given the limited data available for the May event, it was not possible to estimate what these data errors or physical processes were. Hence, the model calibration could not be improved upon.

<u>July 1999 Event</u>

A.49 The July event provided the best data set for model calibration. Hydraulic model discharge estimates showed that while the gauged flow at the Moora Caravan Park was accurate for the stream branch in which it is located; there was flow occurring simultaneously in another branch that was not recorded. Hence, hydrologic model estimates for discharge at Moora Caravan Park, which do not account for stream branching, had to be greater than the recorded flow. Figure 4 in Addendum E shows good model representation of peak timing and an over-estimation of recorded flows, accurately reflecting the total catchment discharge.

A.50 The model also over-predicts peak discharges at Nardy and Roundhill. The difference between recorded and predicted discharge at Nardy is much greater than the extra flow requirement at Moora Caravan Park discussed previously. Given that the Nardy and Roundhill catchments are nested within the catchments of Long Pool and Moora Caravan Park gauging stations, it is possible that the difference between the recorded and predicted peak discharges at Nardy and Roundhill are due to errors in the derived rating curves at these sites. This is supported by:

- The maximum gauged discharge being lower than the recorded July discharges at both sites (e.g. at Roundhill, the maximum gauged discharge is only 1 m^3/s, but the estimated peak discharge in July is 27 m^3/s; see Table 2.4).
- The observation that if the predicted and recorded peak discharges are matched at both Nardy and Roundhill, discharges at Long Pool and Moora Caravan Park are poorly predicted.

A.51 It is suspected that the error at Nardy is due to the braided nature of the stream at that location. This allows streamflows to bypass the gauging station, resulting in the gauging station under-estimating total discharge. The error at the Roundhill gauging station is most likely due to extrapolation of the rating curve. The predicted timing of the flood peak at Woury Pool on Moore River East, together with the predicted hydrograph shape at Quinn's Ford are both excellent.

August 1999 Event

A.52 The August event was caused by a localised, high intensity storm. This is in contrast to the previous three calibration events, which were caused by widespread low pressure systems. The advantage of modelling rainfall - runoff from widespread rainfall is that rainfall totals and temporal patterns can be interpolated spatially between rainfall stations with a degree of confidence. For an intense local storm, rainfall totals and patterns can vary greatly within short distances and interpolation between rainfall stations can be highly inaccurate (especially if stations are scattered widely). Table 1 (Addendum A) shows high spatial variability between pluviograph stations, with the Moora West Station in particular showing recorded event rainfall totals some three times greater than nearby stations.

A.53 Hydraulic modelling of the August event in Yadgena Brook estimated the peak discharge at Walebing Road to be about 132 m^3/s. Model calibration revealed that it was not physically possible to obtain a peak discharge of this magnitude with the recorded rainfall. It is most likely that intense rainfall in the lower portion of the Yadgena Brook catchment was not recorded at any of the nearby rainfall stations. Rainfall estimates for the Yadgena Brook sub-catchments No. 29, 30 and 31 were multiplied by a factor of 2.1 to generate a peak model discharge estimate in Yadgena Brook of 132 m^3/s. Table A3.4 shows the original and adopted rainfall estimates for the Yadgena Brook catchment. Table A3.5 shows the ARI for sub-catchment No. 31 rainfall for a range of durations.

Table A3.4 Original and Adopted Rainfall Estimates, Yadgena Brook, August 1999

Sub-Catchment	Rainfall (mm)	
	Original Estimate	Adopted Estimate
29	18	38
30	23	48
31	52	109

Table A3.5 ARI of Adopted Rainfall, URBS Sub-Catchment No. 31, Yadgena Brook August 1999

Rainfall Duration (Hours)	Adopted Rainfall Intensity (mm/hr)	ARI (Years)
2	24.6	71
6	15.3	>100
9	10.7	>100
12	8.2	>100
18	5.6	95
24	4.2	64
30	3.5	58
36	2.9	44
48	2.2	35
72	1.5	28

A.54 The model results in Addendum F show that the fit between recorded and predicted discharges at the four gauging stations in the Moore River North catchment are fair. The two principal sources of error are:

- The poor spatial representation of event rainfall, and

- The small size of the event. The discharges are very low and are probably near the resolution of the model.

A.55 Note that while discharges are over-predicted at Long Pool, they are under-predicted just downstream at Moora Caravan Park. Further, the shape of the recorded hydrograph at Moora Caravan Park shows peaks which are not present at Long Pool and are not reflected in the predicted hydrograph. This indicates localised rainfall that has not been adequately represented by the raingauge network.

A.56 The timing of the predicted flood peak at Woury Pool is good, but the hydrograph shape is too 'broad'. Considering the good agreement between recorded and predicted discharge at Woury Pool for the May and July events, the poor fit for the August event is most likely due to recorded rainfall distribution errors.

A.57 The predicted hydrograph at Quinn's Ford is reasonable. A compromise was made in the calibration of flood hydrographs at Woury Pool and Quinn's Ford to obtain a reasonable model prediction at both sites. Initial losses for sub-catchments No. 32, 34 and 35 were set at 50 mm, which resulted in no runoff being produced from these sub-catchments. Runoff from these sub-catchments normally enters the Moore River North between Moora and the confluence with Moore River East (see Figure A3.1).

A.58 It is clear that any extra inflow to this section of the river would only serve to increase flows at Quinn's Ford, further degrading the model discharge estimate. It is most likely that very little rain fell in URBS sub-catchments 32, 34 and 35 and the high initial loss rates used in these sub-catchments merely compensates for this.

Timing of Discharge Peaks

A.59 The calibrated URBS model provides a good representation of the contribution of different sub-catchments to flood peaks in Moora. The source of the different peaks that occurred during the July event at Long Pool and Moora Caravan Park are marked in Figures 3 and 4 respectively of Addendum E. The 'local' contributing areas are sub-catchments 24 to 27 (see Figure A3.1). Note that there is a large degree of superposition between discharge hydrographs from different sub-catchments. Note also that the effect of the different contributing areas is diminished as the flood peak moves downstream (July discharge at Quinn's Ford; Figure 6, Addendum E) and during large events (March discharge at Moora; Figure 4, Addendum C).

A.60 Figure 4, Addendum E shows that the highest peak discharge at Moora is due to runoff from the Roundhill catchment combined with runoff from the local catchment between Long Pool and Moora. Discharge from the catchment upstream of Nardy Road increases the peak discharge at Moora through superposition with discharges from both Roundhill and local Moora catchments, but does not in itself produce the highest peak discharge during an event.

A3.4 Stream Gauge Rating Curves

A.61 In Section A3.3 it was shown that the rating curves for the Nardy Road (617 013) and Roundhill (617 012) gauging stations are thought to be in error. Revised rating curves were derived from the URBS model results for the July 1999 event. Figures A3.3 and A3.4 show the recorded and predicted discharges for the July 1999 event using the revised ratings for Nardy and Roundhill gauging stations respectively. Addendum G tabulates and graphs the stream rating curves, as estimated by the URBS model.

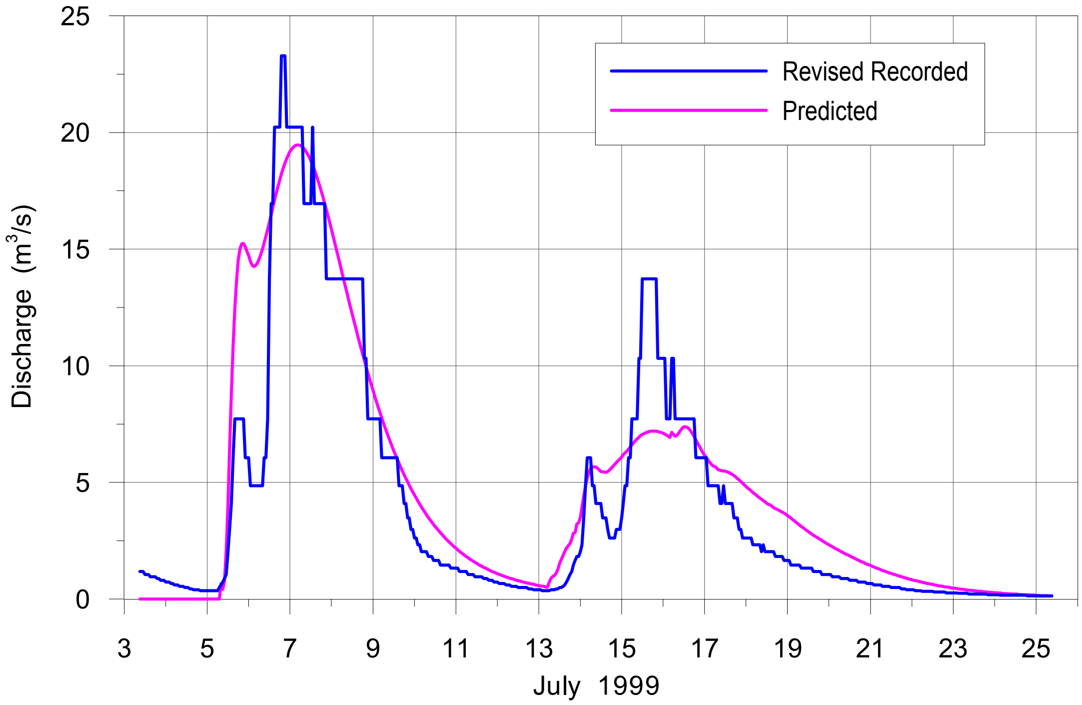

Figure A3.3 Recorded and Predicted Discharge Hydrographs, URBS Model, Moore River North - Nardy Road (617013), July 1999 Event, Using the Revised Rating Curve

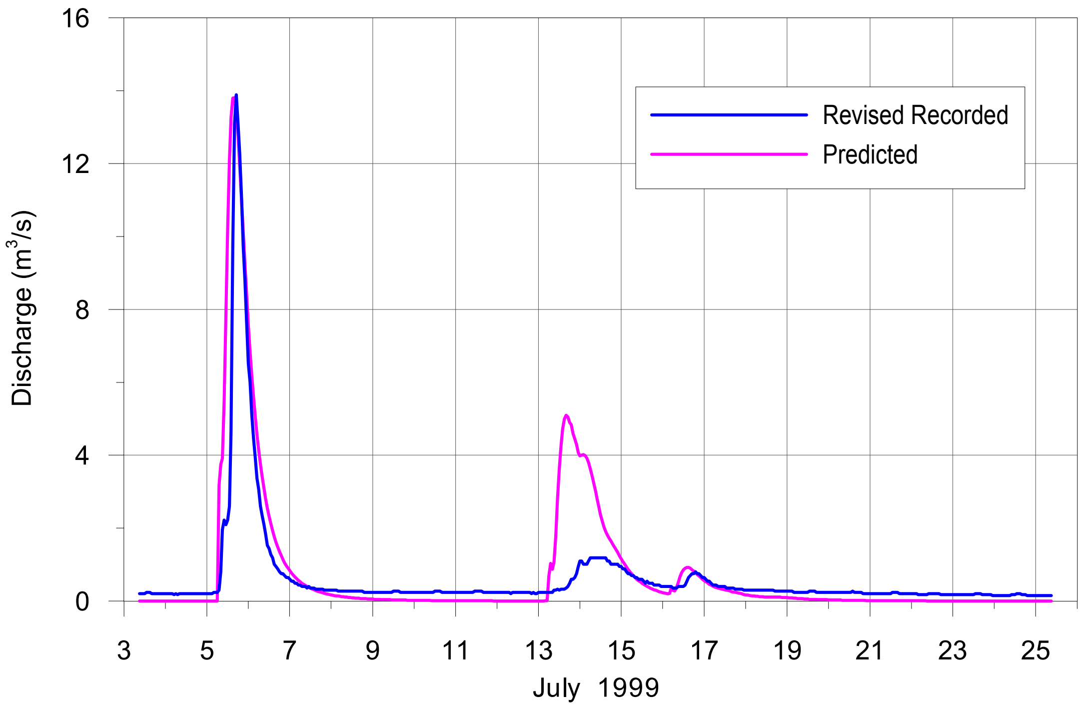

Figure A3.4 Recorded and Predicted Discharge Hydrographs, URBS Model, Dungaroo Creek, Upstream Roundhill Bridge (617012), July 1999 Event, Using the Revised Rating Curve

A4 CONCLUSIONS

A.62 An URBS runoff-routing hydrologic model was developed for the Moore River catchment to Quinn's Ford gauging station (617 001). The model was calibrated against four rainfall events:

- March 1999
- May 1999
- July 1999
- August 1999

A.63 Calibration of the hydrologic model was undertaken simultaneously with the calibration of a hydraulic model of Moora township, developed as a component of the Moora Flood Management Study. This joint calibration of the hydrologic and hydraulic models provided a means of estimating discharges in Moora for ungauged events, but also aided in checking the accuracy of recorded stream discharges.

A.64 Overall, the model calibration was reasonable. Sub-catchments were grouped into regions, with essentially a common set of parameters used for all sub-catchments in a region. Each set of regional parameters was used for all events, with the exception of the initial and continuing loss parameters, which were varied between events.

A.65 Calibration of the model was hampered by:

- A limited number of rainfall events.
- Lack of discharge data at the sub-catchment scale for the two major events of March and May 1999.
- Poor spatial representation of rainfall for the calibration events.

A.66 Good model fits were obtained for the March and July events. These were key calibration events for the model since:

- The March event represented the highest ever recorded discharge at Quinn's Ford gauging station of 435 m^3/s.
- The July event provided detailed discharge data at five locations within the catchment.

A.67 Calibration of the May 1999 event was poor because a good model fit at Quinn's Ford could not be obtained simultaneously with the estimated peak discharge at Moora Caravan Park. The poor fit is most likely due to data errors or unidentified physical processes that are not adequately represented in the model. The lack of detailed gauging data for the Moore River North catchment during the May event prevented the definition of these data errors or physical processes.

A.68 Calibration of the August event was reasonable. Errors in the calibration were due to:

- The localised nature of the rainfall, which was not adequately defined by the pluviograph and raingauge network, and
- The event being very small.

A.69 Model predictions for the July event indicated that the rating curves at Nardy and Roundhill Stations were suspect. These rating curves were revised based on the results of the URBS model output.

A5 REFERENCES

Carroll, D.G. (1998).	'URBS-CM, A Catchment Management and Flood Forecasting Rainfall Runoff Model', Users Manual, 1998.
Watermark Computing (1998)	'PEST, Model Independent Parameter Estimation', Users Manual, 1998.

ADDENDUM A

Adopted Pluviograph Stations and Event Rainfalls for URBS Model Sub-Catchments, Calibration Events

Table 1 Adopted Pluviograph Station and Event Rainfall for URBS Model Sub-Catchments, Calibration Events

Sub-Catchment No.	March 1999 Event			May 1999 Event			July 1999 Event			August 1999 Event		
	Station No.	Station Name	Event Rainfall	Station No.	Station Name	Event Rainfall	Station No.	Station Name	Event Rainfall	Station No.	Station Name	Event Rainfall
1	508001	Berkshire Valley	123	508001	Berkshire Valley	124	008160	Meridale	38	008160	Meridale	16
2	508001	Berkshire Valley	117	508001	Berkshire Valley	119	008160	Meridale	50	008160	Meridale	19
3	508001	Berkshire Valley	103	508001	Berkshire Valley	127	008160	Meridale	47	008160	Meridale	12
4	508001	Berkshire Valley	102	508001	Berkshire Valley	131	008174	Noondine	50	008174	Noondine	18
5	508001	Berkshire Valley	119	508001	Berkshire Valley	123	008174	Noondine	48	008174	Noondine	19
6	508001	Berkshire Valley	164	508001	Berkshire Valley	115	008174	Noondine	56	008174	Noondine	24
7	508001	Berkshire Valley	186	508001	Berkshire Valley	103	008174	Noondine	48	008174	Noondine	22
8	508001	Berkshire Valley	181	508001	Berkshire Valley	105	008174	Noondine	56	008174	Noondine	16
9	508001	Berkshire Valley	160	508001	Berkshire Valley	108	008174	Noondine	58	008174	Noondine	15
10	508001	Berkshire Valley	138	508001	Berkshire Valley	116	008038	Moora West	57	008038	Moora West	55
11	508001	Berkshire Valley	169	008039	Dalwalanu + 6 Hrs	95	008160	Meridale	34	008160	Meridale	17
12	508001	Berkshire Valley	179	008039	Dalwalanu + 6 Hrs	97	008160	Meridale	43	008160	Meridale	18
13	508001	Berkshire Valley	180	008039	Dalwalanu + 6 Hrs	100	008160	Meridale	44	008160	Meridale	21
14	508001	Berkshire Valley	187	508001	Berkshire Valley	102	008160	Meridale	44	008160	Meridale	23
15	508001	Berkshire Valley	200	508001	Berkshire Valley	101	008160	Meridale	45	008160	Meridale	23
16	508001	Berkshire Valley	187	008039	Dalwalanu + 6 Hrs	103	008160	Meridale	46	008160	Meridale	17
17	508001	Berkshire Valley	188	508001	Berkshire Valley	108	008160	Meridale	48	008160	Meridale	17
18	508001	Berkshire Valley	177	508001	Berkshire Valley	108	008115	Round Hill	56	008115	Round Hill	13
19	508001	Berkshire Valley	186	508001	Berkshire Valley	106	008009	Bindi Bindi	54	008009	Bindi Bindi	22
20	508001	Berkshire Valley	175	508001	Berkshire Valley	102	008115	Round Hill	59	008115	Round Hill	18
21	508001	Berkshire Valley	167	508001	Berkshire Valley	102	008115	Round Hill	54	008115	Round Hill	16
22	508001	Berkshire Valley	177	508001	Berkshire Valley	107	008115	Round Hill	56	008115	Round Hill	13
23	508001	Berkshire Valley	166	508001	Berkshire Valley	108	508001	Berkshire Valley	56	508001	Berkshire Valley	16
24	508001	Berkshire Valley	162	508001	Berkshire Valley	103	008115	Round Hill	49	008115	Round Hill	14
25	508001	Berkshire Valley	166	508001	Berkshire Valley	106	508001	Berkshire Valley	54	508001	Berkshire Valley	17
26	508001	Berkshire Valley	162	508001	Berkshire Valley	107	008174	Noondine	54	008174	Noondine	17
27	508001	Berkshire Valley	139	508001	Berkshire Valley	115	008038	Moora West	57	008038	Moora West	53
28	508001	Berkshire Valley	137	508001	Berkshire Valley	118	008038	Moora West	56	008038	Moora West	56
29	508001	Berkshire Valley	155	508001	Berkshire Valley	97	508001	Berkshire Valley	38	508001	Moora West + 10 Min	38
30	508001	Berkshire Valley	146	508001	Berkshire Valley	98	508001	Berkshire Valley	36	508001	Moora West + 10 Min	48
31	508001	Berkshire Valley	133	508001	Berkshire Valley	116	008038	Moora West	56	008038	Moora West + 10 Min	109
32	508001	Berkshire Valley	133	508001	Berkshire Valley	113	008038	Moora West	53	008038	Moora West	46
33	508001	Berkshire Valley	125	508001	Berkshire Valley	119	008038	Moora West	59	008038	Moora West	49
34	508001	Berkshire Valley	128	508001	Berkshire Valley	117	008038	Moora West	57	008038	Moora West	46
35	508001	Berkshire Valley	105	508001	Berkshire Valley	118	008038	Moora West	67	008038	Moora West	45
36	508001	Berkshire Valley	95	508001	Berkshire Valley	124	008038	Moora West	80	008038	Moora West	49
37	008137	Wongan Hills PO	110	008137	Wongan Hills PO	81	008009	Bindi Bindi	57	008009	Bindi Bindi	27
38	508001	Berkshire Valley	140	508001	Berkshire Valley	101	008009	Bindi Bindi	53	008009	Bindi Bindi	36
39	508001	Berkshire Valley	138	508001	Berkshire Valley	95	008009	Bindi Bindi	52	008009	Bindi Bindi	32
40	008137	Wongan Hills PO	106	008137	Wongan Hills PO	76	008137	Wongan Hills PO	61	008137	Wongan Hills PO	30
41	008137	Wongan Hills PO	105	008137	Wongan Hills PO	81	008137	Wongan Hills PO	64	008137	Wongan Hills PO	34
42	508001	Berkshire Valley	91	508001	Berkshire Valley	99	008009	Bindi Bindi	71	008009	Bindi Bindi	30
43	508001	Berkshire Valley	82	508001	Berkshire Valley	119	008038	Moora West	79	008038	Moora West	49
44	508001	Berkshire Valley	84	508001	Berkshire Valley	123	008038	Moora West	83	008038	Moora West	49
45	508001	Berkshire Valley	90	508001	Berkshire Valley	125	008038	Moora West	86	008038	Moora West	49
46	508001	Berkshire Valley	180	008039	Dalwalanu + 6 Hrs	100	008160	Meridale	44	008160	Meridale	21

ADDENDUM B

URBS Model Parameters

Table 1 URBS Model Sub-Catchment Reach Length Factors

Region	Sub-Catchment No	Reach Length Factor (f)
1	1	2.3
1	2	2.3
1	3	2.3
1	4	2.3
1	5	2.3
1	6	3.3
1	10	3.3
1	7	0.8
1	8	0.8
1	9	0.8
2	11	2.3
2	12	2.3
2	46	2.3
2	13	2.3
2	14	2.3
2	15	2.3
3	16	0.8
3	17	0.8
3	19	0.8
3	20	0.8
3	21	0.8
3	22	0.8
3	18	0.8
3	23	0.8
3	24	0.9
3	25	0.9
3	26	0.9
3	27	0.5
4	29	0.3
4	30	0.3
4	31	0.3
4	32	0.6
4	34	1.8
4	35	1.8
4	37	0.6
4	38	0.6
4	40	0.6
4	39	0.6
4	41	0.6
4	42	0.6
4	43	0.6
4	44	0.3
5	28	0.9
5	33	0.8
5	36	0.8
5	45	0.3

Table 2 URBS Model Sub-Catchment Continuing Losses

Region	Sub-Catchment No	Continuing Loss (mm/hr)			
		March	May	July	August
1	1	3	3	3	3
1	2	3	3	3	3
1	3	3	3	3	3
1	4	3	3	3	3
1	5	3	3	3	3
1	6	3	3	3	3
1	7	3	3	3	3
1	8	3	3	3	3
1	9	3	3	3	3
1	10	3	3	3	3
2	11	3	0	3.1	1
2	12	3	0	3.1	1
2	46	3	0	3.1	1
2	13	3	0	3.1	1
2	14	3	0	3.1	1
2	15	3	0	3.1	1
3	16	3	0	3.1	0
3	17	1.2	0	3.1	0
3	18	1.2	0	3.1	0
3	19	1.2	0	3.1	0
3	20	1.2	0	3.1	0
3	21	1.2	0	3.1	0
3	22	1.2	0	3.1	0
3	23	1.2	0	3.1	0
3	24	1.2	0	1.5	0
3	25	1.2	0	1.8	0
3	26	1.2	0	1.8	0
3	27	1.2	0	1.8	0
4	29	2.1	2.5	2.3	1.7
4	30	2.1	2.5	2.3	1.7
4	31	2.1	2.5	2.3	1.7
4	32	2.1	2.5	2.3	1.7
4	34	2.1	2.5	2.3	1.7
4	35	2.1	2.5	2.3	1.7
4	37	2.1	4	2.4	2.7
4	38	2.1	4	2.4	2.7
4	40	2.1	4	2.4	2.7
4	39	2.1	4	2.4	2.7
4	41	2.1	4	2.3	2.7
4	42	2.1	4	2.3	2.7
4	43	2.1	4	2.3	2.7
4	44	2.1	4	3	1.7
5	28	2.1	3	3	1.7
5	33	2.1	3	3	1.7
5	36	2.1	3	3	1.7
5	45	2.1	3	3	1.7

Table 3 URBS Model Initial Losses

Region	Sub-Catchment No	Initial Loss (mm)			
		March	May	July	August
1	1	190	120	70	50
1	2	190	120	70	50
1	3	190	120	70	50
1	4	190	120	70	50
1	5	190	120	70	50
1	6	190	120	70	50
1	10	190	120	70	50
1	7	190	120	70	50
1	8	190	120	70	50
1	9	190	120	70	50
2	11	160	60	2	15
2	12	160	60	2	15
2	46	160	60	2	15
2	13	160	60	2	15
2	14	160	60	2	15
2	15	160	60	2	14
3	16	160	60	2	14
3	17	100	60	12	14
3	18	100	60	12	16
3	19	100	20	12	16
3	20	100	20	12	16
3	21	100	20	12	16
3	22	100	20	12	16
3	23	100	90	12	16
3	24	110	90	0	20
3	25	110	90	0	20
3	26	110	90	0	25
3	27	110	90	0	25
4	29	105	10	0.5	12
4	30	105	10	0.5	12
4	31	105	10	0.5	12
4	32	105	10	0.5	50
4	34	105	10	0.5	50
4	35	105	10	0.5	50
4	37	105	90	0.5	12
4	38	105	90	0.5	12
4	40	105	90	0.5	12
4	39	105	10	0.5	12
4	41	105	10	0.5	12
4	42	105	10	0.5	12
4	43	105	10	0.5	45
4	44	190	120	70	50
5	28	190	120	70	50
5	33	190	120	70	50
5	36	190	120	70	50
5	45	190	120	70	50

Table 4 URBS Model Sub-Catchment Channel Capacities and Manning's 'n' Factor for In-Channel and Overbank Flow

Region	Sub-Catchment No	Manning's 'n' Channel flow	Manning's 'n' Overbank flow	Channel Capacity (m^3/s)
1	1	1	1	30
1	2	1	1	30
1	3	1	1	30
1	4	1	1	30
1	5	1	1	30
1	6	1	1	30
1	10	1	1	100
1	7	1	1	20
1	8	1	1	20
1	9	1	1	20
2	11	1	0.6	30
2	12	1	0.6	30
2	46	1	0.6	30
2	13	1	0.6	30
2	14	1	0.6	30
2	15	1	0.6	40
3	16	1	0.6	40
3	17	1	0.6	60
3	19	1	0.6	20
3	20	1	0.6	20
3	21	1	0.6	20
3	22	1	0.6	30
3	18	1	0.6	60
3	23	1	0.6	110
3	24	1	0.6	110
3	25	1	0.6	110
3	26	1	0.6	110
3	27	1	1.1	110
4	29	1	1	30
4	30	1	1	30
4	31	1	1	30
4	32	1	1	30
4	34	1	1	40
4	35	1	1	40
4	37	1	1.1	30
4	38	1	1.1	30
4	40	1	1.1	30
4	39	1	1.1	90
4	41	1	1.1	60
4	42	1	1.1	90
4	43	1	1.1	90
4	44	1	1.1	100
5	28	1	1	110
5	33	1	0.8	90
5	36	1	0.8	90
5	45	1	1	150

Note. See Section 4.1 for a full explanation of the model significance of the Manning's `n` parameters

ADDENDUM C

Recorded and Predicted Discharges, URBS Model March 1999 Event

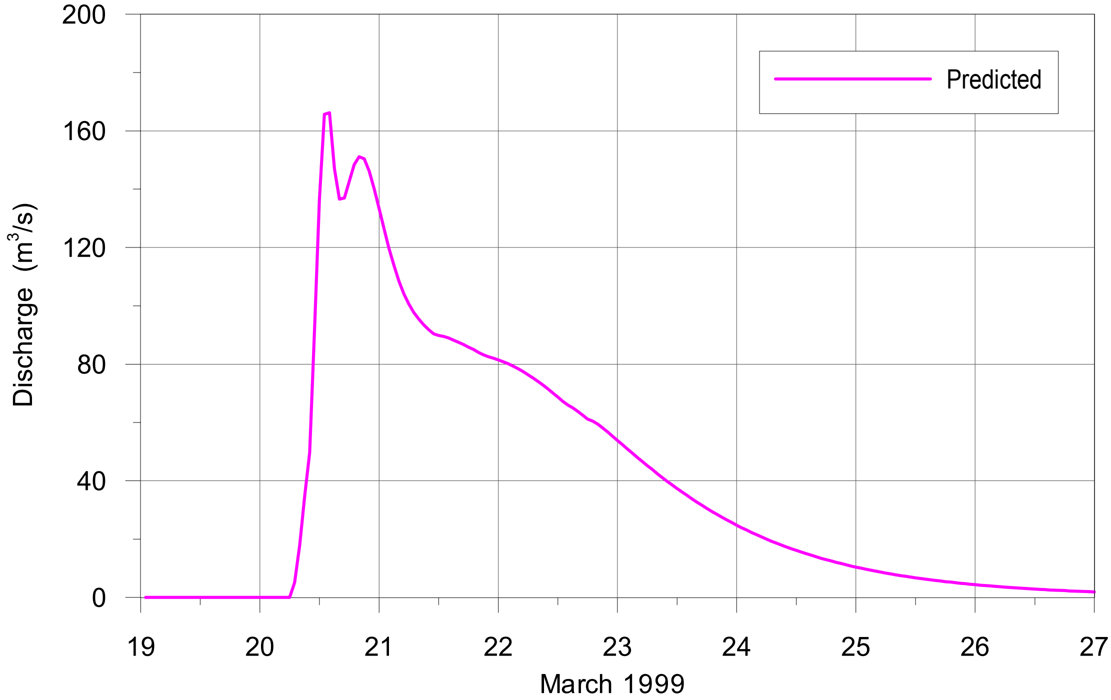

Figure 1 **Predicted Discharge Hydrograph, URBS Model, Moore River North - Nardy Road (617013), March 1999 Event**

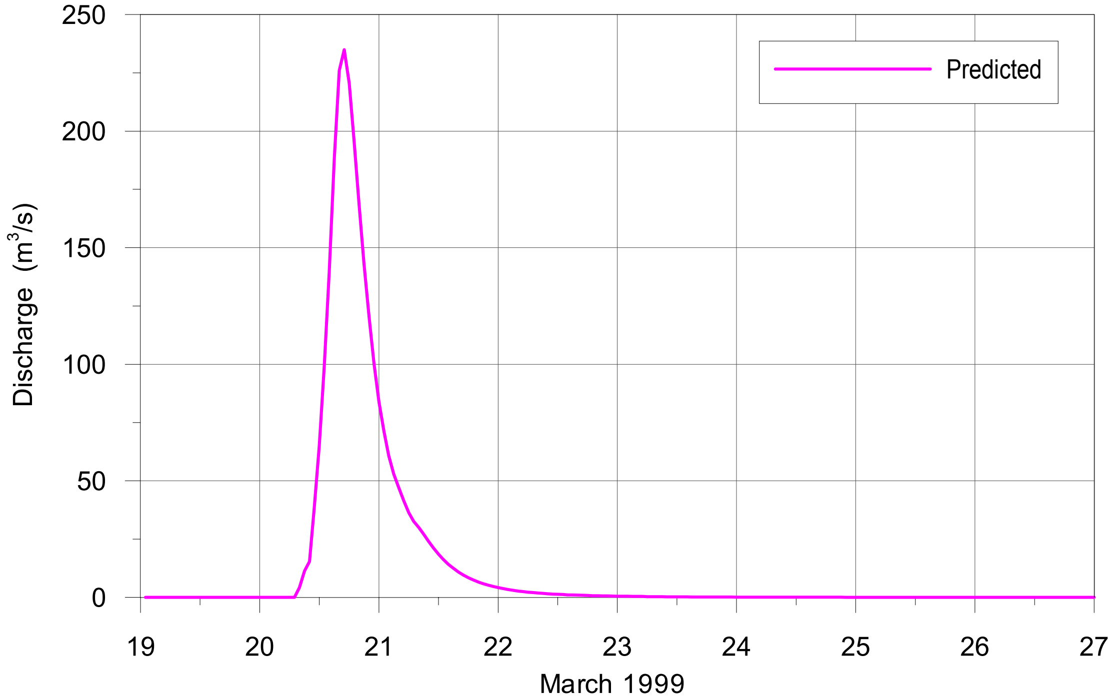

Figure 2 **Predicted Discharge Hydrograph, URBS Model, Dungaroo Creek, Upstream Roundhill Bridge (617012), March 1999 Event**

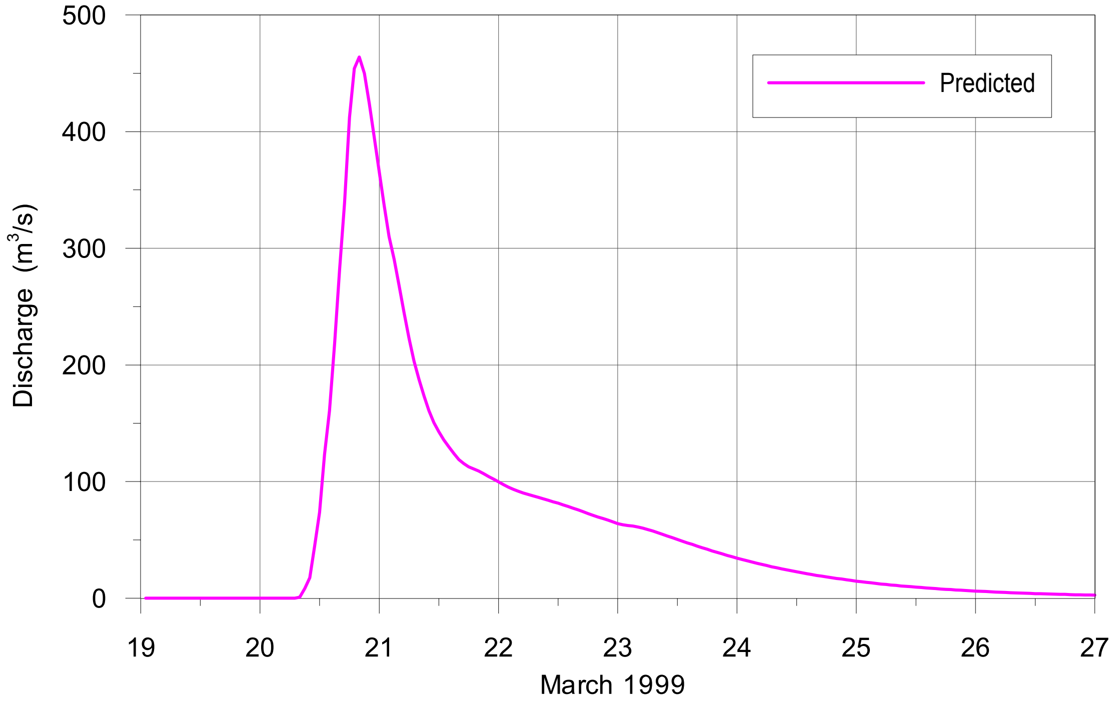

Figure 3 **Predicted Discharge Hydrograph, URBS Model, Moore River North - Longpool Bridge (617011), March 1999 Event**

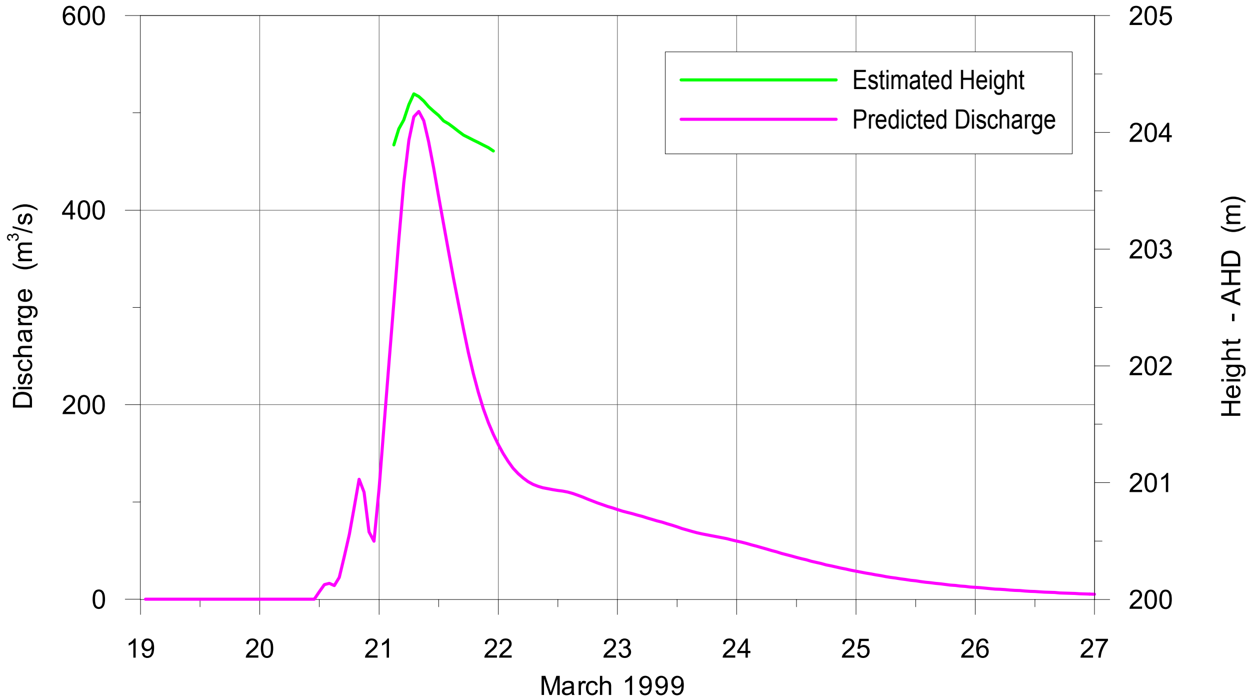

Figure 4 **Recorded Stage Hydrograph and Predicted Discharge Hydrograph, URBS Model, Moore River North - Moora Caravan Park (617010), March 1999 Event.**

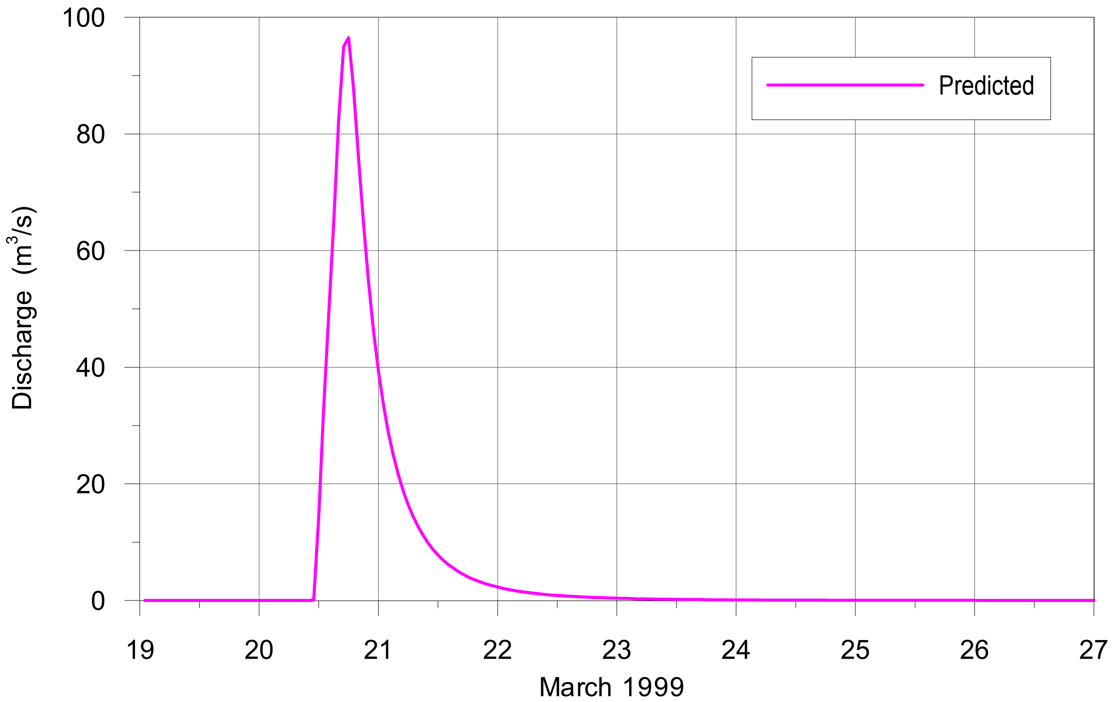

Figure 5 **Predicted Discharge Hydrograph, URBS Model, Yadgena Brook – Walebing Road, March 1999 Event**

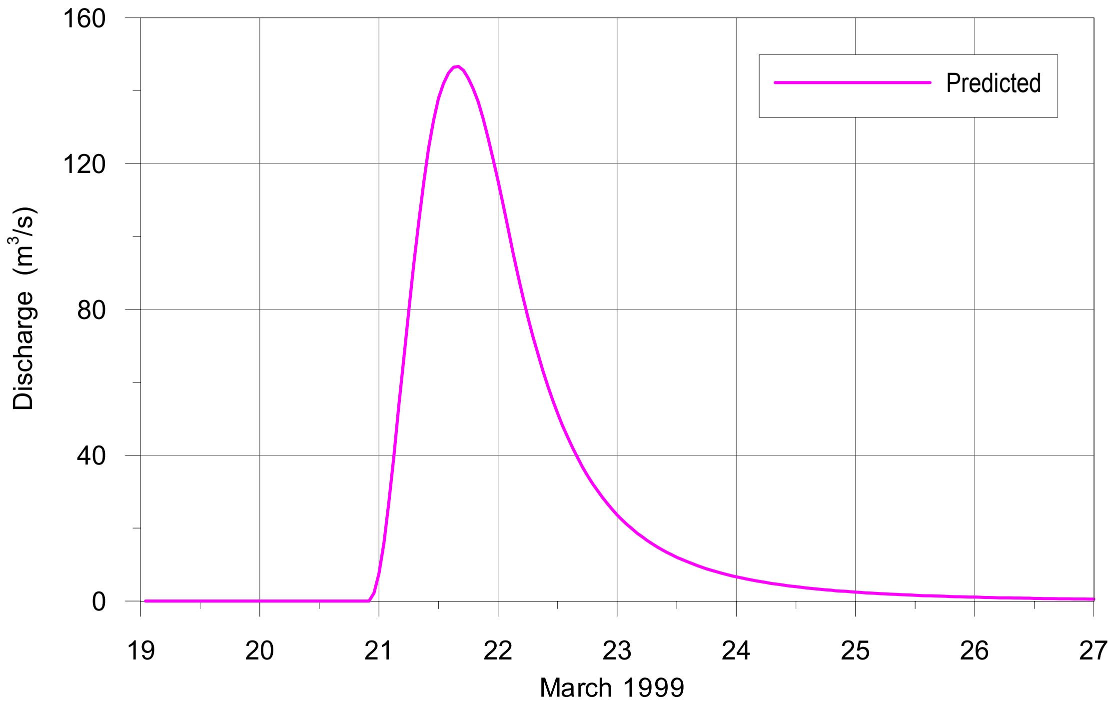

Figure 6 (617009), **Predicted Discharge Hydrograph, URBS Model, Moore River East - Woury Pool**

March 1999 Event

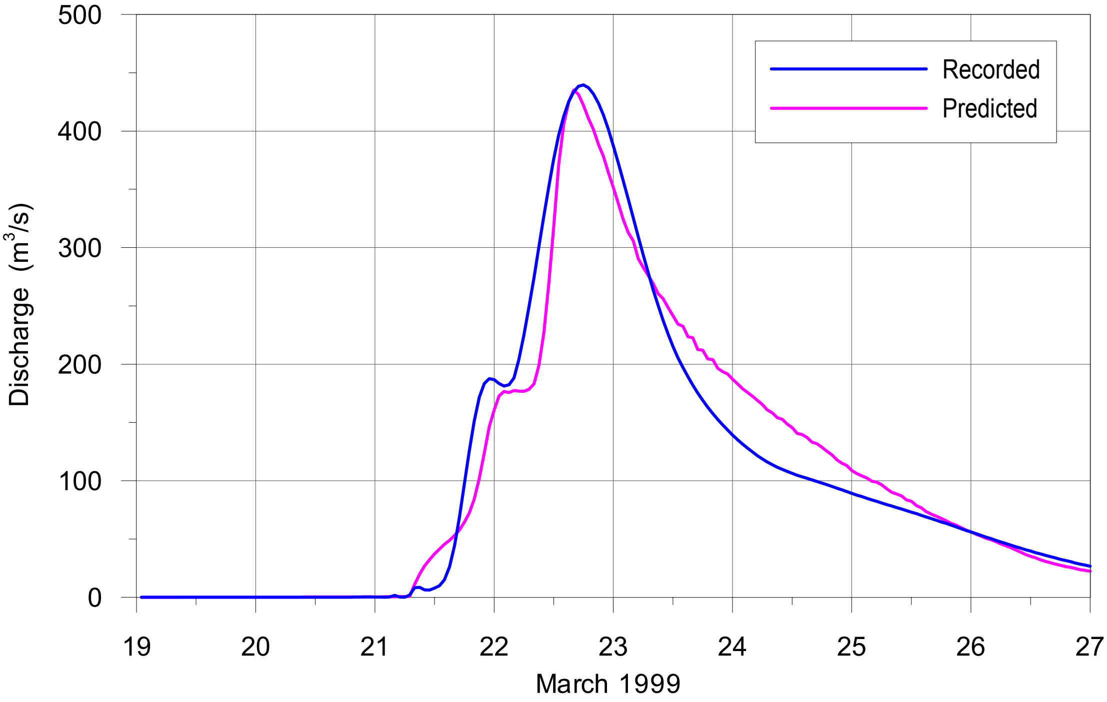

Figure 7 **Recorded and Predicted Discharge Hydrographs, URBS Model, Moore River - Quinns Ford (617001), March 1999 Event**

ADDENDUM D

Recorded and Predicted Discharges, URBS Model May 1999 Event

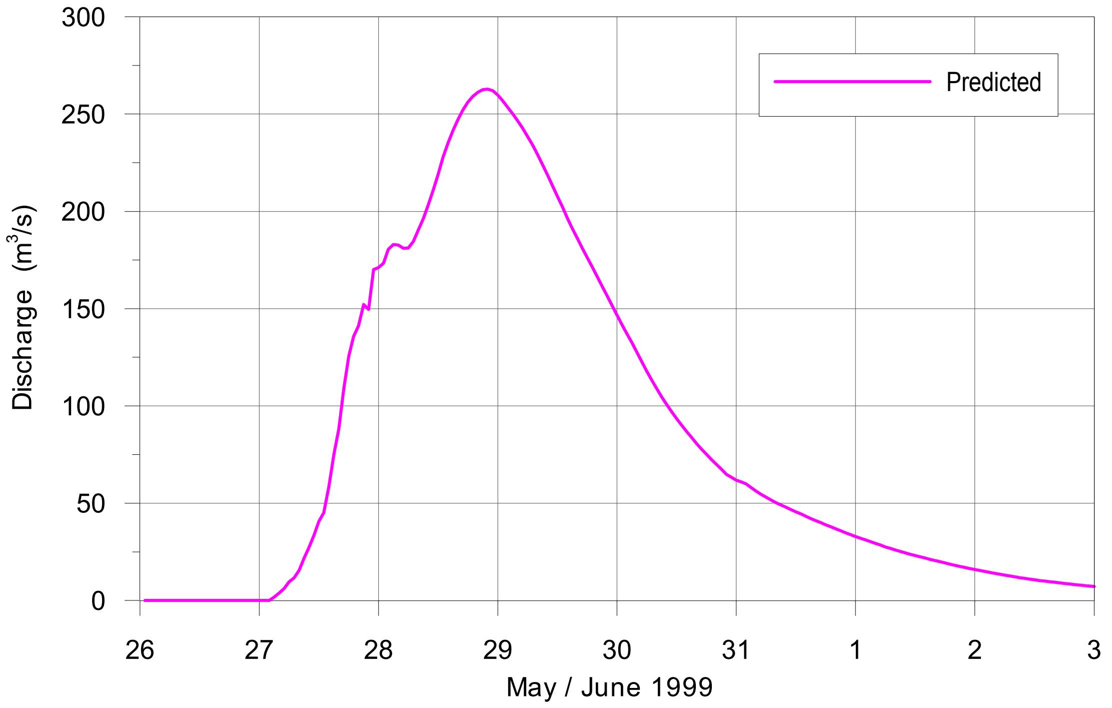

Figure 1 **Predicted Discharge Hydrograph, URBS Model, Moore River North - Nardy Road (617013), May 1999 Event**

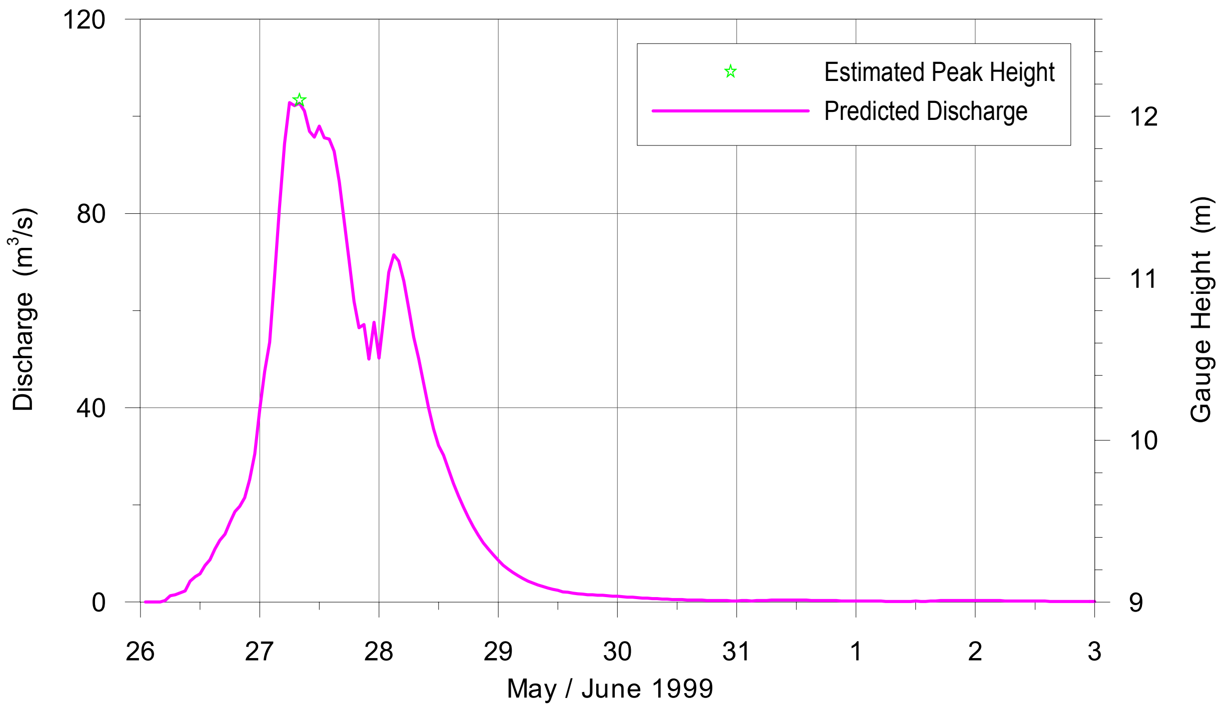

Figure 2 **Recorded Peak Gauge Height and Predicted Discharge Hydrograph, URBS Model, Dungaroo Creek, Upstream Roundhill Bridge (617012), May 1999 Event**

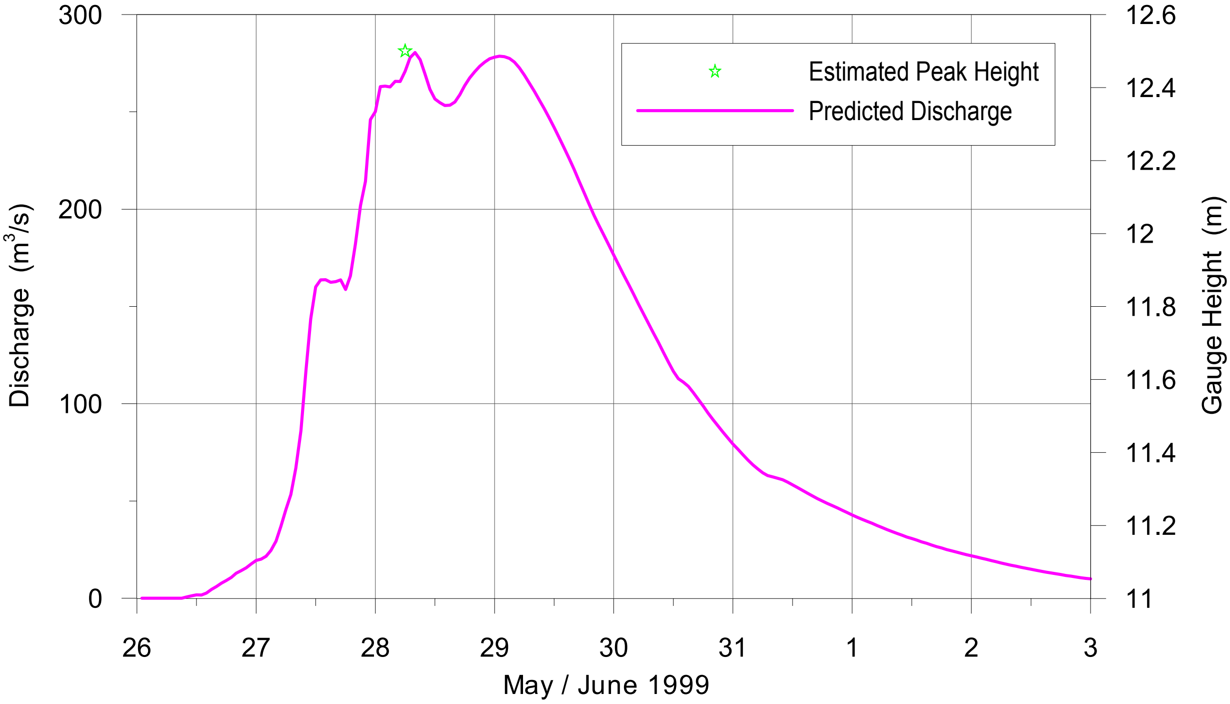

Figure 3 **Recorded Peak Gauge Height and Predicted Discharge Hydrograph, URBS Model, Moore River North - Longpool Bridge (617011), May 1999 Event**

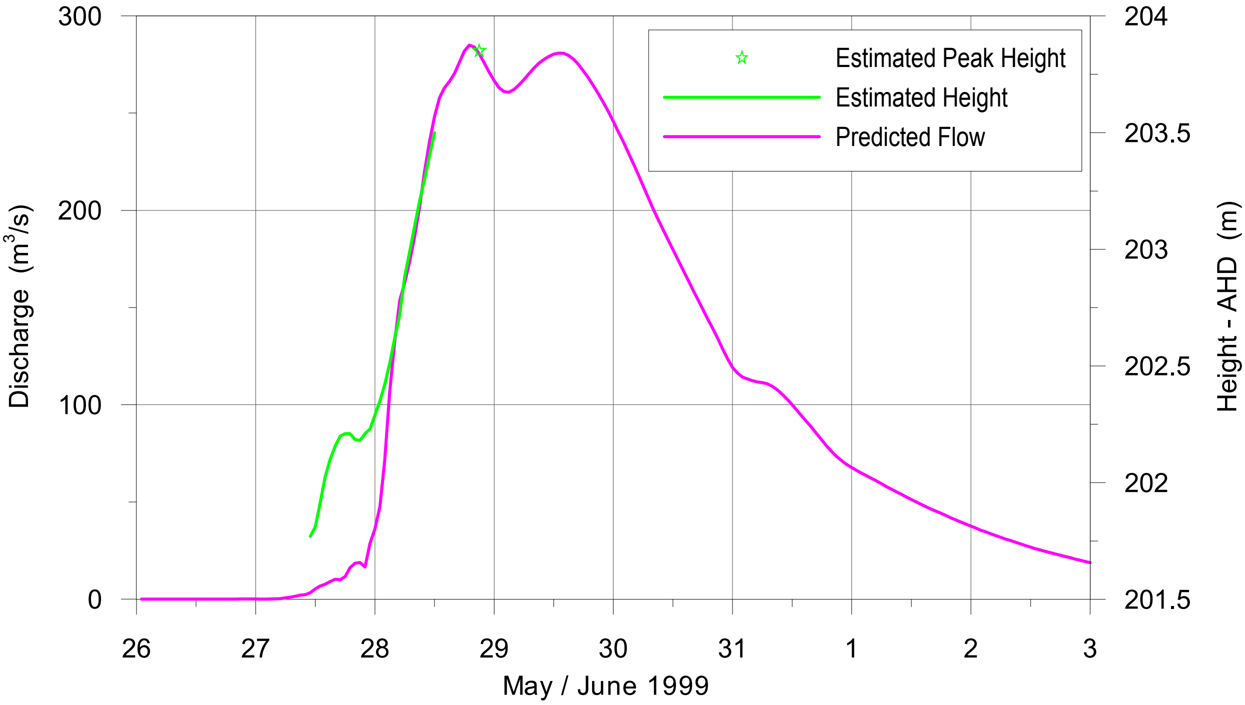

Figure 4 **Recorded Stage Hydrograph, Peak Stage Height and Predicted Discharge Hydrograph, URBS Model, Moore River North - Moora Caravan Park (617010), May 1999 Event.**

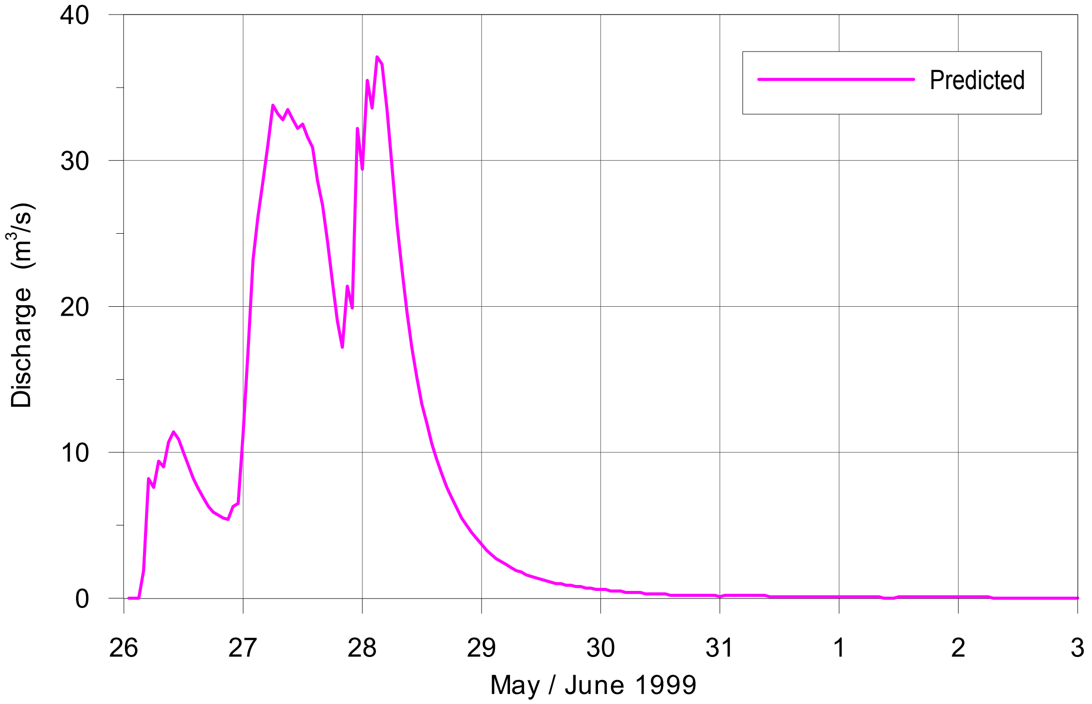

Figure 5 **Predicted Discharge Hydrograph, URBS Model, Yadgena Brook – Walebing Road, May 1999 Event**

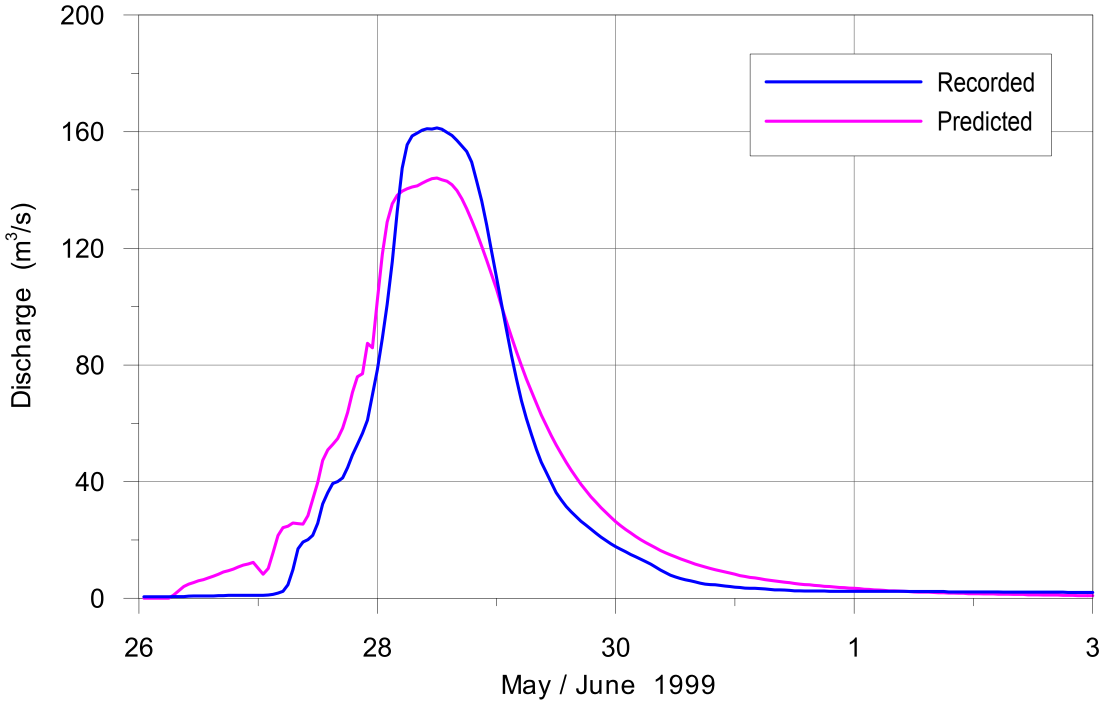

Figure 6 **Recorded and Predicted Discharge Hydrograph, URBS Model, Moore River East - Woury Pool (617009), May 1999 Event**

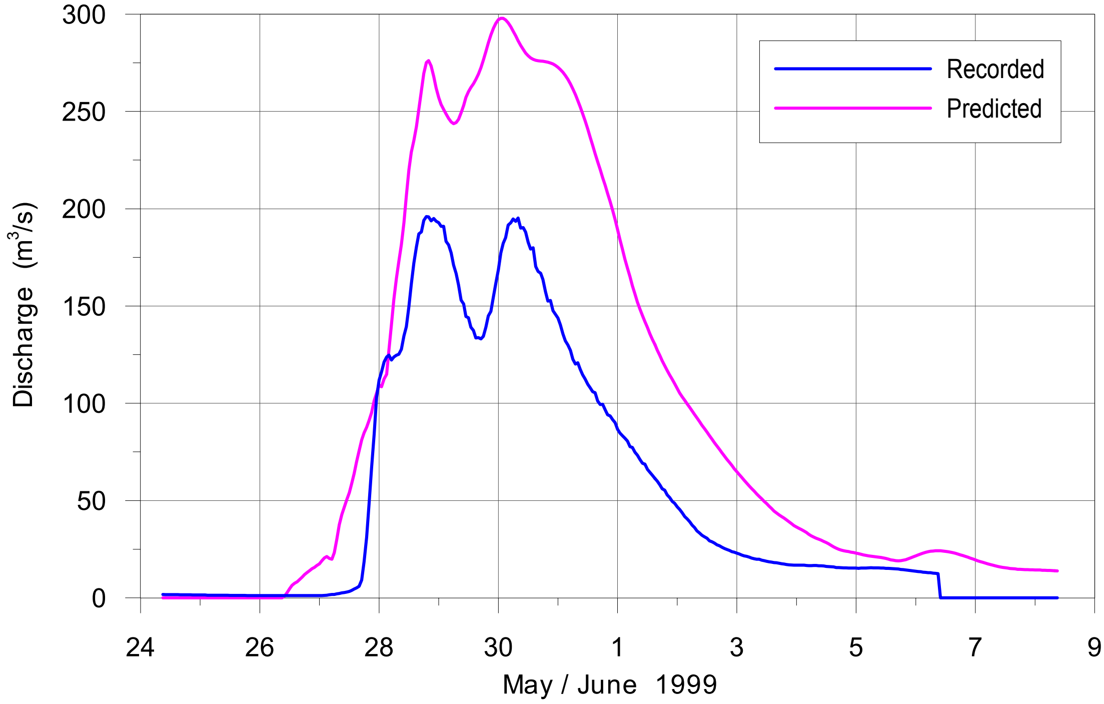

Figure 7 **Recorded and Predicted Discharge Hydrographs, URBS Model, Moore River - Quinns Ford (617001), May 1999 Event**

ADDENDUM E

Recorded and Predicted Discharges, URBS Model July 1999 Event

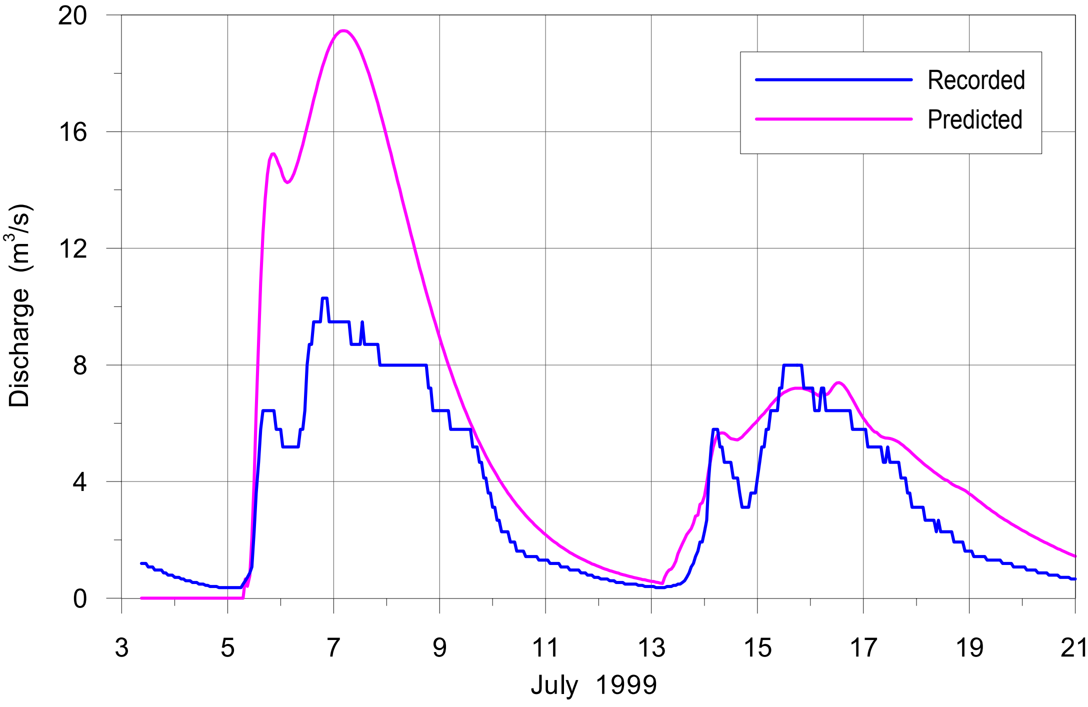

Figure 1 **Recorded and Predicted Discharge Hydrographs, URBS Model, Moore River North - Nardy Road (617013), July 1999 Event**

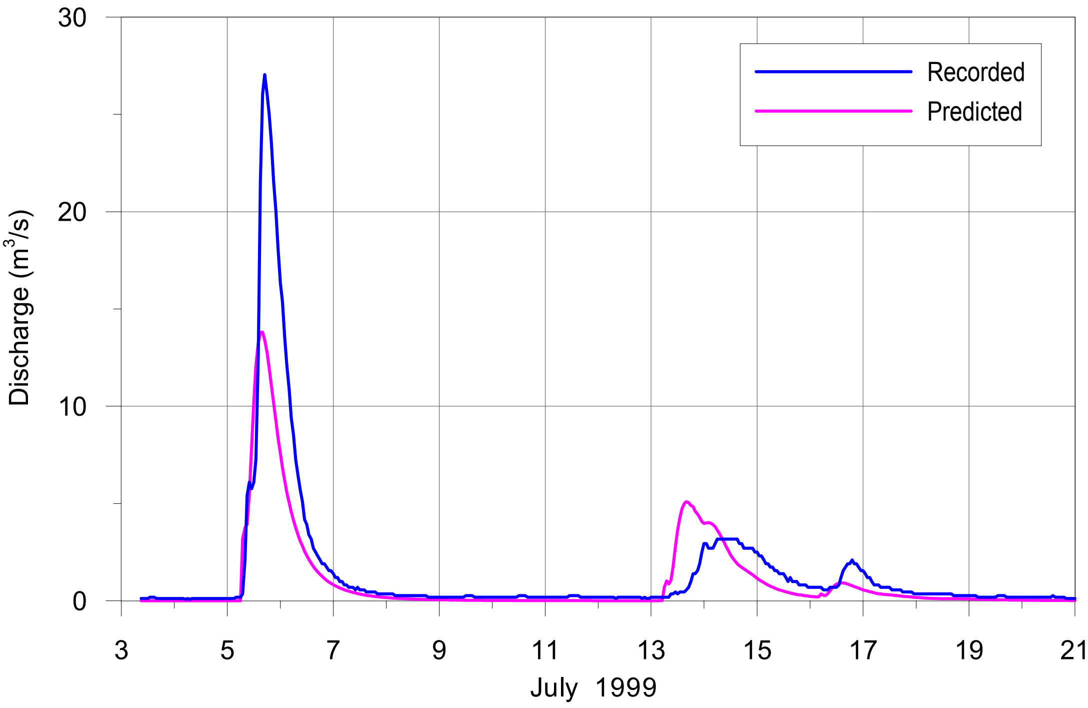

Figure 2 **Recorded and Predicted Discharge Hydrographs, URBS Model, Dungaroo Creek, Upstream Roundhill Bridge (617012), July 1999 Event**

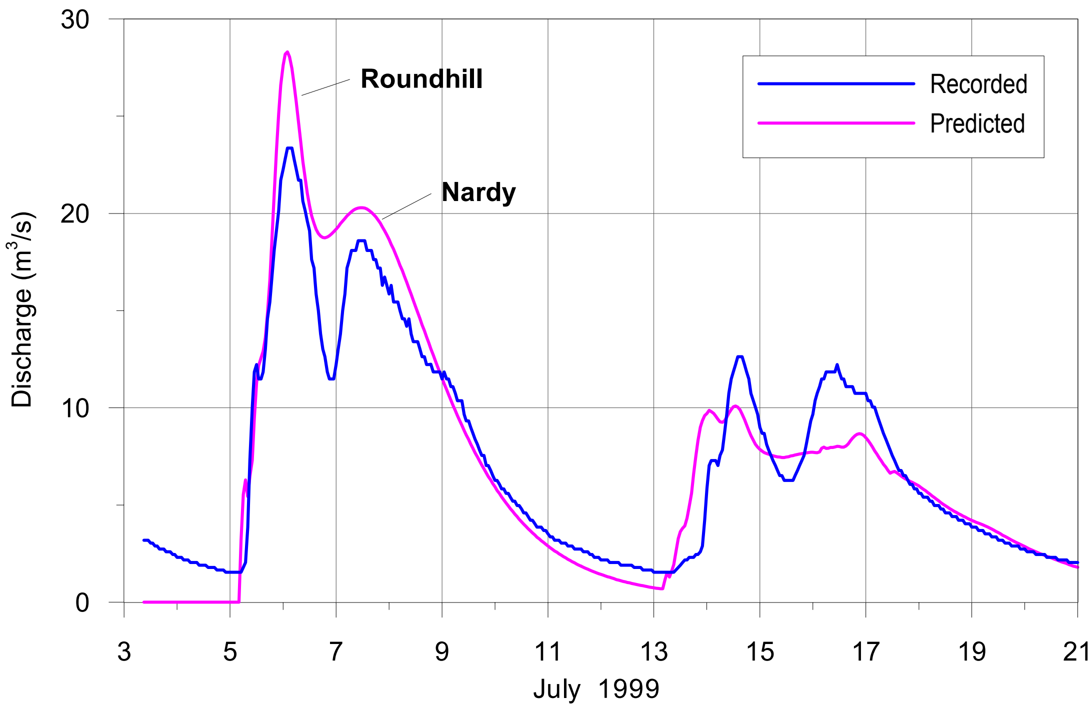

Figure 3 **Recorded and Predicted Discharge Hydrographs, URBS Model, Moore River North - Longpool Bridge (617011), July 1999 Event. Labels Indicate Contributing Sub-Catchments to Discharge Hydrograph.**

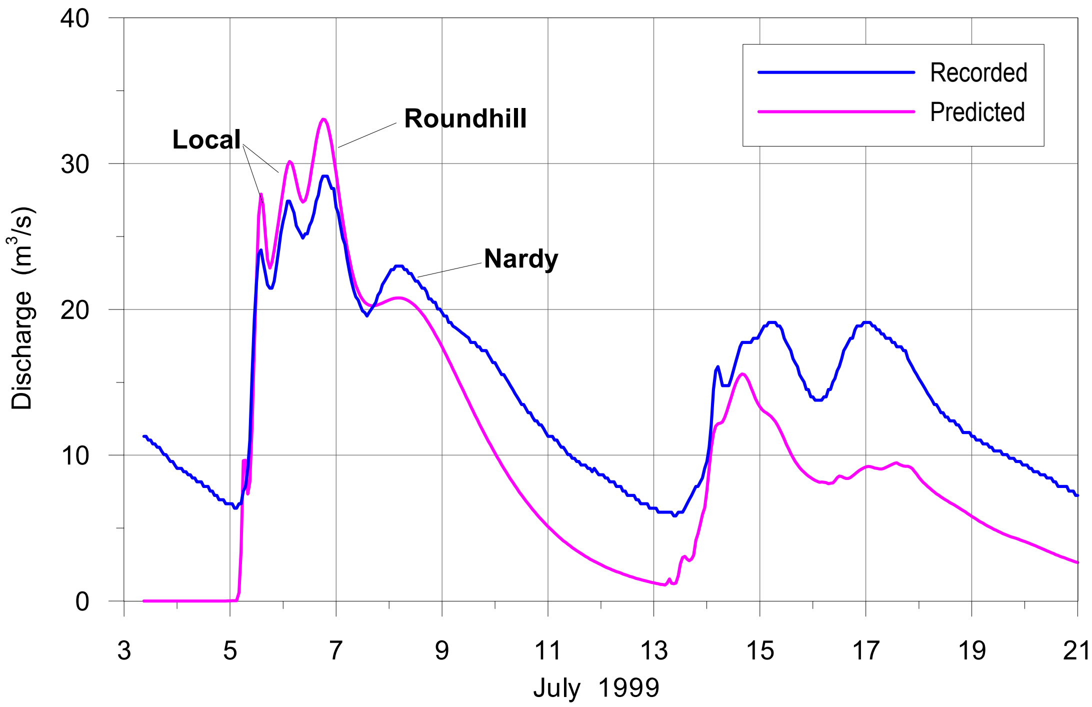

Figure 4 **Recorded and Predicted Discharge Hydrographs, URBS Model, Moore River North - Moora Caravan Park (617010), July 1999 Event. Labels Indicate Contributing Sub-Catchments to Discharge Hydrograph.**

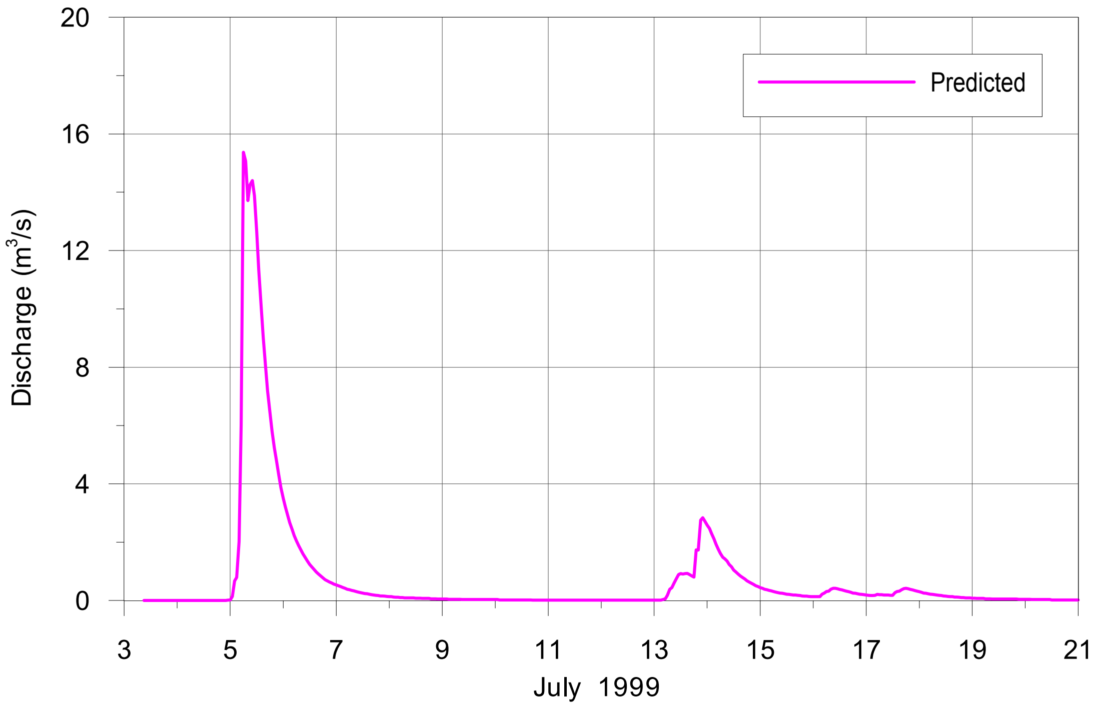

Figure 5 Recorded Stream Height and Predicted Discharge Hydrographs, URBS Model, Yadgena Brook – Walebing Road, July 1999 Event

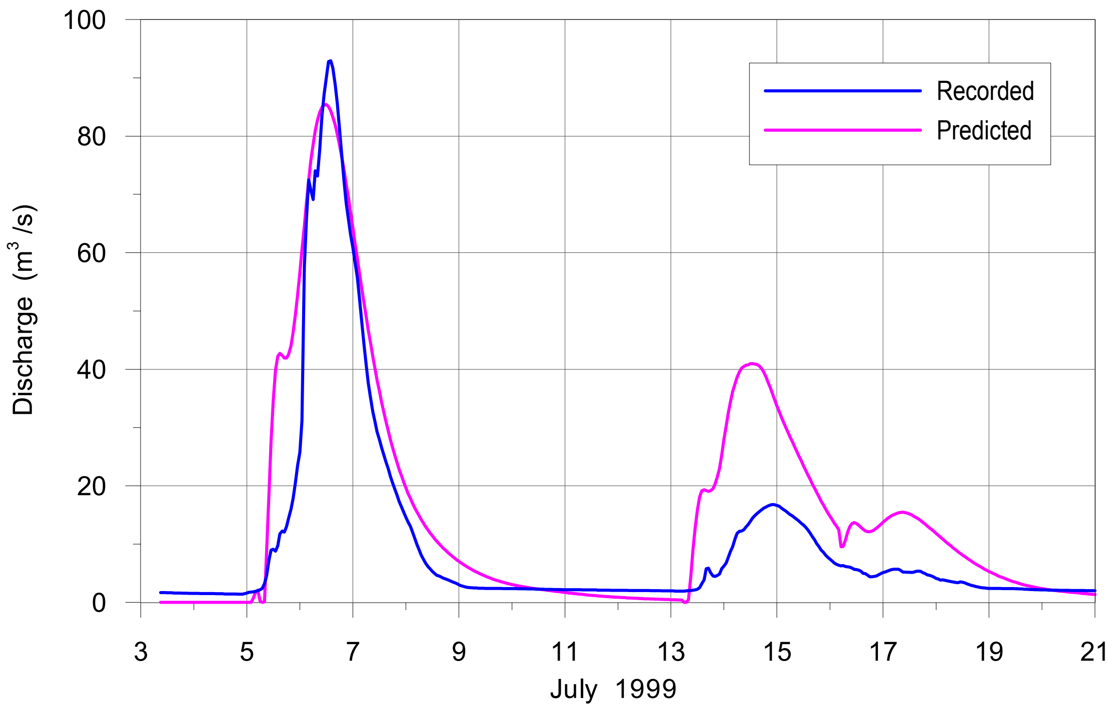

Figure 6 Recorded and Predicted Discharge Hydrographs, URBS Model, Moore River East - Woury Pool (617009), July 1999 Event

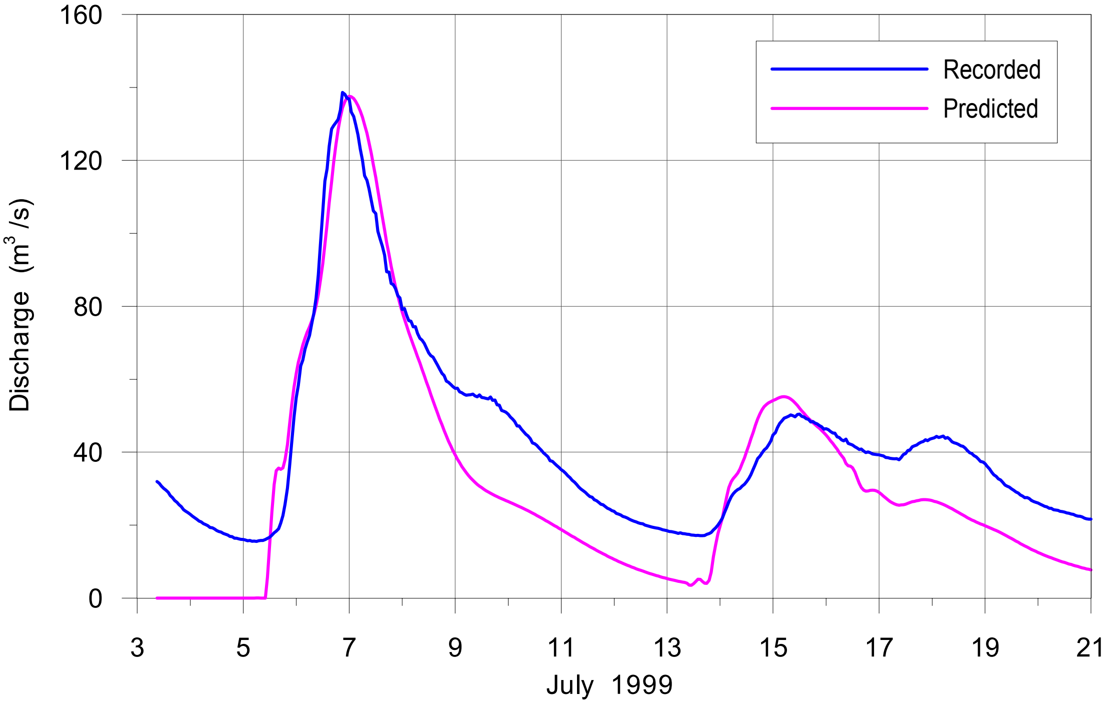

Figure 7 **Recorded and Predicted Discharge Hydrographs, URBS Model, Moore River - Quinns Ford (617001), July 1999 Event**

ADDENDUM F

Recorded and Predicted Discharges, URBS Model August 1999 Event

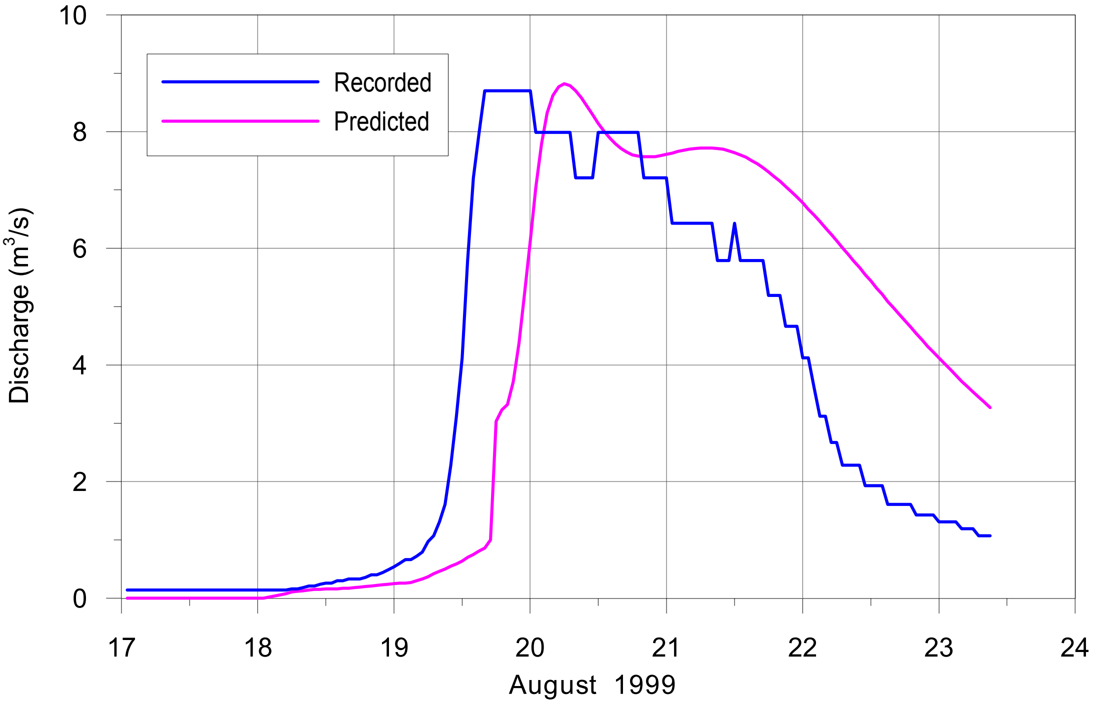

Figure 1 Recorded and Predicted Discharge Hydrographs, URBS Model, Moore River North - Nardy Road (617013), August 1999 Event

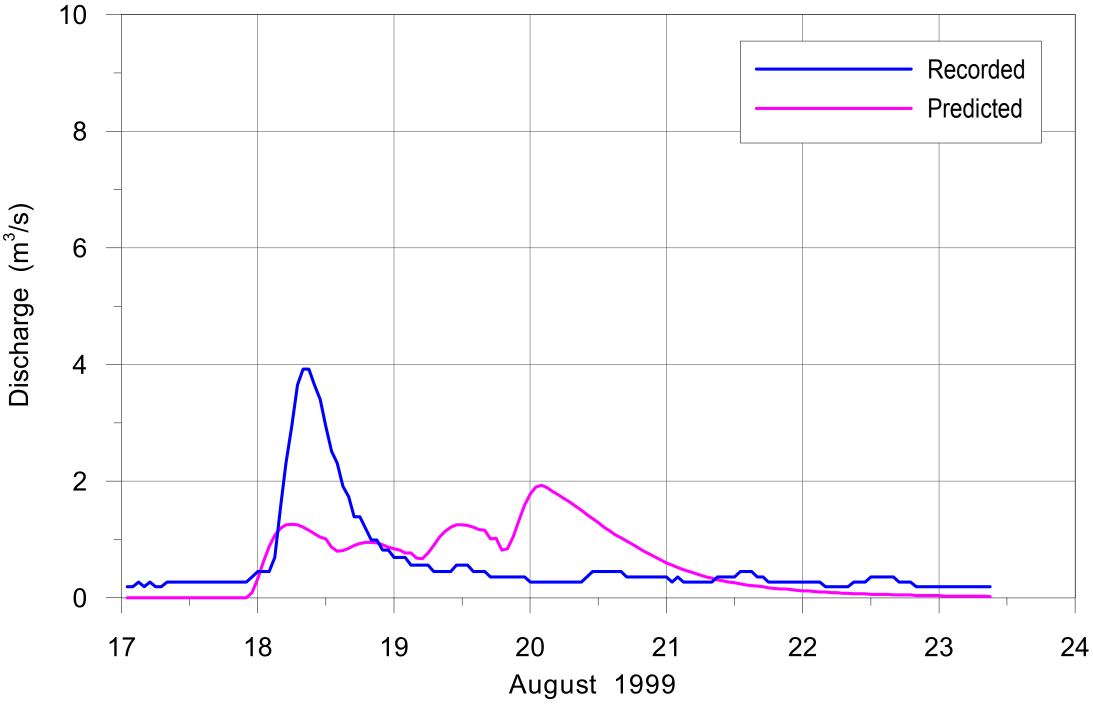

Figure 2 Recorded and Predicted Discharge Hydrographs, URBS Model, Dungaroo Creek, Upstream Roundhill Bridge (617012), August 1999 Event

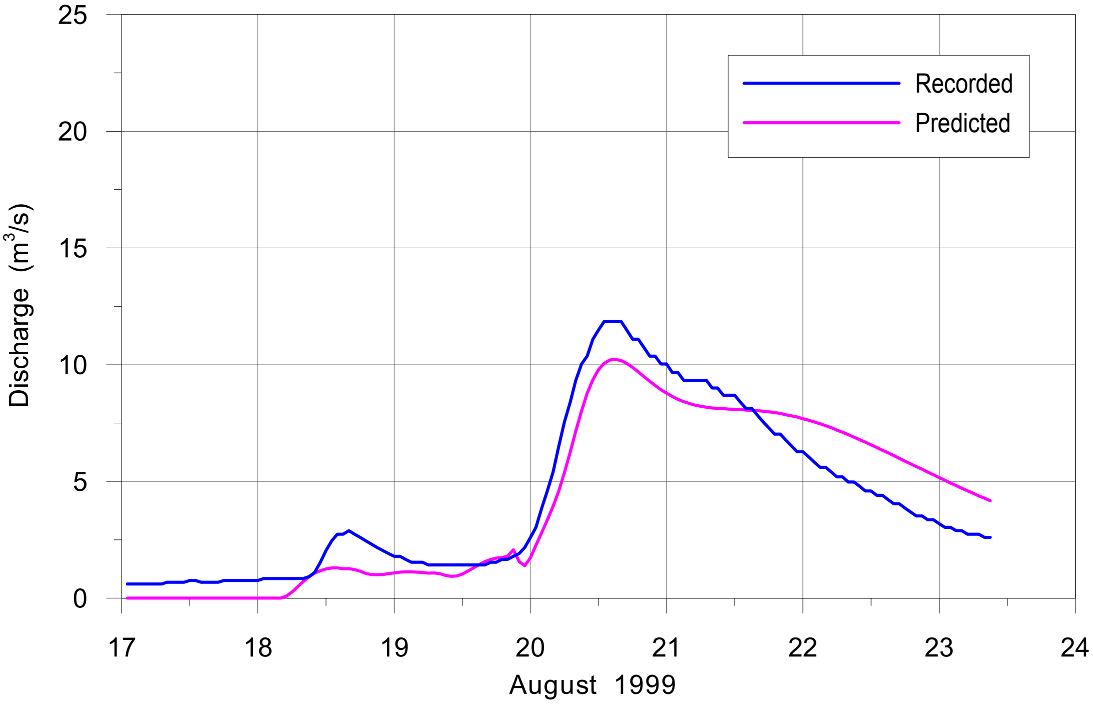

Figure 3 **Recorded and Predicted Discharge Hydrographs, URBS Model, Moore River North - Longpool Bridge (617011), August 1999 Event**

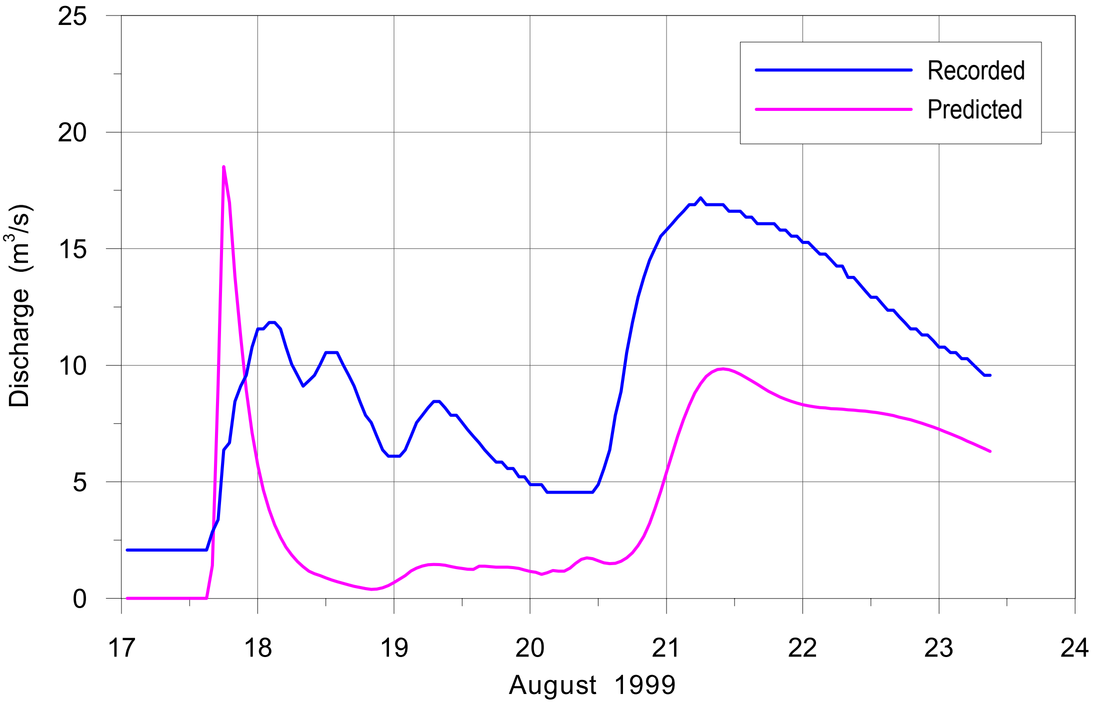

Figure 4 **Recorded and Predicted Discharge Hydrographs, URBS Model, Moore River North - Moora Caravan Park (617010), August 1999 Event**

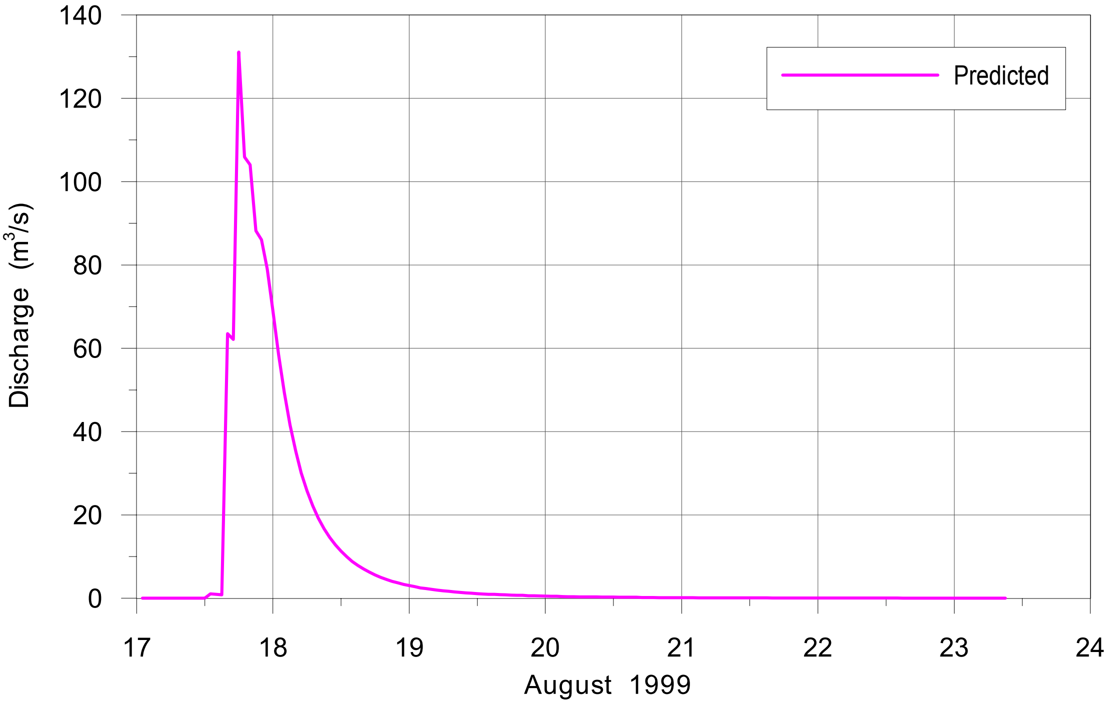

Figure 5 **Predicted Discharge Hydrographs, URBS Model, Yadgena Brook – Walebing Road, August 1999 Event**

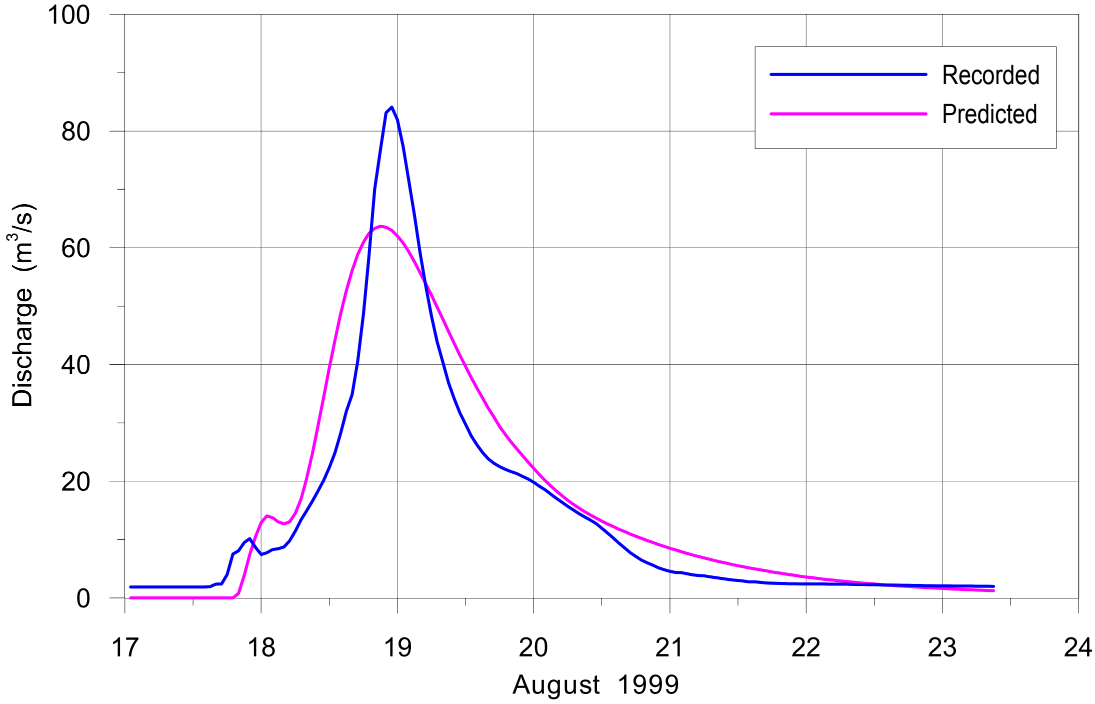

Figure 6 **Recorded and Predicted Discharge Hydrograph, URBS Model, Moore River East - Woury Pool (617009), August 1999 Event**

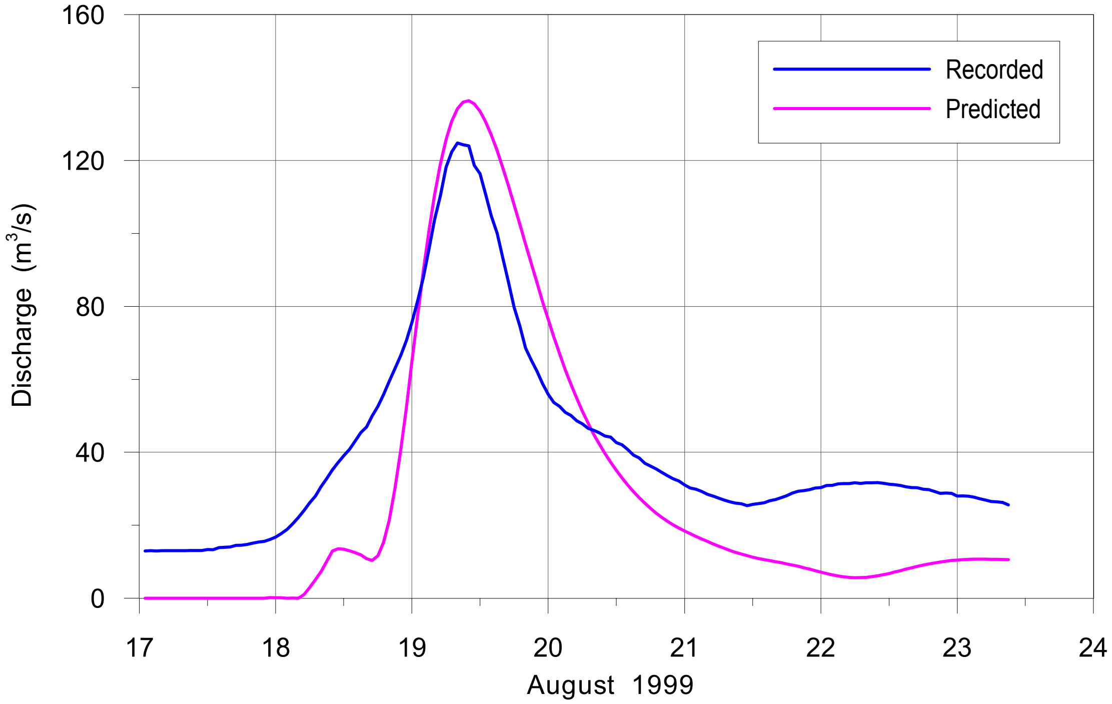

Figure 7 **Recorded and Predicted Discharge Hydrographs, URBS Model, Moore River - Quinns Ford (617001), August 1999 Event**

ADDENDUM G

Rating Curves Estimated by the URBS Model for Nardy Road and Roundhill Bridge Stream Gauging Stations

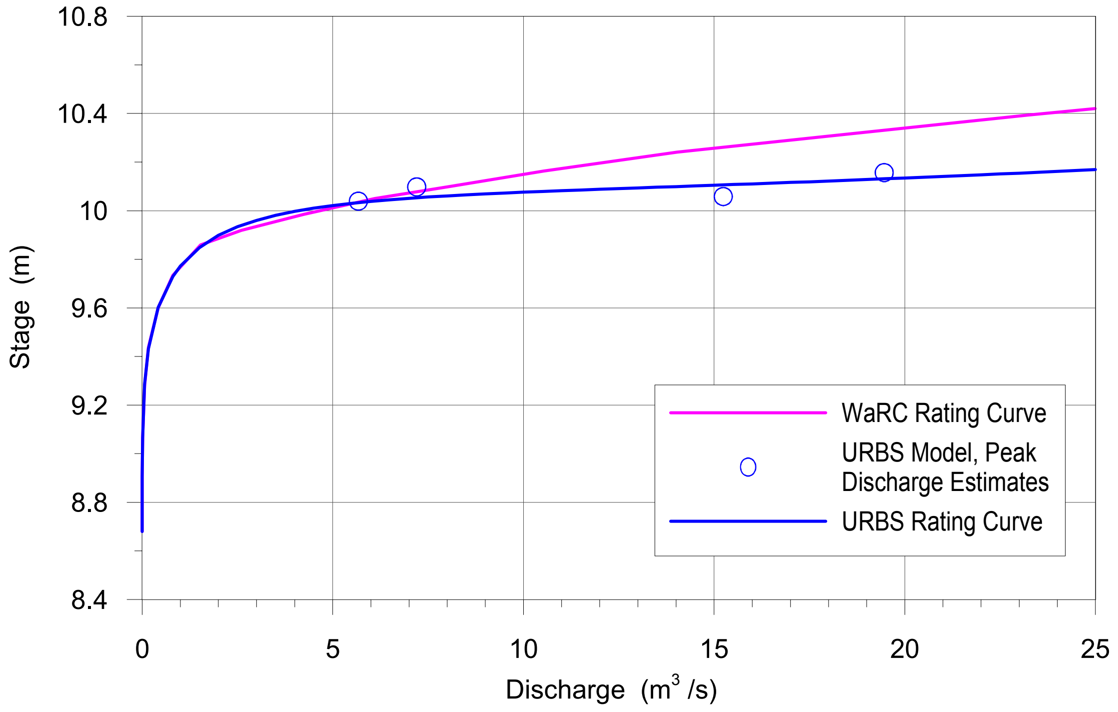

Figure 1 Rating Curves for Moore River North - Nardy Road (617 013) Gauging Station

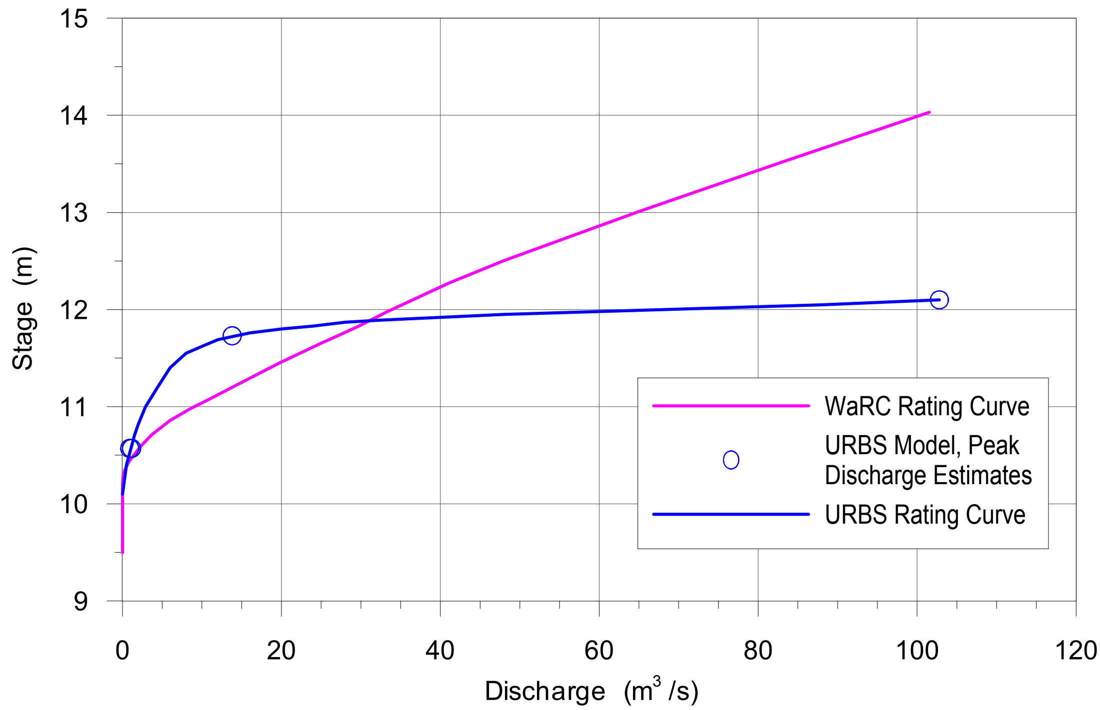

Figure 2 Rating Curves for Moore River North, Upstream of Roundhill Bridge (617 012) Gauging Station

Table 1 Rating Curve for Moore River North u/s Nardy Road Derived from URBS model output

Stage (m)	Discharge (m^3/s)
8.680	0.000
8.800	0.000
8.906	0.002
9.066	0.009
9.286	0.060
9.432	0.162
9.601	0.419
9.730	0.807
9.852	1.531
9.940	2.604
10.004	4.236
10.040	6.101
10.062	8.088
10.080	10.541
10.100	14.031
10.124	18.321
10.156	23.207
10.214	25.000

Table 2 Rating Curve for Dungaroo Creek u/s Roundhill Bridge Derived from URBS Model Output

Stage (m)	Discharge (m^3/s)
10.10	0.00
10.40	0.50
10.62	1.23
10.82	2.00
11.00	2.90
11.20	4.42
11.40	6.00
11.62	10.00
11.69	12.00
11.76	16.00
11.80	20.00
11.83	24.00
11.89	32.00
11.95	48.00
12.00	68.00
12.05	88.00
12.10	102.80

B Appendix HYDRAULIC MODEL SET UP AND CALIBRATION

B1 MODEL DESCRIPTION

B.01 The MIKE 11 hydrodynamic model was used for the hydraulic modelling of this study. MIKE 11 is an implicit, finite difference model for the computation of unsteady flows in rivers and estuaries. The model can analyse subcritical as well as supercritical flows using a numerical scheme that adapts according to the local flow conditions (in time and space). Advanced computational modules are included in the model for the analysis of flow over and through hydraulic structures. The model can be applied to looped networks and quasi two-dimensional flow simulation on floodplains. The computational scheme of the model is applicable for vertically homogenous flow conditions extending from steep river flows to tidally influenced estuaries. The MIKE 11 modelling system has been used in numerous engineering studies around the world (DHI, 1992).

B.02 In MIKE 11, the floodplain area to be modelled is represented by a series of 'branches'. The branches start and end at model boundaries represented by a discharge hydrograph at the upstream boundary and a rating curve or a stage hydrograph at the downstream boundary. Branches can also be connected to each other to model the distribution of flow across a floodplain. The branches are represented by a series of 'H' points and 'Q' points. Surveyed cross sections are located at 'H' points to estimate peak flood levels, stage hydrographs and flood depths. 'Q' points are located centrally between 'H' points to show the distribution of flow between the branches. Hydraulic structures, such as bridges, culverts, weirs or a combination thereof are also located at 'Q' points. The floodplain storage area is defined at each 'H' point as the summation of the flooded width times the distance between the 'Q' points on either side of the cross section.

B.03 A series of link branches are used to distribute flows between the channel branches. Link branches do not contain any floodplain storage.

B2 MIKE 11 MODEL CONFIGURATION

B.04 The MIKE 11 hydraulic model configuration of the Moore River extends from about 4 km to the north-east of Moora Railway Bridge to Webb Street to the southeast of Moora. The model also includes the Coonderoo River to about 800 m upstream of Ferguson Road and Yadgena Brook to the Moora-Walebing Road Bridge. The model consists of 15 channel branches, 58 link branches, 168 cross sections and 4 boundary conditions. The four boundary conditions include:

- Inflow hydrographs at the Moore River and Coonderoo River and Yadgena Brook upstream boundaries.
- A rating curve at the downstream boundary of the model.

The adopted model configuration is shown in Figure B2.1. Table B2.1 provides details on reach lengths, junction locations and branch types used in the model.

B3 AVAILABLE TOPOGRAPHIC DATA

B.05 Topographic data available from various sources was used in the Moora MIKE 11 model:

- Cross section data surveyed in 1989 for the previous flood study of the Moore River (GHD, 1991) were provided by WRC. The 1989 survey was mostly confined to the main channels of the Moore River, the Coonderoo River and Yadgena Brook.

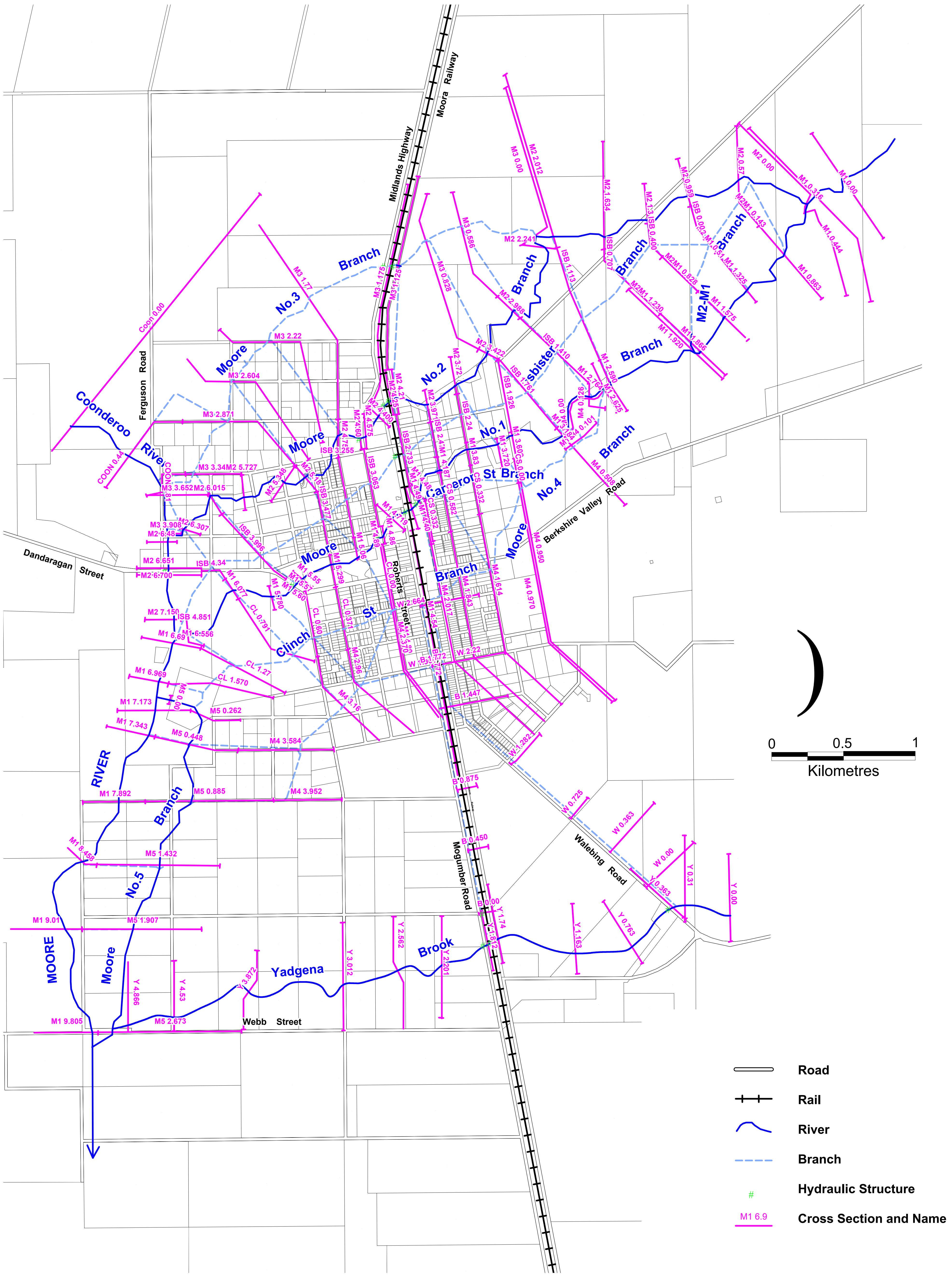

Figure B2.1 MIKE 11 Hydraulic Model Configuration of the Moora Township Area.

Table B2.1 MIKE 11 Hydraulic Model Configuration Details, Moora

Branch Name	Branch Length (m)	Upstream End		Downstream End		Branch Type
		Branch Name	Chainage (m)	Branch Name	Chainage (m)	
Moore 1	10656	-	-	-	-	Regular
Yadgena Brook	4866	-	-	Moore 1	9910	Regular
Coonderoo	810	-	-	Moore 3	3652	Regular
Moore 5	2673	Moore 1	7173	Moore 1	9910	Regular
Moore 2	7147	Moore 1	316	Moore 1	6690	Regular
Moore 3	3908	Moore 2	2118	Moore 2	6480	Regular
M2-M1	1230	Moore 2	570	Moore 1	1920	Regular
Isbister	4851	M2-M1	510	Moore 1	6690	Regular
Moore 4	3952	Moore 1	2625	Moore 5	885	Regular
Clinch St	1564	Moore 4	2360	Moore 5	262	Regular
Cameron St	712	Moore 4	950	Moore 1	4480	Regular
M1m4-1	101	Moore 1	3184	Moore 4	508	Regular
Walebing Rd	2664	Yadgena Brook	364	Moore 4	2125	Regular
Mogumber Rd	2208	Yadgena Brook	1740	Moore 4	2125	Regular
Link 1	80	Moore 1	863	Moore 2	570	Link Channel
Link 2	65	Moore 1	1325	M2-M1	510	Link Channel
Link 3	63	M2-M1	510	Moore 2	959	Link Channel
Link 4	56	Moore 1	1575	M2-M1	828	Link Channel
Link 5	57	M2-M1	828	Isbister	400	Link Channel
Link 6	73	Isbister	400	Moore 2	1300	Link Channel
Link 7	83	Moore 1	1866	M2-M1	1200	Link Channel
Link 8	70	M2-M1	1200	Isbister	707	Link Channel
Link 9	44	Isbister	707	Moore 2	1634	Link Channel
Link 10	45	Moore 1	2590	Isbister	1113	Link Channel
Link 11	74	Isbister	1113	Moore 2	2012	Link Channel
Link 12	51	Moore 1	2764	Isbister	1410	Link Channel
Link 13	65	Isbister	1410	Moore 2	2988	Link Channel
Link 14	117	Moore 2	2988	Moore 3	586	Link Channel
Link 15	61	Moore 1	3184	Isbister	1761	Link Channel
Link 16	63	Isbister	1761	Moore 2	3422	Link Channel
Link 17	92	Moore 2	3422	Moore 3	828	Link Channel
Link 18	114	Moore 4	950	Moore 1	3720	Link Channel
Link 19	74	Moore 1	3720	Isbister	2240	Link Channel
Link 20	67	Isbister	2240	Moore 2	3720	Link Channel
Link 22	179	Moore 4	1614	Cameron St	582	Link Channel
Link 22a	68	Cameron St	582	Moore 1	4187	Link Channel
Link 23	92	Moore 1	4187	Isbister	2478	Link Channel
Link 24	75	Isbister	2478	Moore 2	3971	Link Channel
Link 60	140	Moore 4	2017	Moore 1	4540	Link Channel
Link 61	116	Moore 1	4540	Isbister	2733	Link Channel
Link 62	149	Isbister	2733	Moore 2	4250	Link Channel
Link 63	450	Moore 2	4210	Moore 3	1125	Link Channel
Link 27	130	Moore 1	5060	Isbister	3477	Link Channel
Link 28	110	Isbister	3477	Moore 2	5180	Link Channel
Link 29	325	Moore 2	5180	Moore 3	2604	Link Channel
Link 25	395	Moore 2	4756	Moore 3	2220	Link Channel
Link 26	154	Moore 4	2360	Moore 1	5060	Link Channel
Link 30	168	Moore 4	2960	Clinch St	600	Link Channel
Link 31	352	Clinch St	600	Moore 1	5780	Regular
Link 33	204	Isbister	3996	Moore 2	6015	Link Channel
Link 34	68	Moore 2	6015	Moore 3	3340	Link Channel
Link 35	75	Moore 3	3340	Coonderoo	800	Link Channel
Link 32	115	Moore 1	5780	Isbister	3996	Link Channel
Link 36	181	Moore 4	3160	Clinch St	791	Link Channel
Link 37	230	Clinch St	791	Moore 1	6077	Link Channel
Link 38	63	Moore 1	6077	Isbister	4340	Link Channel
Link 39	110	Isbister	4340	Moore 2	6307	Link Channel
Link 40	183	Moore 2	6307	Moore 3	3652	Link Channel
Link 41	143	Clinch St	1270	Moore 1	6556	Link Channel
Link 44	122	Clinch St	1527	Moore 1	6969	Link Channel
Link 45	330	Moore 4	3584	Moore 5	448	Link Channel
Link 46	63	Moore 5	448	Moore 1	7343	Link Channel
Link 48	179	Moore 5	885	Moore 1	7892	Link Channel
Link 50	255	Moore 5	1432	Moore 1	8458	Link Channel
Link 52	207	Moore 5	1907	Moore 1	9010	Link Channel
Link 53	434	Coonderoo	440	Moore 3	2871	Link Channel
Link 64	142	Moore 1	4719	Isbister	3063	Link Channel
Link 65	139	Isbister	3063	Moore 2	4575	Link Channel
Link 66	23	Walebing Rd	1906	Mogumber Rd	1447	Link Channel
Link 67	21	Walebing Rd	2220	Mogumber Rd	1772	Link Channel
Link 68	272	Mogumber Rd	1772	Moore 4	2960	Link Channel
Link 69	46	Cameron St	332	Moore 1	3930	Link Channel

- Detailed survey of the road bridges in Moora and cross section surveys along some of the major roads in the town area were undertaken by Fugro Pty Ltd in October 1999.

- Cross section surveys in the town area were undertaken by Morris Heyhoe and Richards in December 1999 upstream of Moora, along the Moore No. 4 branch, along Yadgena Brook and along the Walebing and Mogumber Roads.

- Half metre contour data for the study area shown on the 1991 Moore River Flood Study plans for Moora Town (Plan No's CC98-1, 2, and 3) was provided by WRC.

- Digital half metre contour data of the study area was also provided by WRC was also used.

B.06 Data from the 1989 and 1999 cross section surveys generally compared well in areas common to both surveys. The representativeness of the 1989 survey in other areas is not known. Some widening of channels may have occurred as a result of the 1999 floods. Some road levels may have also changed. Where possible, the 1999 survey data was used in the hydraulic model instead of the 1989 survey data. The half metre contour data shown on the Moore River Flood Study maps (Plan Nos CC98-1, 2 and 3) also compared well with the 1989 and 1999 cross section survey data. However, the digital half metre contour data varied considerably from the surveyed cross section data. Errors were encountered in the town area as well as in the open grassland areas. As a result, the digital half metre contour data was only used to obtain general information on the relative level differences across the study area.

B.07 The 1991 Moore River Flood Study plans for Moora Town (i.e; half metre contour plans) were used to define the configuration of the MIKE 11 hydraulic model within Moora town area. The digital half metre contour data was used to define the model configuration outside the town area. The branch layout was defined and drawn onto a hardcopy of these maps by manually identifying potential flow paths and controlling sections across the floodplain on the basis of all available topographical data.

B4 HYDRAULIC STRUCTURES

B4.1 Hydraulic Structures Modelled

B.08 The locations of the hydraulic structures included in the Moora MIKE 11 model are given in Table B4.1. Note that there are several small pipe culverts along Moore River No. 3 and Moore River No. 4 that were ignored in the hydraulic model. Also, the Moore River No. 4 pipe culvert between Melbourne and Long Streets has been ignored. These culverts are in place to drain local runoff rather than flood flows in the Moore River. The capacity of each of these culverts is no more than 2 m^3/s. During significant flood events, these small culverts, including the Moore River No. 4 culvert between Melbourne and Long Street, will drown out and convey little water and have no impact on peak flood levels.

B4.2 Afflux at Bridges and Culverts

B.09 The MIKE 11 model calculates afflux generated at bridges and culverts using different algorithms for critical flow, drowned flow and pipe full flow as appropriate. For estimation of afflux at culverts, head loss factors such as contraction, expansion, friction and bend losses are incorporated into these algorithms. In the MIKE 11 model, the head loss factors that represent the losses at bridges, such as losses due to piers, abutment shape, eccentricity and skew must be estimated using the culvert loss equations. This is a model simplification that has to be made to handle bridge structures. Little data is available to calibrate the model to accurately reflect the losses that occur at bridges. In the absence of better data, the standard culvert head loss factors were adopted for all bridges and culverts in the model. The adopted head loss factors are shown in Table B4.2.

Table B4.1 Hydraulic Structures Included in the Moora MIKE 11 Model

Branch Name	Model Chainage (m)	Location of Structure
Moore 1	2600	Access Road
Moore 1	4490	Gardiner Street
Moore 1	4588	Railway
Moore 1	4875	Roberts Street
Moore 1	5586	Dandaragan Street
Moore 2	4228	Tootra Street
Moore 2	4265	Railway
Moore 2	4590	Roberts Street
Moore 2/(Coonderoo)	6675	Dandaragan Street
Moore 3	1150	Moora Railway
Moore 3	1205	Midlands Highway
Isbister	2775	Moora Railway
Yadgena Brook	392	Walebing Road
Yadgena Brook (x2)	1795	Railway
Yadgena Brook	1843	Mogumber Road

Table B4.2 Adopted Head Loss Factors for Bridges and Culverts, Moora MIKE 11 Model

Loss Type	Value
Contraction loss	0.5
Expansion Loss	1.0
Bend Loss	0.0
Manning's n (bridges)	0.03
Manning's n (concrete culverts)	0.015

B.10 The waterway opening area at each bridge was modified to remove the cross sectional area of piers. At some locations, the upstream and downstream cross sections were more of a flow constriction than at the bridge itself. In these instances, the waterway area was appropriately reduced to reflect the smaller upstream or downstream cross sectional area to ensure numerical stability in the model.

B.11 The bridge across Moore River No. 1 channel at Chainage 2600 m was not included in the hydraulic model. Preliminary hydraulic modelling indicated that this bridge does not redistribute flows or increase flood levels in the town.

B4.3 Afflux at Weirs

B.12 The road surface above the bridges were modelled as broad crested weirs. The standard formulations for flow over a broad crested weir are established automatically by the model on the basis of weir geometry and the user specified head loss and calibration factors. These formulations assume a hydrostatic pressure distribution on the weir crests. Different algorithms are used for drowned flow and free overflow, with automatic switching between the two modes of flow. The head loss factors adopted for the Moora MIKE 11 model for weirs are shown in Table B4.3.

Table B4.3 Adopted Head Loss Factors for Weirs, Moora MIKE 11 Model

Loss Type	Value
Inflow Loss Factor	0.5
Outflow Loss Factor	1.0
Free Overflow Loss Factor	1.0

B5 COONDEROO FLOODPLAIN STORAGE

B.13 The residents who live near the confluence of Moore River No. 3 channel and Coonderoo River stated that March 1999 floodwaters from the Moore River flowed upstream along the Coonderoo River into the Coonderoo Lake system located immediately to the north-west of Moora township. After about 6 hours, it 'turned around' and flowed to the south. Hydraulic modelling of both the March and May 1999 flood events support this observation.

5.14 The digital half metre contour data was used to estimate the stage-storage capacity relationship of the Coonderoo Lake system. In the MIKE 11 model, it was assumed that this additional floodplain storage occurs upstream of Ferguson Road at cross section 440, (Coonderoo Branch) (See Figure B2.1). Table B5.1 shows the stage-storage relationship adopted for the Coonderoo Lake system.

Table B5.1 Stage-Storage Relationship Adopted for the Coonderoo Lakes, Cross Section 440, Moora MIKE 11 Model

Stage (m AHD)	Storage Area (ha)
200.86	0
201.23	156
201.50	312
201.75	566
202.00	820
202.61	1,600
203.05	2,215
204.00	3,296

B6 CALIBRATION METHODOLOGY

B.15 The Moora MIKE 11 hydraulic model was calibrated against recorded discharge and water level data for the March, May, July and August 1999 flood events. Recorded discharge hydrographs were available only for the July and August flood events at the Moora Caravan Park (GS 617 010). Both these events were small and the flows were contained within the main channels of Moore River No. 1 and Moore River No. 2. For the March and May 1999 flood events, only partial stage hydrographs recorded in the town and some peak flood level data across the floodplain were available. It is also noted that the railway embankment between Moore River No. 1 and Moore River No. 2 breached during the March 1999 flood event. The railway embankment remained intact during the other calibration events.

B.16 As explained in Section A3.3 of Appendix A, difficulties were encountered with the calibration of the URBS hydrologic model against the May and August 1999 events. As a consequence, the calibration of the hydraulic model focused on the March and July 1999 flood events.

B.17 Due to the absence of discharge data for the overbank flow events (March and May), calibration of the Moora MIKE 11 model was undertaken in three stages. The first stage involved calibrating the model against the recorded stage and discharge hydrograph for the July 1999 event to establish the main channel roughness (i.e. Manning's 'n') values. The second stage involved an iterative joint calibration of the URBS hydrologic model and the MIKE 11 hydraulic model against the partial stage hydrographs recorded for the March and May 1999 events. The third stage involved the fine tuning the overall calibration by adjusting roughness parameters to obtain the best possible fit between the recorded and predicted flood levels across the floodplain.

B.18 Table B6.1 shows the Manning's n values adopted in the Moora MIKE 11 hydraulic model. The calibration involved assigning representative Manning's n values to the different segments of the model. For instance, a single Manning's n value (0.05) was adopted for the entire main channels of the Moore River No's 1 and 2 branches, Coonderoo River and Yadgena Brook. Representative Manning's n values were adopted for the town area and floodplain areas to reflect the density of buildings and vegetation in respective areas. Minor fine tuning of the roughness factors were undertaken where required to improve model calibration. The adopted Manning's n values are consistent with hydraulic characteristics of the Moore River and its floodplain areas.

Table B6.1 Adopted Manning's n Roughness Values, Moora MIKE 11 Hydraulic Model

Area	Manning's n
Main Channel	0.05
High Density Urban Area	0.15
Low Density Urban Area	0.1
Heavily Vegetated Floodplain	0.075
Low Vegetated Floodplain	0.065

B7 CALIBRATION RESULTS - JULY 1999 EVENT

B7.1 Overview

B.19 Calibration results indicated that all flow was not contained within the Moore River No. 1 channel during the July 1999 event. The results show that up to 2 m^3/s was carried by the Moore River No. 2 channel and a further 0.5 m^3/s was carried by the Moore River No. 4 channel. As a result, the gauging station at the Moora Caravan Park (617010) did not account for the total flow in the Moore River in July 1999. It was assumed that the rating at 617010 did not include these bypass flows.

B.20 The MIKE 11 model was used to obtain a suitable match between the recorded and predicted discharge hydrographs in the Moore River No. 1 branch at 617010. Figure B7.1 shows a comparison between the recorded and predicted discharge hydrographs at 617010. Note that flows that bypassed the Moore River No. 1 branch at 617010 were not included in the comparison. The URBS predicted discharge hydrograph at 617010, which shows that total Moore River flows (including bypass flows), is also shown in Figure B7.1. The difference between the URBS and the MIKE 11 discharge hydrographs represents the discharge that was diverted into the Moore River No. 2 and No. 4 branches.

B.21 The recorded and predicted (MIKE 11) discharge hydrographs compare well for the July 1999 event. The shape and timing of the first two peaks are well represented. The predicted third (and the largest) peak is some 2 m^3/s greater than the recorded peak discharge and occurs about 1 hour earlier than the recorded event. In view of the small size of the event and the large catchment size, the model calibration of this event is considered satisfactory.

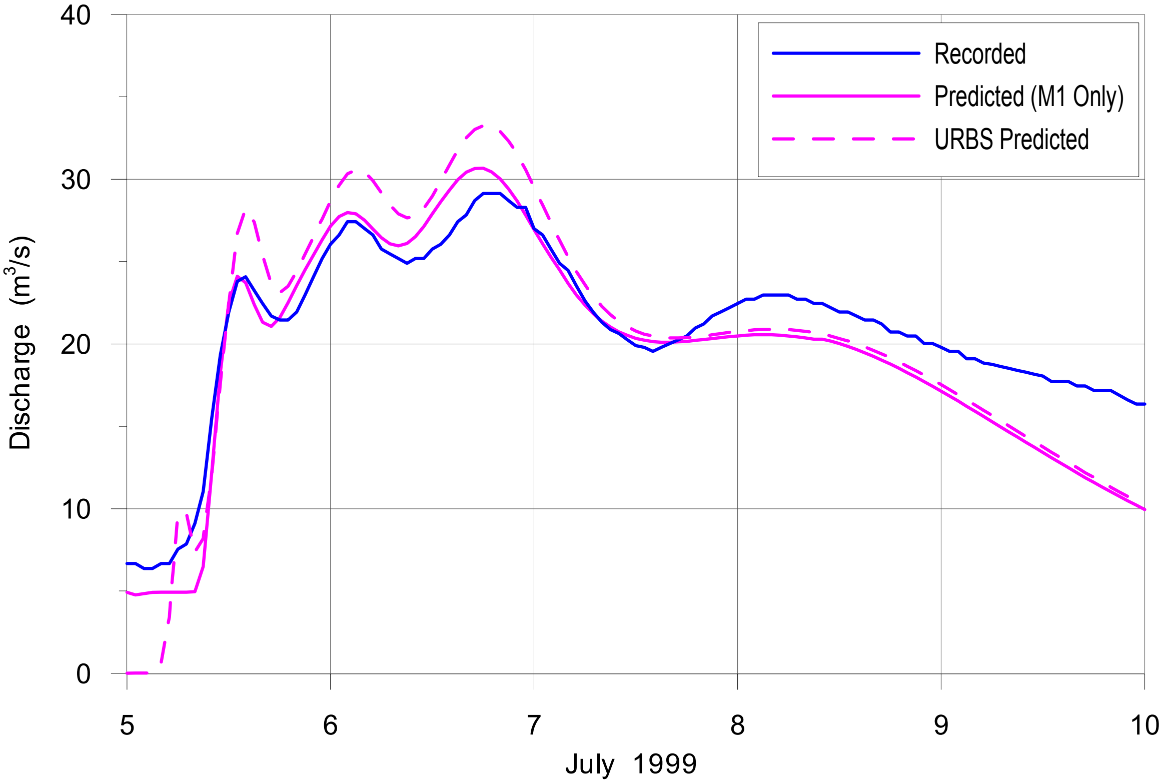

Figure B7.1 Recorded and Predicted Discharge Hydrographs at the Moora Caravan Park (617010)

B7.2 Stage Hydrographs

B.22 Figure B7.2 shows a comparison between the recorded and predicted stage hydrographs at Moora Caravan Park (GS 617010) for the July 1999 event. The predicted stage hydrograph, with a main channel Manning's 'n' of 0.046 is in close agreement with the recorded stage hydrograph. The predicted peak flood level is only 0.03 m higher than the recorded value.

B.23 Figure B7.2 also shows the predicted stage hydrograph for a main channel Manning's 'n' value of 0.05. For this case, predicted peak flood levels are about 0.1 m higher than the recorded levels. Although it was intended to adopt the main channel Manning's n estimated from the July event for all subsequent events, a Manning's 'n' value of 0.05 was found to give better calibration results for the March event. There are two possible reasons for this:

- The predicted peak discharge in the Moore River No 1 channel for the July 1999 event is 2 m^3/s greater than the recorded peak discharge.

- The MIKE 11 model of the Moore River No. 1 branch includes some 'fringe' areas adjacent to the main channel. These fringe areas have more vegetation than the bed and thus will have a higher roughness. The July 1999 peak flood level is some 3 m lower than the March 1999 flood and would not have been significantly affected by the vegetated fringe areas.

B.24 Data available is insufficient to estimate different Manning's n values for the main channel and fringe areas. Because the focus of the flood study is to assess the behaviour of large flood events, a global main channel and fringe area Manning's 'n' value of 0.05 was adopted for all flood events.

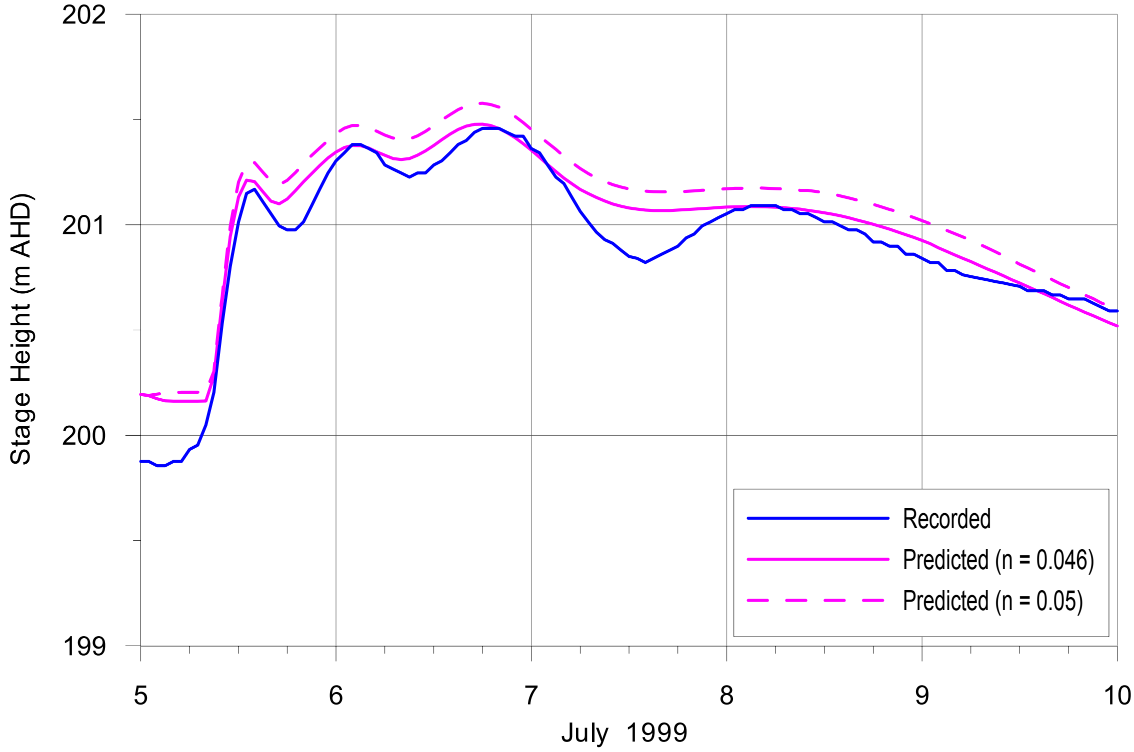

Figure B7.2 Recorded and Predicted Stage Hydrographs at Moore River at Moora Caravan Park, GS 617010

B8 CALIBRATION RESULTS - MARCH 1999 EVENT

B8.1 Overview

B.25 During the March 1999 flood event, properties in Moora were inundated by floodwater from both Moore River and Yadgena Brook. However, the data available to calibrate the Moore River flood event includes only a portion of a stage hydrograph recorded at Melbourne Street and some peak flood levels measured after the event. Only anecdotal data is available for the Yadgena Brook flood event.

B8.2 Moore River Stage Hydrograph

B.26 Although the available data is limited, a good calibration of the hydrologic and hydraulic models was achieved for the March 1999 flood event of the Moore River. Figure B8.1 shows the comparison between the recorded and predicted stage hydrographs at Melbourne Street near the Moore River No. 1 branch. The recorded levels were obtained from the resident at Lot 159 Melbourne Street who recorded the variation in the flood level above the floor level of his house over a period of time. (The floor level of the house was surveyed and tied to AHD in December 1999).

B8.3 Peak Flood Levels

B.27 Peak flood levels recorded throughout Moora town area were used to fine tune the hydraulic model calibration and ensure the distribution of predicted flows along each of the main branches of the Moore River was accurately represented. Table B8.1 shows a comparison of recorded and predicted peak flood levels throughout the town during the March 1999 flood event. Figures B8.2, B8.3, B8.4 and B8.5 are longitudinal water surface profiles showing a comparison between recorded and predicted peak flood levels along Moore River No. 1, No. 2, No 3 (including the downstream end of Coonderoo River) and No. 4 (including Clinch Street) branches.

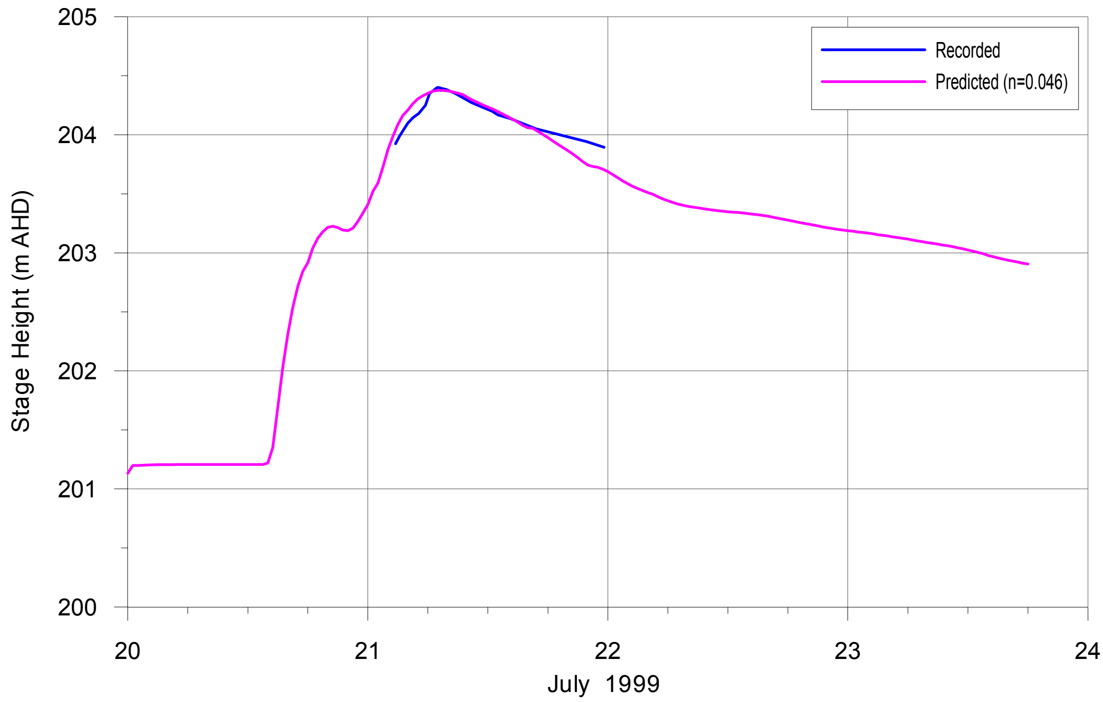

Figure B8.1 Recorded and Predicted Stage Hydrographs Moore River at Lot 159 Melbourne Street, March 1999 Flood Event

Table B8.1 Recorded and Predicted Peak Flood Levels in Moora, March 1999 Flood Event

Location	Peak Flood Level (m AHD)		Difference (Rec-Pred) (m)
	Recorded	Predicted	
Lot 238 Barber Street	204.75	204.69	-0.06
Lot 230 Ranfurley Street	204.43	204.52	+0.09
Cnr Berkshire Valley Road & Ranfurley Street	204.59	204.41	-0.18
Cnr Melbourne & Cameron Streets	204.34	204.29	-0.05
Cnr Melbourne & Woolawa Streets	204.42	204.54	+0.12
Cnr Berkshire Valley Road & Melbourne Street	204.33	204.27	-0.06
Cnr Gardiner & Cameron Streets	204.29	204.27	-0.02
Moore River No.2 Branch @ Tootra Street	204.54	204.48	-0.06
Cnr Roberts & Clinch Streets	203.71	203.70	-0.01
Cnr Roberts & Moore Streets	203.71	203.68	-0.03
Cnr Roberts & Beasley Streets	203.55	203.63	+0.08
Cnr Roberts & Clarke Streets	203.69	203.57	-0.12
Cnr Kintore & Drummond Streets	202.38	202.5	+0.12
Cnr Kintore & Dandaragan Streets	203.31	202.98	-0.03
Cnr Lefroy & Dandaragan Streets	203.34	203.23	-0.11
Cnr Keane & Stafford Streets	203.53	203.29	-0.24
Cnr Keane Street & Riley Rd	203.48	203.38	-0.1
Cnr Long & Drummond Streets	201.95	202.06	+0.11
Cnr Long & Dandaragan Streets	202.7	202.79	+0.09
Cnr Long & Beasley Streets	203.03	202.84	-0.19
Cnr Long & Clarke Streets	202.89	202.84	-0.05
Cnr Long & Carrick streets	203.02	202.5	-0.52
Cnr Bishops & Stafford Streets	202.39	202.32	-0.07
Lot 198 Riley Road	202.05	202.18	+0.13
Cnr Ferguson & Clarke Streets	201.8	201.88	+0.08
Cnr Ferguson & Stafford Streets	201.9	201.84	-0.06

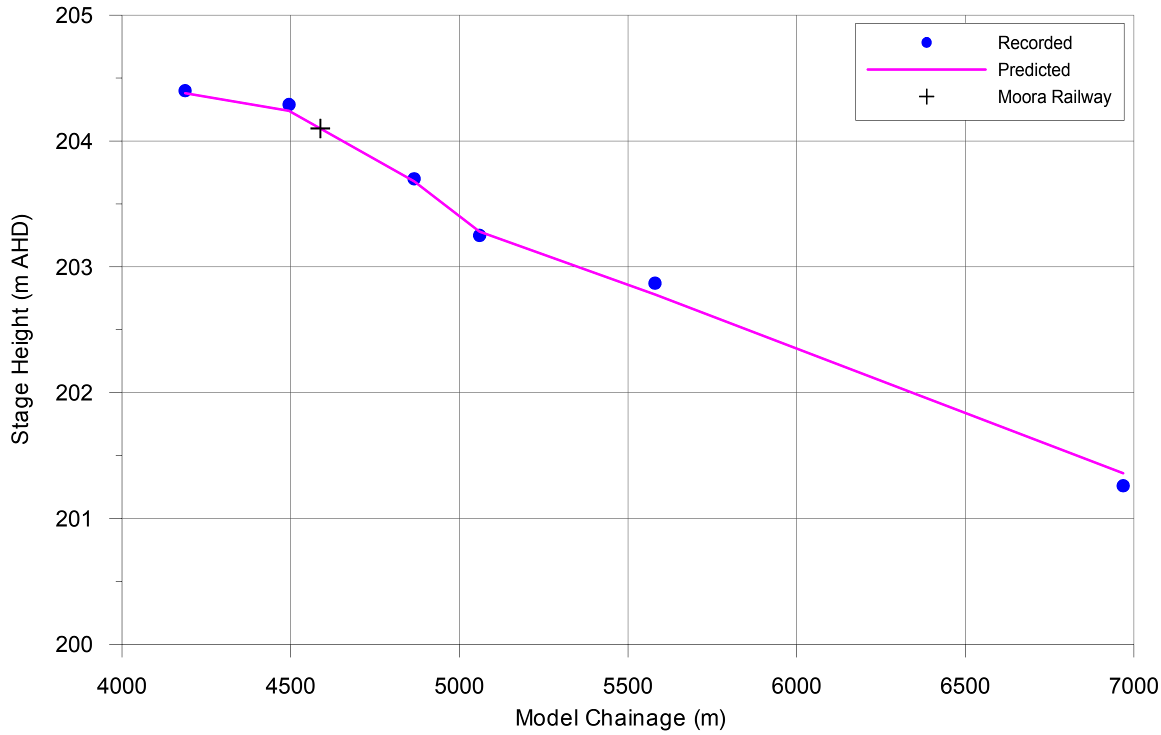

Figure B8.2 Longitudinal Peak Water Surface Flood Profile, Moore River No. 1 Branch March 1999 Flood

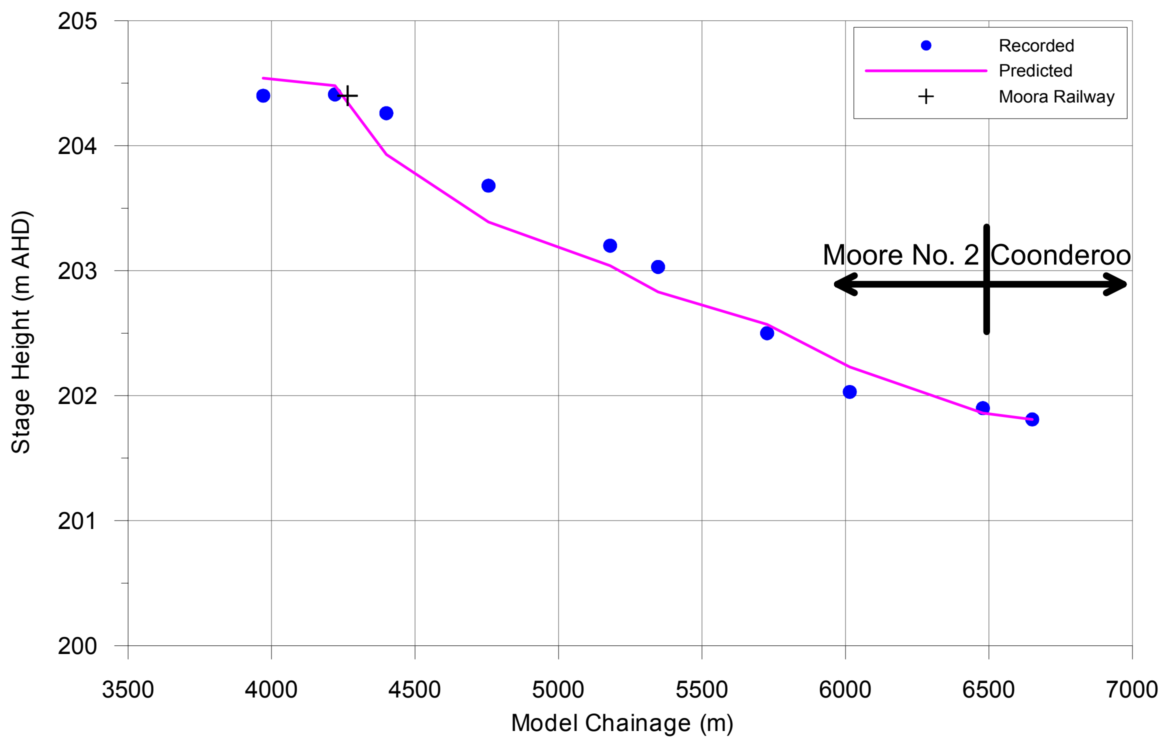

Figure B8.3 Longitudinal Peak Water Surface Flood Profile, Moore River No. 2 Branch, March 1999 Flood

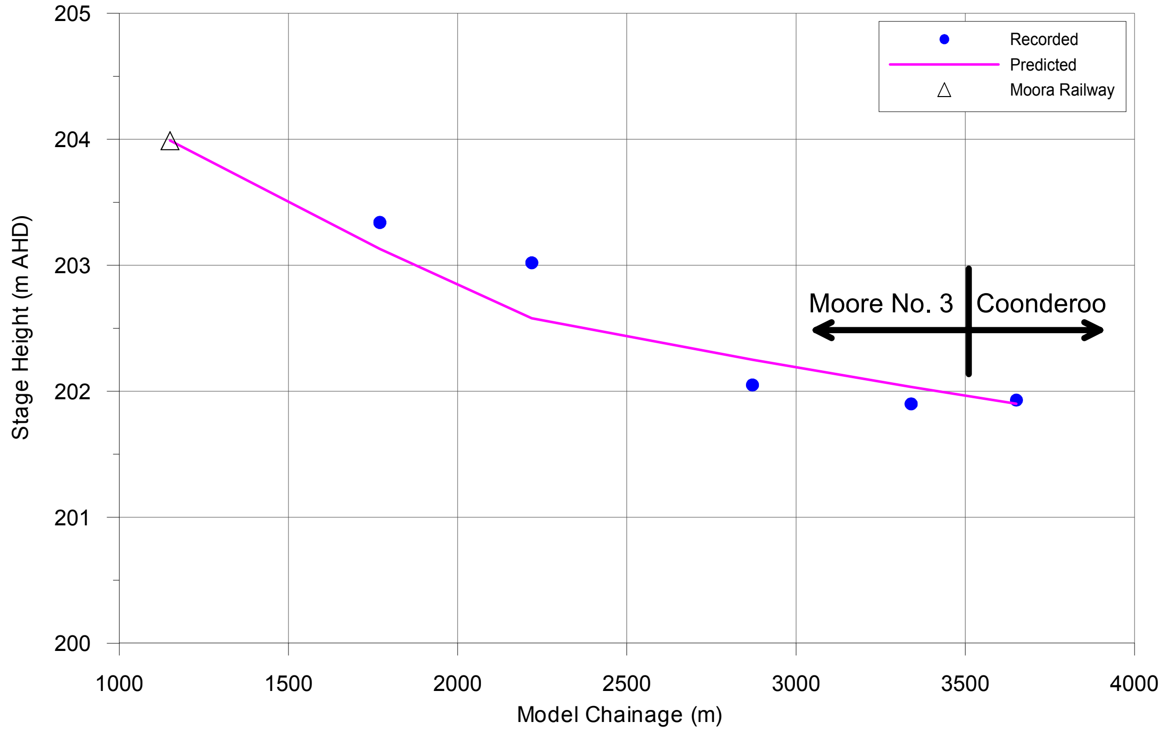

Figure B8.4 Longitudinal Peak Water Surface Profile,Moore River No. 3 Branch including Coonderoo River March 1999 Flood

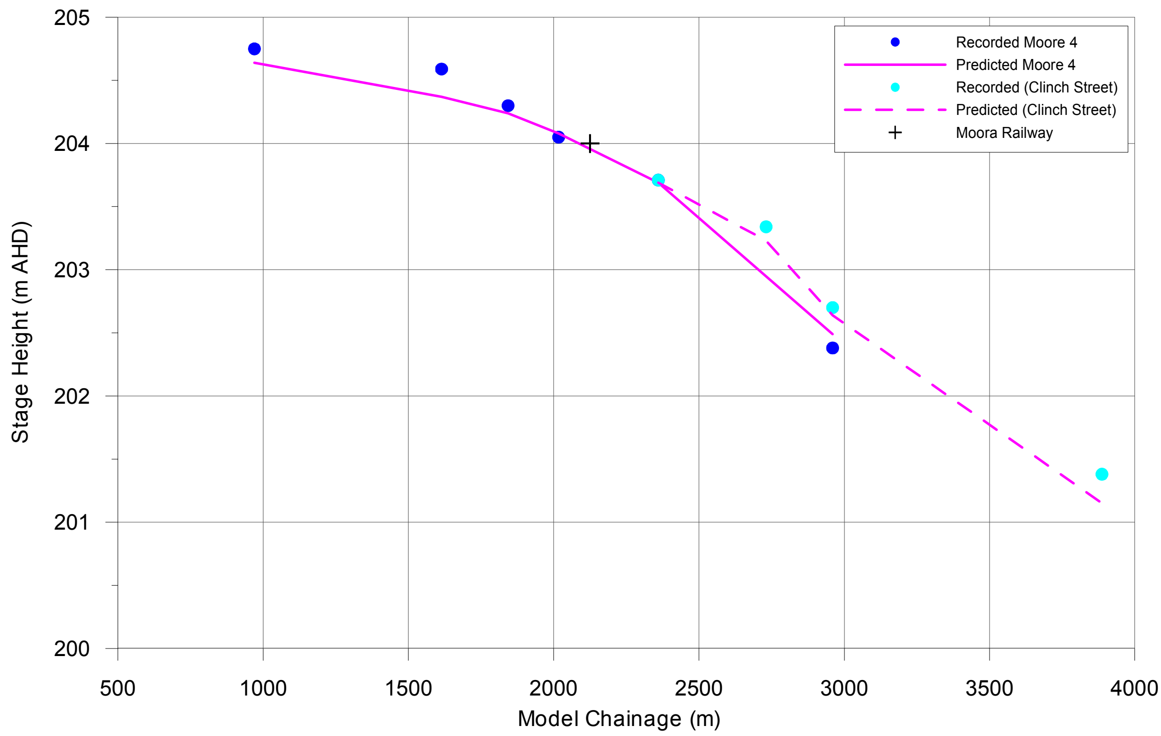

Figure B8.5 Longitudinal Peak Water Surface Profile Moore River No. 4 and Clinch Street Branches, March 1999 Flood

B.28 The predicted peak flood levels along Moore River No. 1, No 2 and No 4 branches are generally within 0.2 m of the surveyed peak flood levels provided by WRC. An exception to this occurs downstream of the railway line along the Moore No. 2 branch. Predicted peak flood levels are some 0.3 m lower than the surveyed levels at this location. This is probably due to of the railway ballast breaching between Moore No.1 and 2 branches early on Sunday morning 21st March. The breach of the railway was not included in the model.

B.29 Along Moore River No 3 branch, between the Railway Line and Carrick Street, the predicted peak flood levels are up to 0.5 m lower than the surveyed peak flood levels. Sensitivity analysis with the model showed that increasing peak flows in the Moore River No. 3 branch upstream of the railway had little affect on flood levels downstream of the railway because the additional flows were diverted to the south towards the Moore No. 2 channel. It is possible that local floodwaters from an unknown flow path along Midlands Road or from the Coonderoo River may have contributed to flooding along the Moore River No. 3 channel.

B.30 Peak flood levels along the Coonderoo River are shown in the lower sections of Figures B8.3 and B8.4. Predicted peak flood levels along the Coonderoo River are within 0.1 m of the surveyed levels. The modelling results indicate that flows of up to 100 m^3/s flowed into the Coonderoo Lakes system from the Moore River during the March flood. Without this loss of flow, peak flood levels in this area would have been up to 0.5 m higher than the surveyed levels. Peak outflows from the Lakes cannot be accurately modelled without a recorded stage hydrograph for this area to calibrate the model. Nevertheless, it appears that the peak outflows from the lake system would have been less than 50 m^3/s. As a result, the modelling indicates that the Coonderoo River lowered peak flood levels in the Moora town area.

B8.4 Yadgena Brook

B.31 Table B8.5 shows the estimated March 1999 peak flood levels along Yadgena Brook and in Moora at Hamilton Road. The March 1999 peak discharge estimated for Yadgena Brook upstream of Walebing Road is 95 m^3/s. Of this discharge, approximately 15 m^3/s was found to flow along the Walebing Road. According to the hydraulic model results, there was no flow along the Mogumber Road.

B.32 The hydraulic model predicted that floodwater from Yadgena Brook arrived at Hamilton Road at about 2000 hours. This is in agreement with the reported flood behaviour.

B.33 Sensitivity analyses were undertaken by increasing the Yadgena Brook peak discharge to determine the discharge required for flood waters to flow along the Mogumber Road as the anecdotal evidence had suggested. The model results indicated that the constriction at the Walebing Road crossing distributes any additional flows coming down Yadgena Brook along Walebing Road rather than towards Mogumber Road. This suggests that Yadgena Brook floodwaters did not flow along Mogumber Road during the March 1999 flood event. The floodwaters that appeared to have flowed along the Mogumber Road may have been due to local runoff, a temporary blockage at the Mogumber Road bridge or from an unknown flow path from Yadgena Brook between the Walebing and Mogumber Roads.

Table B8.2 Estimated Peak Flood Levels along Yadgena Brook and at Hamilton Road, March 1999

Location	Peak Flood Level (m AHD)
Yadgena Brook U/S Walebing Road	207.36
Yadgena Brook D/S Walebing Road	207.10 [a]
Yadgena Brook U/S Mogumber Road	203.57
Moora at Hamilton Road	204.19

[a] Interpolated Level

B8.5 Extent of Flooding Predicted

B.34 Figure B8.6 shows the predicted extent of flooding for the March 1999 flood event. The predicted extent of flooding is generally consistent with the reported extent of flooding.

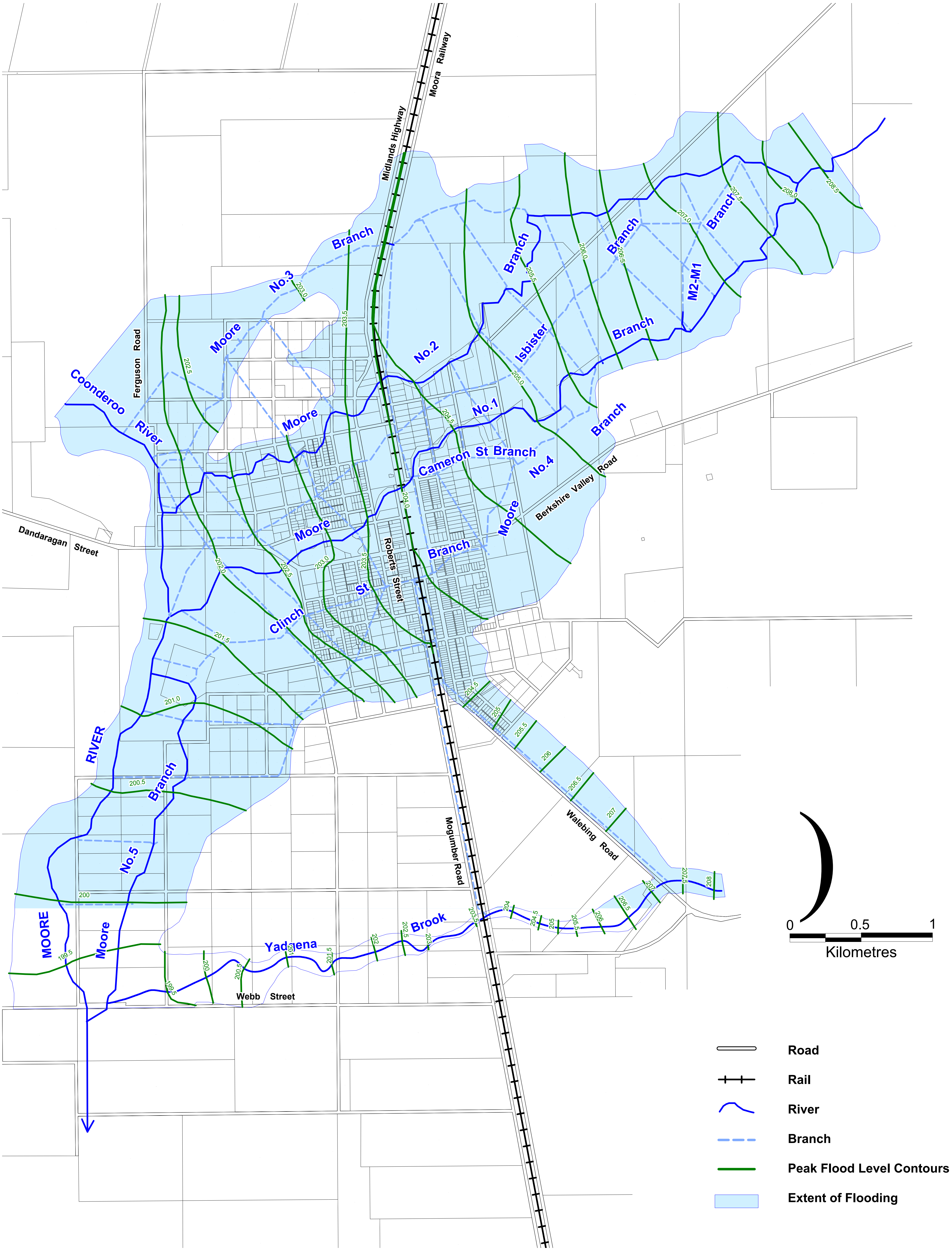

Figure B8.6 Predicted Extent of Flooding for the March 1999 Flood event, Moore Mike 11 Model

B9 CALIBRATION RESULTS - MAY 1999 EVENT

B9.1 Stage Hydrographs

B.35 Figure B9.1 shows a comparison between the recorded and predicted stage hydrographs of the Moore River No. 1 branch at Gardiner/Tootra Street for the May 1999 event. The recorded levels were provided by Rob Lennox of the Water Corporation who measured the rising water levels at gauge boards and at the Cameron/Tootra Street sign post. The levels recorded at the gauge boards and sign post were surveyed and tied into AHD in December 1999. Figure B9.1 also shows an estimate of the peak flood level at this location, which has been determined from recorded flood levels nearby. The timing of the peak was reported by WRC to be about at 1600 hours on the 28th May 1999.

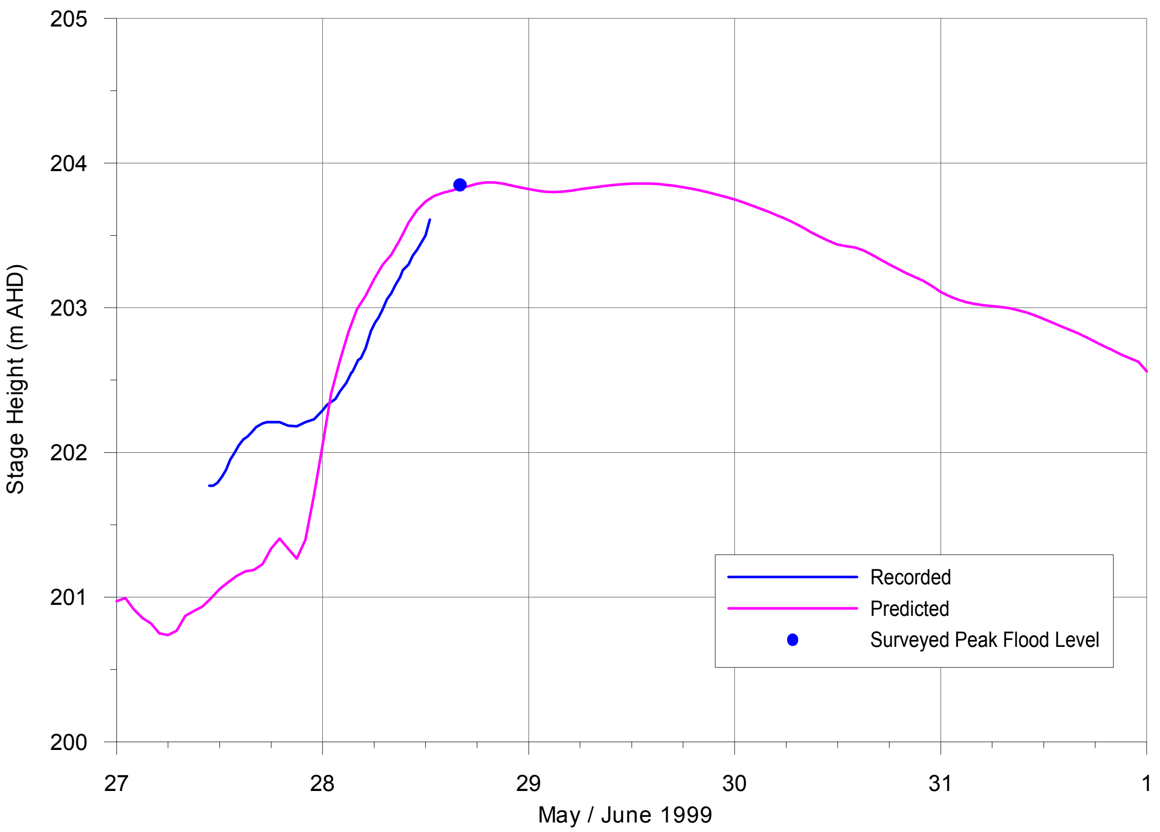

Figure B9.1 Recorded and Predicted Stage Hydrographs at Moore River at Gardiner/Tootra Street, May 1999.

B.36 The rising limb of the recorded and predicted stage hydrographs are generally in close agreement. The predicted rise in water level at Gardiner Street is about 2 hours earlier than what was recorded at Gardiner Street. However, the predicted time of the peak is about 3 hours later than the actual time of the peak provided by WRC. It is noted that the predicted water level at Gardiner Street varies only by 0.1 m from the peak level for more than 30 hours. Thus the true timing of the peak at Moora would have been difficult to determine. No water level data was recorded on the falling limb of the May 1999 flood to support this finding. Not withstanding this, the calibrated May 1999 flood event involved a trade off between the timing of the rising limb and the estimated time of the peak.

B.37 The difference between recorded and predicted peak flood level at Gardiner/Tootra Street is less than 0.1 m. Note that the record level was estimated from surveyed peak flood levels nearby.

B9.2 Comparison of Peak Flood Levels

B.38 Table B9.1 shows a comparison of recorded and predicted peak flood levels at various locations in Moora for the May 1999 flood event. The predicted flood levels are generally within 0.1 m of the surveyed flood marks for the May event. Exceptions to this occur along the Coonderoo River near Ferguson Road where predicted peak flood levels vary by !0.2 m from the recorded levels. It appears that less floodwater flowed into the Coonderoo Lakes system from the Moore River and more floodwater flowed out of the lakes during this event than what has been predicted by the model. Noting the lack of available inflow and outflow data from the Coonderoo Lakes system, the variation between the recorded and predicted peak flood levels in this area is considered acceptable.

Table B9.1 Recorded and Predicted Peak Flood Levels in Moora, May 1999 Flood Event

Location	Peak Flood Level (m AHD)		Difference (Pred-Rec) (m)
	Recorded	Predicted	
Lot 238 Barber Street	204.28	204.42	+0.14
Lot 230 Ranfurley Street	204.39	204.10	-0.29
Cnr Melbourne & Cameron Streets	203.96	203.91	-0.05
Cnr Berkshire Valley Road & Melbourne Street	203.87	204.05	+0.18
Cnr Lefroy & Dandaragan Streets	202.93	203.00	+0.07
Cnr Kintore & Dandaragan Streets	202.85	202.78	-0.07
Cnr Kintore & Drummond Streets	202.38	202.29	-0.09
Cnr Kintore and King Streets	202.86	203.08	+0.22
Cnr Keane & Stafford Streets	203.09	203.05	-0.04
Cnr Keane & Clarke Streets	203.05	203.11	-0.06
Cnr Roberts & Clarke Streets	203.43	203.32	-0.11
Cnr Stafford & McPherson Streets	203.06	202.84	-0.22
Cnr Long & Moore Streets	202.69	202.65	-0.04
Cnr Ferguson & Riley Roads	201.89	202.70	-0.19
Cnr Ferguson & Clarke Streets	201.50	201.70	+0.20
Cnr Glasfurd & Stafford Streets	201.87	201.85	-0.02

B.39 Other minor variations from the recorded peak flood levels occur along the Cameron Street branch (near Barber and Ranfurley Streets) and at the corner of Stafford and McPherson Streets. The model was unable to reproduce the recorded flood levels at these locations. Based on recorded peak flood levels nearby, the recorded peak flood levels at these locations appear to be unreliable. Also, the recorded peak flood level at Ranfurley Street is higher than the level recorded upstream at Barber Street, which appears to support the above finding.

B9.3 Extent of Flooding

B.40 Figure B9.2 shows the predicted extent of flooding for the May 1999 flood event. The predicted extent of flooding appears to be consistent with the reported flooding behaviour.

B10 CALIBRATION RESULTS - AUGUST 1999 EVENT

B.39 Only Yadgena Brook flows were calibrated for the August 1999 flood event. As explained in Section 4.3 of Appendix A, significant problems were encountered with the calibration of the August event. Using the available rainfall data, hydrologic modelling indicated that Yadgena Brook peak discharge

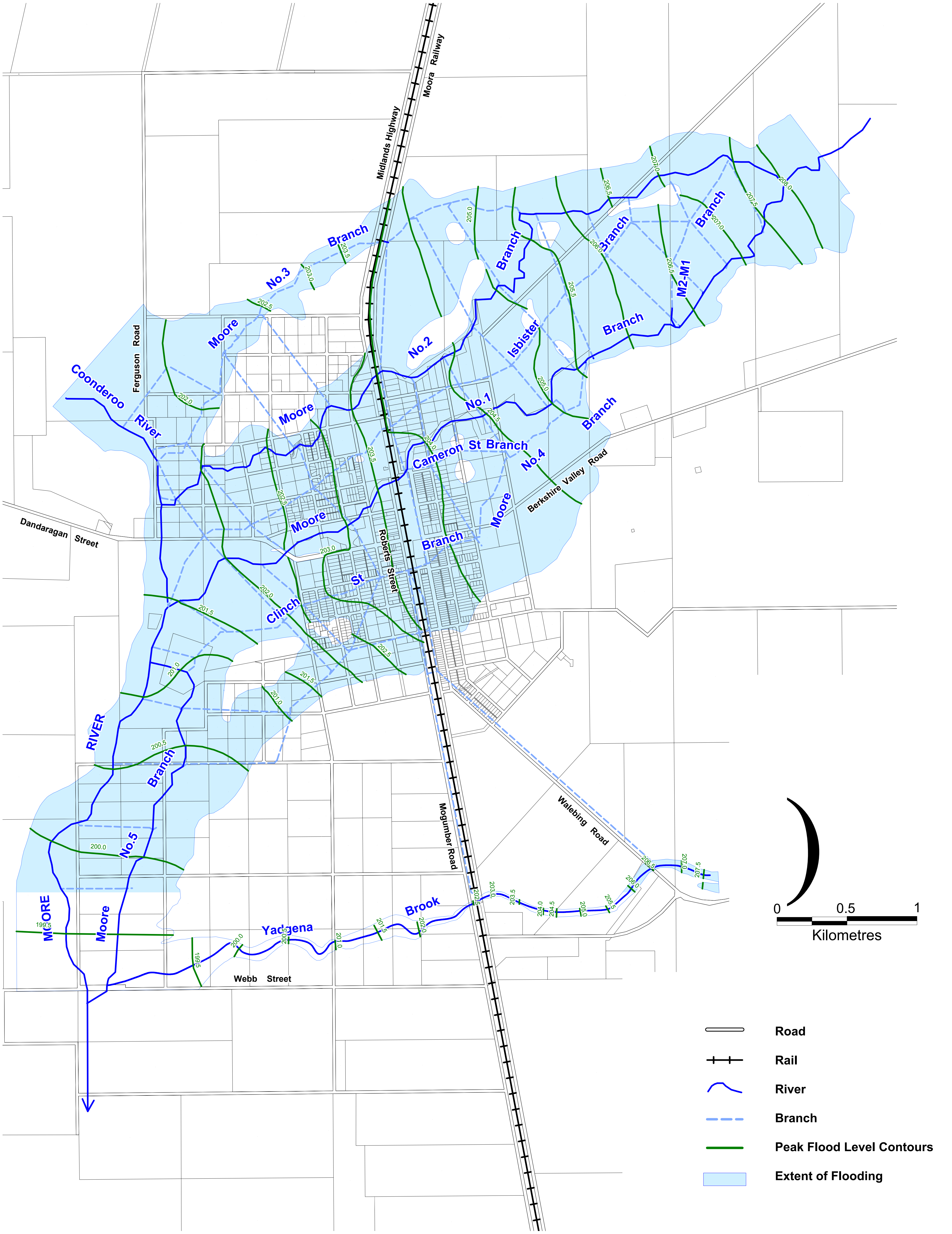

Figure B9.2 Predicted Extent of Flooding for the May 1999 Flood Event, Moore Mike 11 Model

at Walebing Road was only about 24 m^3/s. Sensitivity testing with the hydraulic model indicated that a peak discharge of about 132 m^3/s is required to obtain the recorded peak flood levels i.e. a discharge of more than 5 times the value originally predicted by the hydrologic model. Thus, using anecdotal data provided by WRC, rainfalls used in the hydrologic model were increased to provide the predicted estimated discharge in Yadgena Brook (See Section A33, Appendix A)

B.40 Table B10.1 shows a comparison between the recorded and predicted peak flood levels. An acceptable calibration was achieved for the adopted peak discharge of 132 m^3/s. Note that no cross sectional data was available at the location of the recorded flood level downstream of Walebing Road. Thus, the predicted flood level at that location was estimated by interpolating between the upstream and downstream cross sections.

Table B10.1 Comparison Between Recorded and Predicted Flood Levels for the August 1999 Flood Event

Location	Peak Flood Level (mAHD)	
	Recorded	Predicted
Yadgena Brook, U/S Walebing Road	207.52	207.54
Yadgena Brook, D/S Walebing Road	207.41	207.20[a]
Yadgena Brook, U/S Mogumber Road	203.89	203.76
Hamilton Road	204.02	204.02

[a] Interpolated Level

B11 CONCLUSION

B.41 The calibrated Moora MIKE 11 hydraulic model satisfactorily reproduces recorded flood level behaviour along the modelled reaches of the Moore River, Coonderoo River and Yadgena Brook. The recorded and predicted flood levels compare well for the March, May, July and August flood events.

B12 REFERENCES

DHI (1992) 'MIKE-11 - Technical Reference Manual', Danish Hydraulic Institute, 1992.

C Appendix DESIGN FLOOD DISCHARGE ESTIMATION

C1 INTRODUCTION

C.01 This Appendix describes the estimation of design discharges in the Moore River Catchment. The set up and calibration of the hydrologic model (URBS) for the estimation of design discharges are described in Appendix A and are therefore not repeated herein.

C.02 This Appendix is divided into seven sections:

- Section 2 provides an overview of the methodology used for the estimation of design flood discharges.
- Section 3 presents a flood frequency analysis of peak annual discharges at Quinn's Ford
- Section 4 describes the continuous rainfall-runoff simulation modelling undertaken with the AWBM model.
- Section 5 describes the design flood estimation process. The predicted discharges for a range of storm severities up to the PMP are also presented in this section.
- Section 6 assesses the severity of the March and May 1999 Flood Events.
- Section 7 is a list of references.

C2 METHODOLOGY ADOPTED FOR ESTIMATION OF DESIGN FLOOD DISCHARGES

C2.1 Overview

C.03 Estimation of design flood discharges for large catchments such as the Moore River Catchment is generally undertaken using either:

- Flood frequency analysis, or
- A rainfall-based technique such as runoff-routing.

C.04 Flood frequency analysis has the advantage of incorporating variability in all catchment processes. However, a large body of streamflow data is required to obtain confident estimates of design discharges for extreme events. Furthermore, the results cannot be transposed to other locations within the catchment. In contrast, runoff-routing models can provide design discharge estimates at different locations within a catchment, rather than at a single stream gauge location.

C.05 One of the principal difficulties in estimating design flood discharges using runoff-routing models is the estimation of appropriate rainfall losses. In a semi-arid region such as the Moore River catchment, actual rainfall losses can vary significantly between rainfall events, depending on antecedent conditions. Flavell & Belstead (1986) have estimated average losses for twelve regions in Western Australia for design events of 2 to 50 years average recurrence interval (ARI). However, the appropriateness of adopting these values for design flood estimation in the Moore River Catchment, particularly for floods larger than 50 years ARI, remains uncertain.

C.06 Along the Moore River, continuous streamflow data (31 year period of record) is available only at Quinn's Ford (GS 617001), some 70 km downstream of Moora. Only limited data for one or two flood events since March 1999 is available in Moora itself. To accurately simulate flooding behaviour in the town, design discharge estimates are required along the Moore River and a number of tributaries in and around Moora. In addition, hydrologic behaviour of the Moore River catchment is quite complex and highly variable - certain portions of the catchment rarely produce any runoff.

C2.2 Adopted Methodology

C.07 Due to the limited data availability for flood frequency analysis and the difficulty in estimating appropriate rainfall losses for runoff-routing, the adopted methodology for the estimation of design flood discharges along the Moore River is based on a combination of results from:

- A flood frequency analysis of historical peak annual discharges at Quinns Ford,
- A continuous rainfall-runoff simulation model (AWBM) of the Moore River Catchment, and
- A calibrated URBS runoff-routing model,

C.08 The details and results of each of the three design flood estimation techniques listed above are discussed in the following sections.

C.09 Design rainfalls were estimated for the study area for storm durations of up to 100 year ARI, on the basis of data provided in Australian Rainfall and Runoff (IEAust, 1998). The probable maximum precipitation (PMP) estimates for the Moore River catchment were provided by the WRC.

C.10 Design flood discharges were estimated for 2, 5, 10, 20, 50 and 100 year ARI events and for the PMP. The calibrated URBS model (see Appendix A) was used to estimate the design discharges at locations of interest in the Moore River catchment. Calibrated URBS model catchment parameters and March 1999 event continuous loss rates were adopted as the basis for design events. March 1999 event initial loss rates were adjusted to produce design discharges at Quinn's Ford for the whole range of design events up to 100 years ARI that are consistent with estimates from the flood frequency analysis and the AWBM model. One adjustment factor was applied to all values of initial loss for each design ARI, however the adopted factors varied between different ARI's. The exception to this was the Coonderoo catchment where initial losses were not factored but fixed at March 1999 calibrated values.

C3 FLOOD FREQUENCY ANALYSIS

C3.1 Flood Frequency Analysis

C.11 The methodology given in Australian Rainfall and Runoff (IEAust, 1998) was used to fit log-normal and log-Pearson Type III distributions to the annual series of recorded peak flood discharges at Quinns Ford gauging station. Figure C3.1 shows the cumulative distribution of the peak flood discharges together with the fitted distributions. Table C3.1 shows salient statistics of the annual series. The log - Pearson Type III distribution appears to best fit the recorded peak discharges. Estimates of design flood discharges for a range of AR's based on the log Pearson type III distribution are given in Table C3.2.

C.12 Based on the flood frequency curves , March and May 1999 flood events had ARI's of 50 and 14 years respectively at Quinns Ford. Note that the hydrologic model calibration revealed that the recorded peak discharge at Quinn's Ford for the May 1999 event may be incorrect. Therefore, the ARI estimate of the May 1999 event should be treated with caution (see Appendix A).

Table C3.1 Salient Statistics of Annual Flood Peaks, Moore River at Quinn's Ford, 1969 - 1999

Statistic	Distribution	
	Arithmetic	Log_{10}
Mean (m^3/s)	77.2	1.6
Standard Deviation (m^3/s)	98.2	0.466
Coefficient of Skew	2.59	0.138
No of Values	31	31

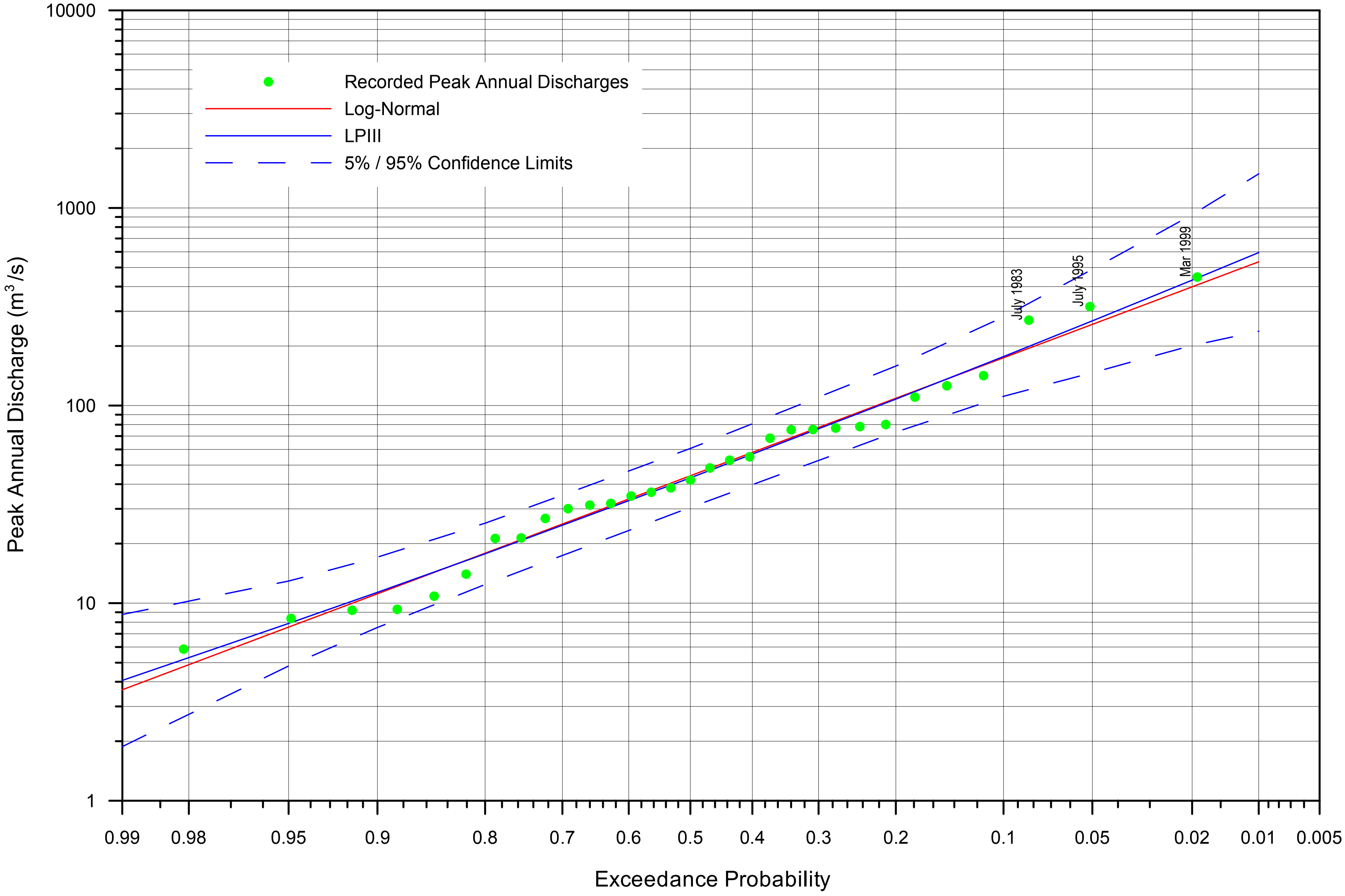

Figure C3.1 Distribution of Peak Annual Flows, Moore River at Quinn's Ford

Table C3.2 Design Discharge Estimates, Moore River at Quinn's Ford

ARI (years)	Design Discharge (m^3/s)
2	43
5	108
10	177
20	268
50	432
100	596

C3.2 Annual Rainfall Trends

C.13 The cumulative sum technique was used to analyse long-term trends in catchment annual rainfall. The cumulative sum (S_i) can be defined as:

$$S_i = \sum_{i=1}^{i} (x_i - \bar{x}) \qquad (C.1)$$

where x_i and $\bar{x}$ are the annual and average annual rainfall respectively.

C.14 The distribution reveals runs of observations greater than the long-term mean with a positive slope, and less than the long-term mean with a negative slope. Persistent positive or negative slopes can be used to detect intermediate trends in annual rainfall totals. The actual ordinate values are not relevant, it is the slope which is important.

C.15 The cumulative sum distribution of annual rainfalls at Berkshire Valley 2 (008008) is shown in Figure C3.2. It can be seen that the period of streamflow record at Quinn's Ford (1969 to present) coincides with a period of below average rainfall. This indicates that flood frequency analysis based on this period of record may under-estimate design discharges.

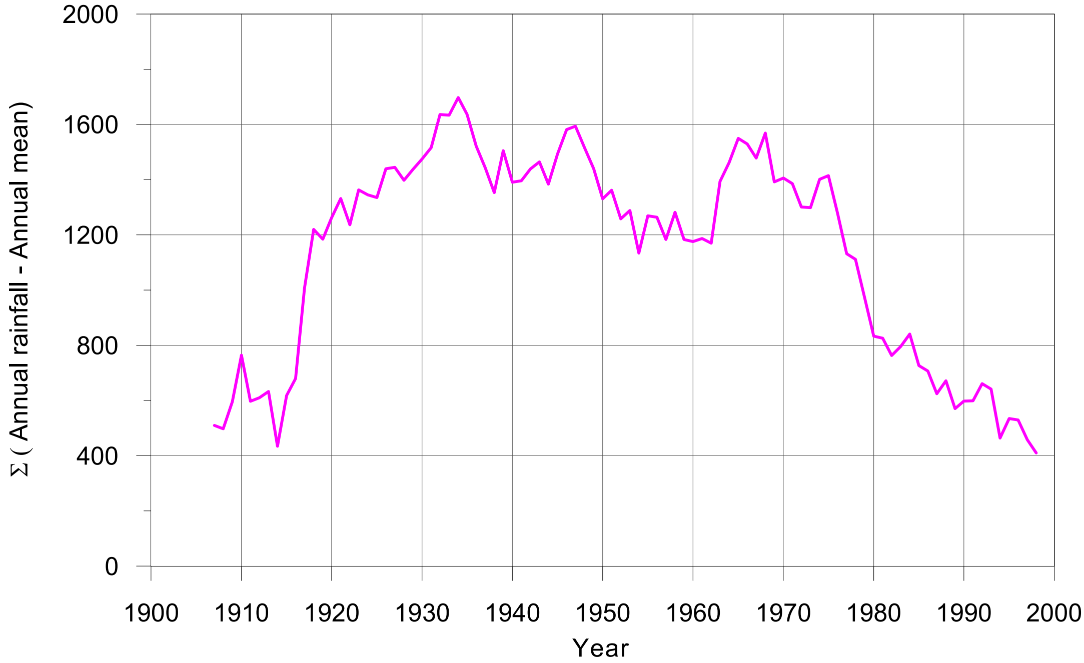

Figure C3.2 Cumulative Sum Distribution of Annual Rainfalls at Berkshire Valley 2 (008008) Rainfall Station

C3.3 Previous Estimates Of Design Discharges

C.16 Two previous studies have estimated design discharges in the Moore River Catchment: GHD (1991) and Main Roads WA (1999).

C.17 GHD (1991) used the following procedure:

- A regression equation relating the 2 year ARI discharge to catchment area and average annual rainfall was derived from recorded discharge data from catchments nearby to the Moore River Catchment.
- Flood frequency analysis was carried out for nearby gauging stations. The 'average' slope of the different flood frequency lines was determined.
- This 'average' slope was then used to extrapolate the discharges from 2 year to 100 year ARI at various catchment locations.

GHD (1991) estimates are provided in Table C3.3.

C.18 The Main Roads WA (1999) used a flood frequency analysis of discharges at Quinn's Ford to estimate the 2 to 100 year ARI discharges at Quinn's Ford. The methodology used in the analysis was not explained. Table C3.3 also shows the Main Roads estimates of design discharges at Quinns Ford.

Table C3.3 Design Discharge Estimates from GHD (1991) and Main Roads WA (1999)

ARI (years)	Discharge m^3/s				
	GHD (1991)				Main Roads WA (1999)
	Moore River At Moora	Coonderoo River at Moora	Yadgena Brook at Moora	Quinn's Ford	Quinn's Ford
2	5.1	13.2	4.1	40	43
5	-	-	-	-	110
10	24	61	16	-	200
20	-	-	-	-	310
25	43	111	33	-	-
50	-	-	-	-	440
100	91	236	77	575	600

'-' Denotes not available

C.19 Both GHD (1991) and Main Roads WA (1999) estimates of design flood discharges at Quinn's Ford are consistent with the estimates of this study. It is noted that all 3 sets of analyses are based on stream discharges recorded during a below average rainfall period.

C4 CONTINUOUS SIMULATION MODELLING

C4.1 Overview

C.20 To overcome some of the difficulties associated with rainfall loss estimation, the Cooperative Research Centre (CRC) for Catchment Hydrology has recently developed a **continuous simulation** approach (Boughton, et al. 1999). This approach is based on a long-term (2000 year) simulation of catchment hydrology that aims to extend the available streamflow record. Design flood discharges are then estimated by flood frequency analysis of the simulated annual peak discharges. Using this technique, the available streamflow record for flood frequency analysis may be extended from the 20 to 50 years generally available to 2000 years. The CRC has developed a suite of computer programs for continuous simulation modelling.

C.21 An overview of the continuous simulation approach is given below:

- A daily rainfall-runoff model (Australian Water Balance Model; AWBM) is developed and calibrated against recorded **monthly** rainfall and streamflow data for an extended period (at least 5 to 10 years if available). The monthly timeframe is adopted to minimise difficulties associated with a potential lack of correlation between recorded rainfalls and streamflows if rainfall recorded on one day is recorded as runoff on another day.

- A flood hydrograph model (WBMOD) is developed and calibrated against recorded hourly data for a number of flood events. The current CRC software adopts a spatially-lumped model based on a unit-hydrograph approach.

- A synthetic 2000 year sequence of daily rainfall with similar statistical properties to recorded rainfall is generated for the catchment of interest. Daily rainfalls are disaggregated to hourly values using IFD data from IEAust (1998).

- The continuous simulation model then performs a 2000 year simulation of catchment hydrology. During periods of little or no rainfall the model uses a daily timestep. When a significant runoff event is identified the model automatically switches from the daily rainfall-runoff model to the hourly hydrograph model. The output from the continuous simulation modelling is a 2000 year sequence of annual maximum design flood discharges.

- A flood frequency analysis of the 2000 year synthetic flood series is undertaken to estimate design flood discharges.

In this study the continuous simulation approach was used to obtain an independent estimate of design flood discharges at Quinn's Ford.

C.22 The continuous simulation approach has only recently been introduced into engineering practice. In addition, the current CRC software is better suited to smaller catchments in temperate regions. For these reasons the initial results of the continuous simulation modelling appeared unpromising. However, following further discussion with the developer of the continuous simulation system (Professor Walter Boughton) it was decided that the continuous simulation results could provide some useful information on the hydrology of the Moore River catchment. Hence, the continuous simulation results have been considered in the estimation of design flood discharges for the Moore River.

C4.2 Rainfall-Runoff Model

C.23 The AWBM daily rainfall-runoff model was calibrated against recorded streamflow data at Quinn's Ford for a 20 year period from 1980 to 1999. The AWBM model is based on a simple 'bucket' type hydrologic model that includes three different surface stores to represent soil moisture variation within the catchment on a daily basis.

C.24 Due to the 'spatially-lumped' nature of the model it was necessary to determine daily catchment-averaged rainfall for the 20 year period.

C.25 Twenty-seven rainfall stations were selected to provide daily rainfall estimates for the period January 1907 to September 1999. Only one station had 93 years of data (Berkshire Valley 2: 008008). Thus, the total period of record was divided into four smaller periods, based on data availability at each station. Table C4.1 shows the number of available rainfall stations and duration of each sub-period. Gaps in the daily rainfall record at each station were filled by records from a nearby station, weighted to account for long term variation between sties. The Theissen Polygon technique was used to spatially distribute point rainfall across the catchment. Daily catchment totals were then derived through addition of rainfall estimates for each polygon, factored by the percentage area of the catchment covered by the polygon. Table C4.2 lists summary statistics for the derived catchment rainfall. The maximum catchment average short duration restricted (to 0900 hours) rainfalls were:

- 1 day 97 mm (April 1961)
- 2 day 108 mm (April 1961)
- 3 day 134 mm (March 1999)

Table C4.1 Rainfall Periods Used for Estimation of Catchment Rainfall

Period	Nº of Rainfall Stations
Jan 1907 - Jul 1913	8
Aug 1913 - Dec 1931	14
Jan 1932 - Mar 1962	21
Apr 1962 - Sep 1999	23

Table C4.2 Summary Rainfall Statistics

Statistic	Average Annual Rainfall[1] (mm)	
	Catchment Average	Berkshire Valley
Minimum	213	230
Mean	410	436
Maximum	740	690

1. 92 years (1907 - 1998)

C.26 Calibration of the AWBM model was achieved using the automatic calibration programs developed by the CRC for Catchment Hydrology. The calibrated values of the AWBM model parameters are given in Table C4.3. Note that the calibration of the model is based on a comparison of recorded and predicted monthly runoff volumes. The use of monthly, rather than daily values reduces difficulties associated with the time differences (i.e. lag) between recorded rainfall and runoff (runoff may take several days to several weeks to traverse the catchment). Note also that the 20 year calibration period is the maximum allowed by the calibration program. However, it is unlikely that the use of a longer calibration period would significantly change the model calibration. The adopted model calibration produced a coefficient of determination (r^2) of 0.84 between recorded and predicted monthly flow volumes.

Table C4.3 Calibrated AWBM Model Parameters

Parameter	Value	Proportion of Catchment (%)
Surface Store (C1) (mm)	5	5
Surface Store (C2) (mm)	36	36
Surface Store (C3) (mm)	180	59
Baseflow Index (BFI)	0.468	-
Baseflow Recession Constant (Kbase)	0.967	-
Surface Flow Recession Constant (Ksurf)	0.492	-

'-' denotes 'not applicable'

C4.3 Flood Hydrograph Model

C.27 The WBMOD model was calibrated against recorded flood hydrographs for the following flood events:

- July 1983,
- July 1988,
- March 1999, and
- July 1999.

C.28 The model was calibrated using the automatic calibration programs developed by the CRC for Catchment Hydrology. Due to the large size of the catchment and the considerable difference in travel time between the various flood events, it was necessary to:

- Adjust the timing of the 1983 and March 1999 recorded hydrographs (the objective of the model is to predict hydrograph shape and peak discharges rather than timing), and
- Modify the program source code to increase the duration of the unit hydrograph model from the default 20 hours to 80 hours.

C.29 Table C4.4 shows a comparison of recorded peak flood discharges with the values obtained from the calibrated hydrograph model. The average difference between recorded and predicted peak discharges for the four calibration events is about 17%. The maximum difference is 33%. Plots of the recorded and predicted hydrographs are shown in Figures C4.1 to C4.4.

Table C4.4 Comparison of Recorded and Predicted Peak Discharges, WBMOD Model, Moore River at Quinn's Ford

Event	Peak Discharge (m^3/s)	
	Recorded	Predicted
July 1983	270	239
July 1988	127	167
March 1999	434	465
July 1999	140	162

C4.4 Synthetic Rainfall Generation

C.30 Catchment averaged daily rainfalls were produced for a 93 year period (1907 to 1999) using a Theissen weighting of available daily rainfall stations (see Section C4.2). The rainfall generation component of the continuous simulation software was then used to produce a 2000 year series of synthetic daily rainfalls.

C.31 Setting the 'A' and 'F' parameters of the rainfall generation model to 9.0 and 1.2 respectively, the estimated average annual rainfall for the generated 2000 year period was 416.1mm, compared to a value of 412.2 mm for the recorded 93 year period. Table C4.5 shows a comparison of frequency analyses of annual maximum daily rainfall for predicted and recorded catchment averaged values. The predicted daily maximums are in good agreement with the recorded values.

Table C4.5 Frequency Analyses of Annual Maximum Daily Rainfall

ARI (Years)	Annual Maximum Daily Rainfall (mm)	
	Recorded Data	Predicted
2	27.3	29.5
5	38.5	39.5
10	47.3	47.1
50	71.3	69.9
100	83.5	79.5

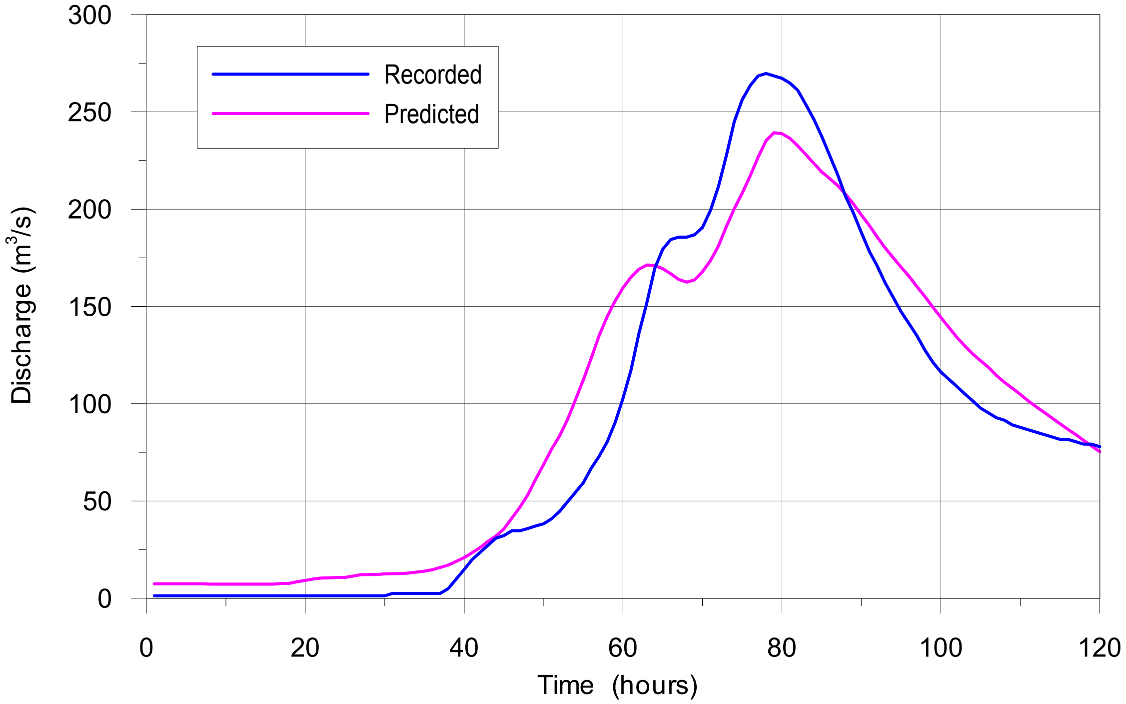

Figure C4.1 Recorded and Predicted Flood Hydrographs, WBMOD Model, Moore River at Quinn's Ford, July 1983 Event

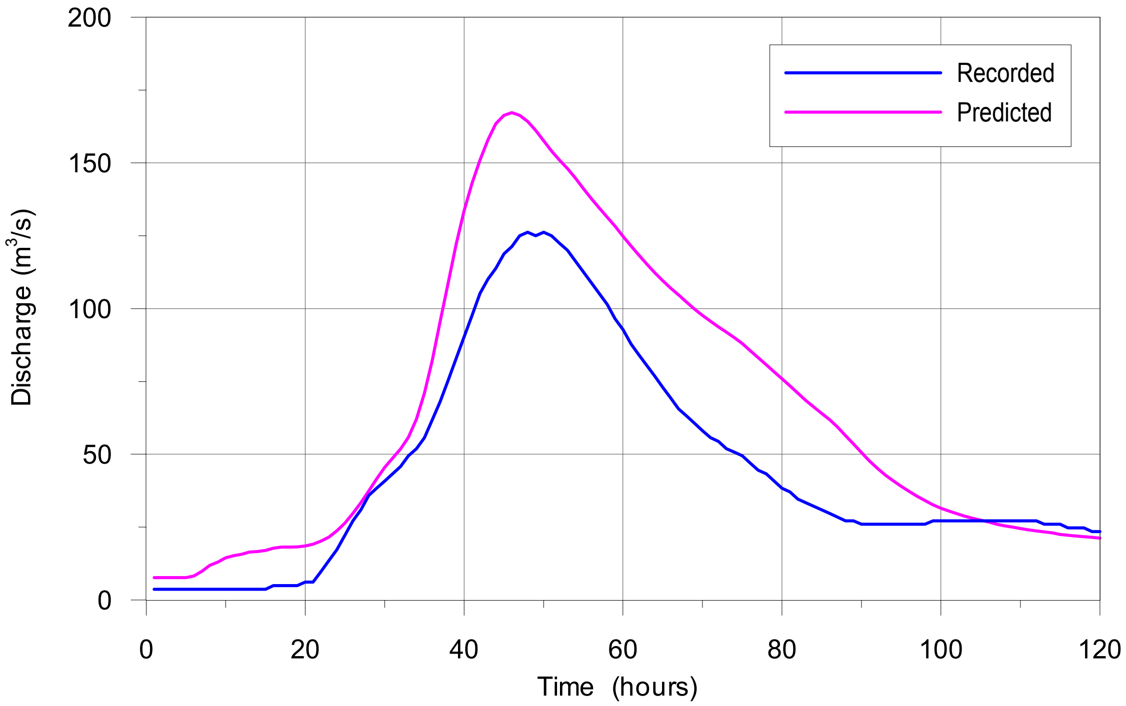

Figure C4.2 Recorded and Predicted Flood Hydrographs, WBMOD Model, Moore River at Quinn's Ford, July 1988 Event

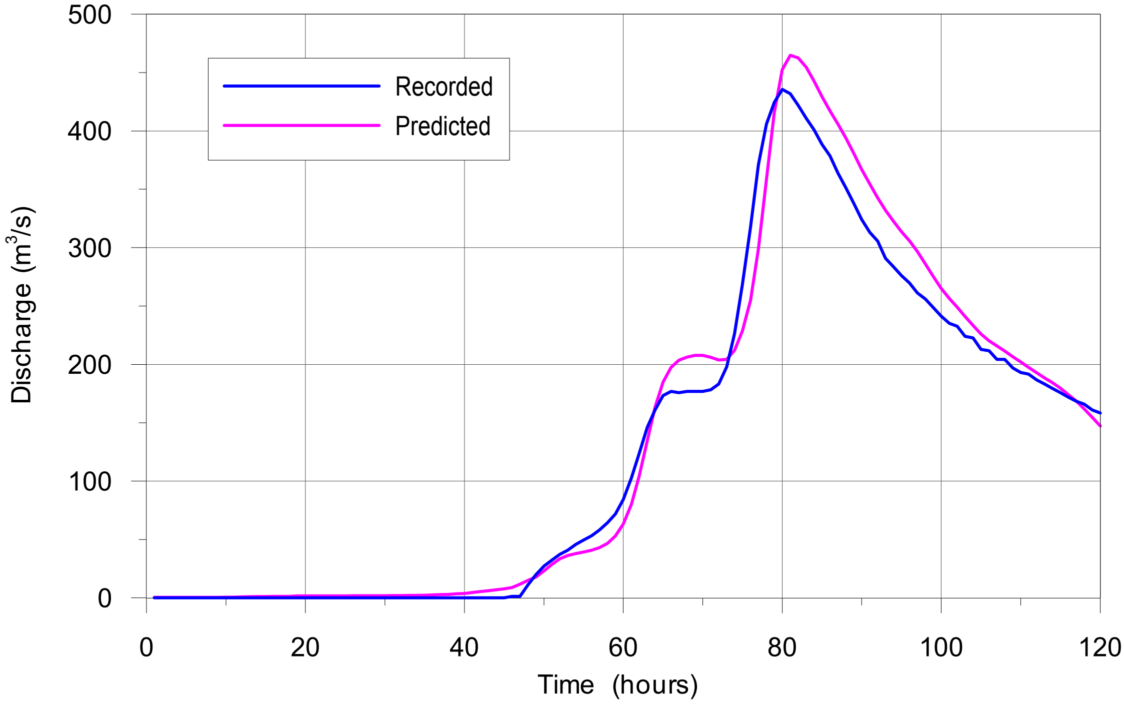

Figure C4.3 Recorded and Predicted Flood Hydrographs, WBMOD Model, Moore River at Quinn's Ford, March 1999 Event

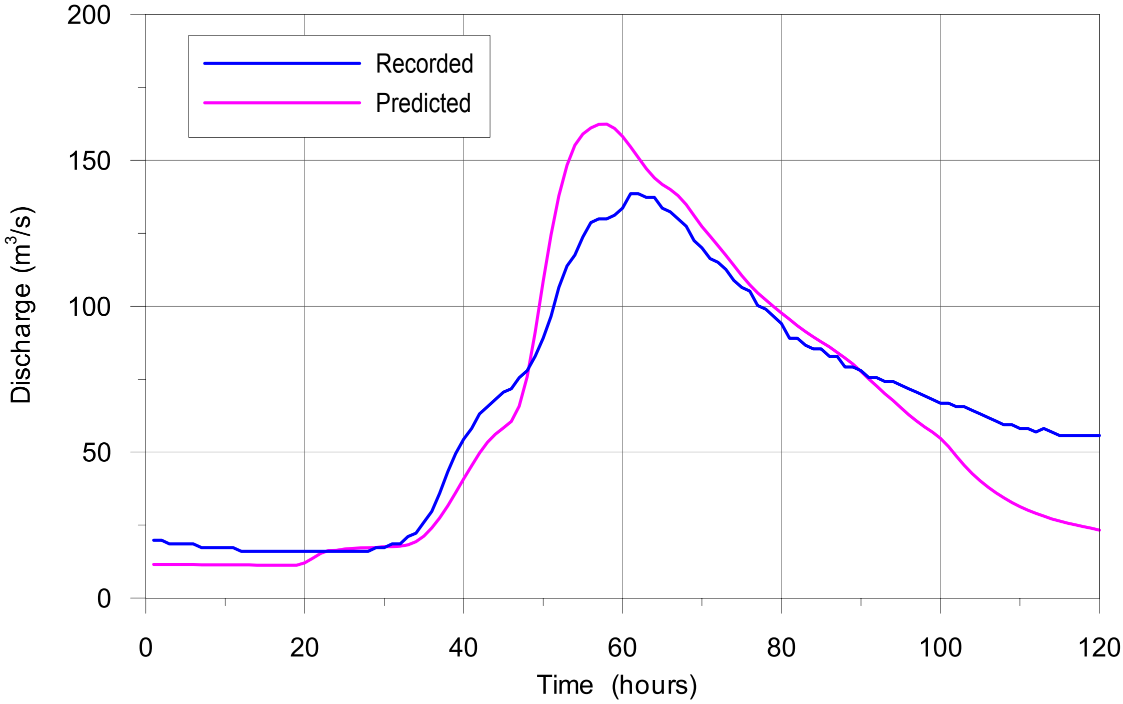

Figure C4.4 Recorded and Predicted Flood Hydrographs, WBMOD Model, Moore River at Quinn's Ford, July 1999 Event

C4.5 Design Streamflow Generation

C.32 The continuous simulation model was used to generate a 2000 year sequence of streamflow hydrographs at Quinn's Ford based on the synthetic rainfall sequence. Note that the continuous simulation model generates a different set of design rainfalls, and hence design discharges, for each run. For this reason, the model was run 10 times to obtain an average value of the design discharge and an indication of the likely range of values. The results are shown in Table C4.6.

Table C4.6 Predicted Minimum, Maximum and Average Design Discharges from Multiple Runs of the Continuous Simulation Model, Moore River at Quinn's Ford

Event ARI	Predicted Design Flood Discharges (m^3/s)		
(Years)	Minimum	Maximum	Average
2	51	98	71
5	157	177	165
10	217	248	231
20	328	373	339
50	430	484	457
100	496	560	529
200	536	635	590
500	617	823	690
1000	673	1060	768
2000	691	2394	1084

C.33 Figure C4.5 shows a comparison between the results of the continuous simulation model and the results of flood frequency analysis of recorded data at Quinns Ford. Note that, for small recurrence intervals, the design discharge estimates from the continuous simulation model are significantly greater than those obtained from flood frequency analysis. Possible reasons for the difference between the two sets of results include:

- Spatial variability across the catchment. The continuous simulation model is a 'spatially-lumped' model and hence cannot simulate variability in rainfall or runoff generation across the catchment. Partial area runoff would be a significant aspect of flooding behaviour in the Moore River catchment.

- Temporal variation of rainfall. It is uncertain whether the dissagregation algorithm used in the model suitably represents the temporal variation in rainfall across a large catchment such as the Moore River.

- Inability of the model to simulate in-stream transmission losses, which could also play a significant role in the hydrology of the Moore River catchment.

- Uncertainty surrounding the suitability of the Transitional Probability Matrix (TPM) approach used in the model for stochastic rainfall generation in semi-arid regions.

C.34 Despite these difficulties, the continuous simulation model produced realistic estimates of maximum annual daily rainfall and flood discharge. It is possible that the available 31 year streamflow record at Quinn’s Ford represents relatively ‘dry’ conditions compared to the expected long-term average. Hence, the results of the continuous simulation model have not been completely disregarded.

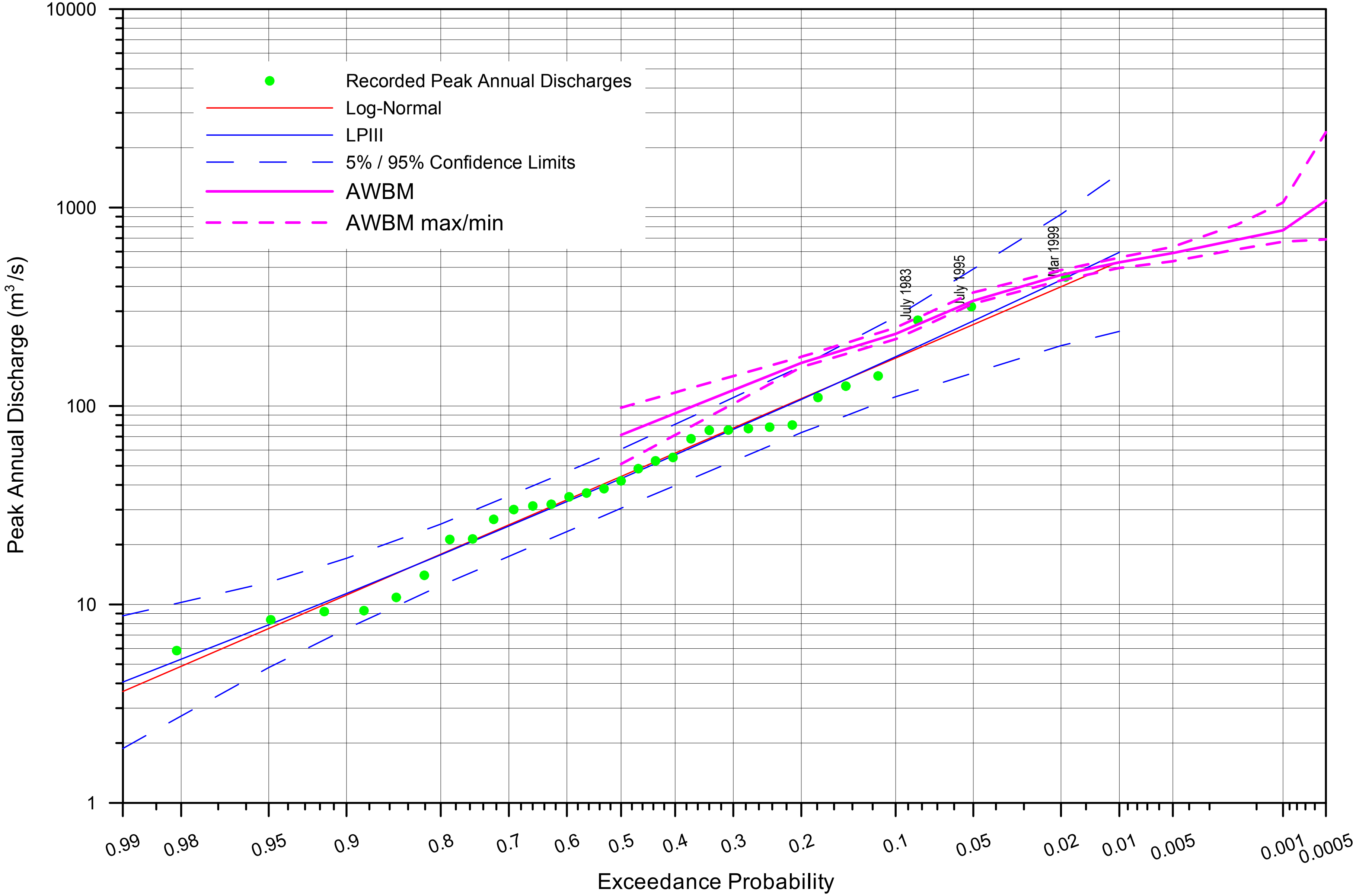

Figure C4.5 Distribution of Peak Annual Flows, Moore River at Quinn's Ford

C5 DESIGN FLOOD DISCHARGE ESTIMATION

C5.1 Design Rainfalls

C.35 Design rainfall intensities and temporal patterns for storms of various durations up to ARI's of 100 years were obtained from IEAust (1998). The variation in rainfall totals across the catchment was determined by calculating design rainfalls at the catchment's northern and southern boundaries. The spatial resolution of the predicted design rainfalls showed very little difference in rainfall estimates between catchment extremities. On this basis, rainfall estimates for the town of Moora were adopted to represent the entire Moore River Catchment. Adopted rainfall intensities for a range of storm durations and ARI's are listed in Table C5.1. An areal reduction factor of 1.0 was adopted for all design events of up to 100 years ARI.

C.36 The WRC provided estimates of the Probable Maximum Precipitation (PMP) intensities for the Moore River north catchment upstream of Moora. The PMP intensities are also listed in Table C5.1.

Table C5.1 Design Rainfall Intensities, Moore River Catchment

Storm duration (hours)	Design Rainfall Intensity (mm/hr)						
	2 yr ARI	5 yr ARI	10 yr ARI	20 yr ARI	50 yr ARI	100 yr ARI	PMP
2	10.9	14.0	16.0	18.9	23.2	26.7	-
6	5.08	6.51	7.42	8.74	10.7	12.2	52
9	3.83	4.90	5.58	6.57	7.99	9.17	-
12	3.14	4.01	4.57	5.38	6.53	7.50	40.0
18	2.34	3.00	3.43	4.05	4.94	5.68	33.9
24	1.89	2.44	2.79	3.30	4.03	4.65	29.2
30	1.59	2.06	2.36	2.79	3.42	3.95	-
36	1.39	1.80	2.07	2.45	3.01	3.48	21.9
48	1.10	1.44	1.66	1.97	2.42	2.81	17.5
72	0.78	1.02	1.18	1.41	1.75	2.03	13.1

'-' denotes not available

C5.2 Design Discharges Based On IEAust (1998) Rainfall Loss Estimates

C.37 For Western Australia, IEAust (1998) provides regional estimates of initial and continuing losses for use in rainfall-runoff models. These loss rates were used in the calibrated URBS model, along with the above design rainfalls, to estimate peak discharges at Quinn's Ford. Adopted values of initial loss for the Moore River Catchment are listed in Table C5.2. A continuing loss of 3 mm/hour was adopted for all ARI's, as recommended in IEAust (1998). Design discharge estimates are plotted in Figure C5.1.

Table C5.2 Adopted Initial Losses for Moore River Catchment Based on IEAust (1998) Estimates

ARI (years)	Initial Loss (mm)
2	22.1
5	28.3
10	30.8
20	26.8
50	28.3

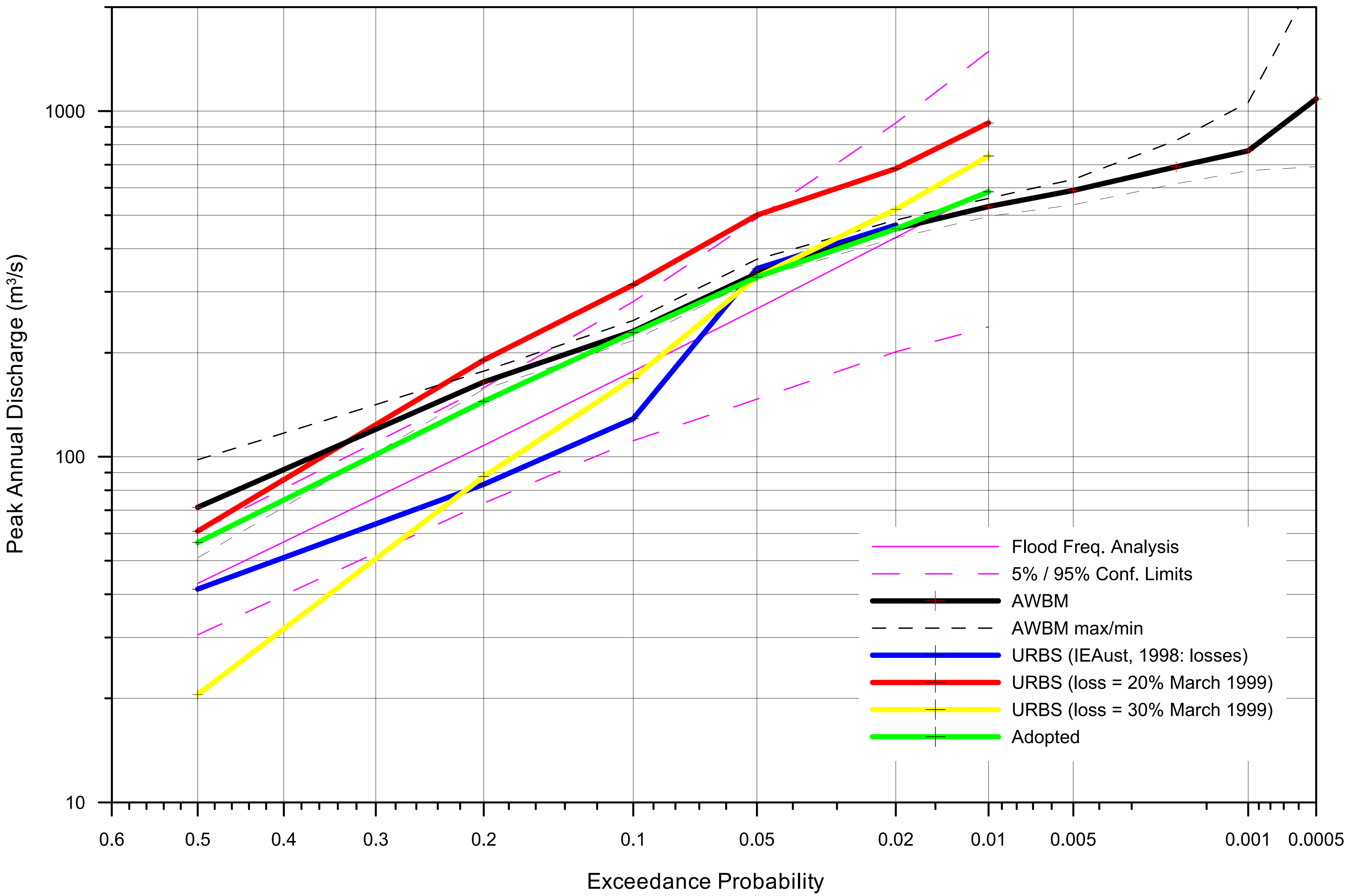

Figure C5.1 Design Discharge Estimates for Moore River at Quinn's Ford

C5.3 Design Discharges Based On March 1999 Rainfall Loss Estimates

C.38 Calibrated initial and continuing loss rates for the March 1999 event were adopted as the basis for estimation of design discharges. Initial losses for each sub-catchment (with the exception of sub-catchments within the Coonderoo catchment) were factored by a set percentage. Initial losses in the Coonderoo catchment (URBS sub-catchments 1 to 10) were fixed at the calibrated March 1999 values. Continuous loss rates were maintained at March 1999 levels. Figure C5.1 shows model output for initial loss values of 20% and 30% of calibrated March 1999 values.

C5.4 Adopted Design Parameters

C.39 Calibrated URBS model catchment parameters were adopted for the estimation of design discharges. As described above, March 1999 initial loss rates were factored to produce an acceptable flood frequency distribution at Quinn's Ford (i.e. a flood frequency distribution that is consistent with the results from other methods). One adjustment factor was applied to all values of initial loss for each design ARI's, (with the exception of sub-catchments within of the Coonderoo Catchment), however factors varied between different ARI. Initial losses in the Coonderoo catchment (URBS sub-catchments 1 to 10) were maintained at the calibrated March 1999 values to ensure that Coonderoo River discharges did not impact on downstream design discharges up to the 100 years ARI event. Adopted initial loss adjustment factors are listed in Table C5.3. Continuing loss rates were maintained at the March 1999 levels for all design events.

C.40 The reduction factors in Table C5.3 indicate that initial loss increases with event magnitude. This appears counter-intuitive since it is generally expected that rainfall losses reduce with increasing event magnitude. This is most likely attributed to rainfall seasonality, with the larger events generally occurring in summer, when losses are high, and the smaller events occurring in winter, when the losses are low.

Table C5.3 Adopted Initial Loss Reduction Factors

ARI (years)	Reduction Factor (% of March 1999 value)
2	0.21
5	0.24
10	0.26
20	0.30
50	0.34
100	0.40

C5.5 Adopted Design Discharges

C.41 There is considerable uncertainty in the estimation of the severity of peak discharges in the Moore River catchment. Figure C5.1 shows the adopted design discharges together with discharge estimates from:

- Flood frequency analysis,
- Continuous Rainfall - Runoff simulations, and
- URBS model simulations using IEAust (1998) loss estimates.

C.42 The adopted design discharges take into account the following:

- The streamflow record is relatively short (31 years), hence the flood frequency analysis most likely does not represent the long-term distribution of annual peak discharges
- The 31 years of streamflow record occurred during a period of below average rainfall (based on the last 93 years) (see section C3.2). Hence, design discharge estimates from the flood frequency analysis are likely to be low.
- A number of different analyses estimate the 100 year ARI peak discharge at Quinn's Ford to be between 520 and 600 m^3/s.
- The design discharge estimates for the 20 and 50 year ARI discharges are similar to the URBS model using IEAust (1998) loss parameters.
- The adopted discharge curve lies between the two 95% confidence limits of the flood frequency analysis.

C.43 Table C5.4 and Figure C5.2 show the adopted design discharges for the Moore River at Moora Caravan Park and Quinn's Ford, as well as for Yadgena Brook at Walebing Road. The critical storm duration producing the design discharge at the three sites for all ARI's was 24 hours, with the one exception being the 50 years ARI discharge in Yadgena Brook at Walebing Road, which had a critical duration of 12 hours. Table C5.4 also shows the peak discharge estimates from the URBS hydologic model for the March, May, July and August 1999 events.

Table C5.4 Adopted Design Flood Discharges, URBS Model

ARI (years)	Peak Discharge (m^3/s)		
	Moore River North at Moora Caravan Park	Yadgena Brook at Walebing Road	Moore River at Quinn's Ford
2	38	6.3	57
5	83	17	145
10	110	28	229
20	159	42	331
50	230	54	457
100	290	68	584
PMF	6,300	1,270	13,300
March 1999	501	97	440
May 1999	285	37	298
July 1999	33	15	138
August 1999	19	131	136

Note: March, May, July and August event discharges are URBS model estimates.

C.44 The calibrated URBS model, was used to determine the probable maximum flood (PMF) discharges at locations of interest in the Moore River catchment. Initial losses were set to zero, with the exception of those applying to the Coonderoo Catchment (URBS sub-catchments 1 to 10) which retained the calibrated March 1999 values. PMF discharge estimates of Moore River at Moora Caravan Park and Quinns Ford and Yadgena Brook at Walebing Road are given in Table C5.4. The critical storm durations for catchments draining to the Moora Caravan Park, Walebing Road and Quinns Ford for the PMF event are 24, 12 and 36 hours respectively.

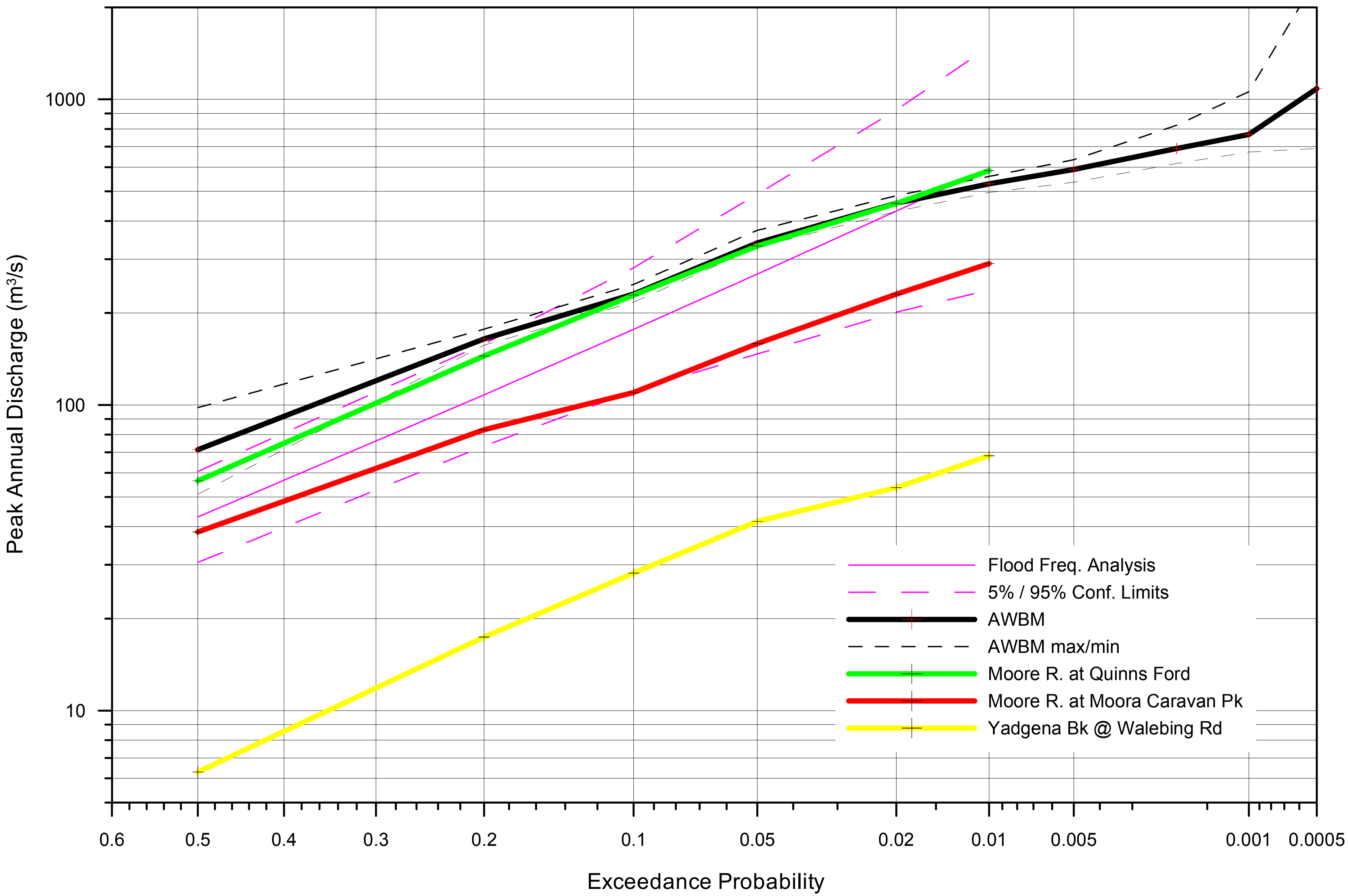

Figure C5.2 Adopted Design Discharge Estimates for Moore River at Quinns Ford and Moora Caravan Park, and Yadgena Brook at Walebing Road

C5.6 Comparison With Previous Design Discharge Estimates

C.45 Table C5.5 shows the comparison of design discharge estimates from the current study with those from the previous study of GHD (1991). Peak discharge estimates at Moora from the current study are significantly greater than those of GHD (1991). The difference in peak discharge estimates at Yadgena Brook between the two studies is smaller.

Table C5.5 Comparison of Design Flood Discharge Estimates, GHD (1991) and Water Studies (2000)

ARI (Years)	Peak Discharge (m^3/s)			
	Moore River North at Moora Caravan Park		Yadgena Brook at Walebing Road	
	Water Studies (2000)	GHD (1991)	Water Studies (2000)	GHD (1991)
2	38	5.1	6.3	4.1
10	110	24	28	16
100	290	91	68	77

C6 SEVERITY OF MARCH AND MAY 1999 EVENTS

C6.1 Rainfall Intensities

C.46 Table C6.1 shows one, two and three day duration rainfall totals for the March, May, July and August 1999 events at Berkshire Valley, together with the design rainfalls for the 100 year ARI event. Figure C6.1 shows the 100 year ARI rainfall intensity-frequency-duration estimates from IEAust (1998) (see section C5.1), together with the recorded data at Berkshire Valley for the four modelled events. It can be seen that the March 1999 rainfall exceeded the design 100 year ARI rainfall for all durations greater than 9 hours.

Table C6.1 Rainfall Intensity-Duration Estimates, March, May, July and August 1999 Events

Duration (days)	Rainfall (mm)				
	Berkshire Valley				100 Year ARI Design Rainfall
	March	May	July	August	
1	121.2	61.2	27.0	9.6	111.6
2	134.8	101.4	27.6	12.2	134.9
3	166.1	105.9	27.6	13.6	146.2

C6.2 Design Discharge Extrapolation

C.47 Linear extrapolation of the estimated design discharges for Moora Caravan Park (see Figure C5.2) suggests that the May 1999 event (with a peak discharge of 285 m^3/s) was equal to the 100 year ARI event, and the March 1999 event (with a peak discharge of about 500 m^3/s) was significantly larger than the 100 year ARI event. It is unlikely that both these events would have had ARI's equal to or greater than 100 years at Moora Caravan Park and much smaller ARI's at Quinns Ford.

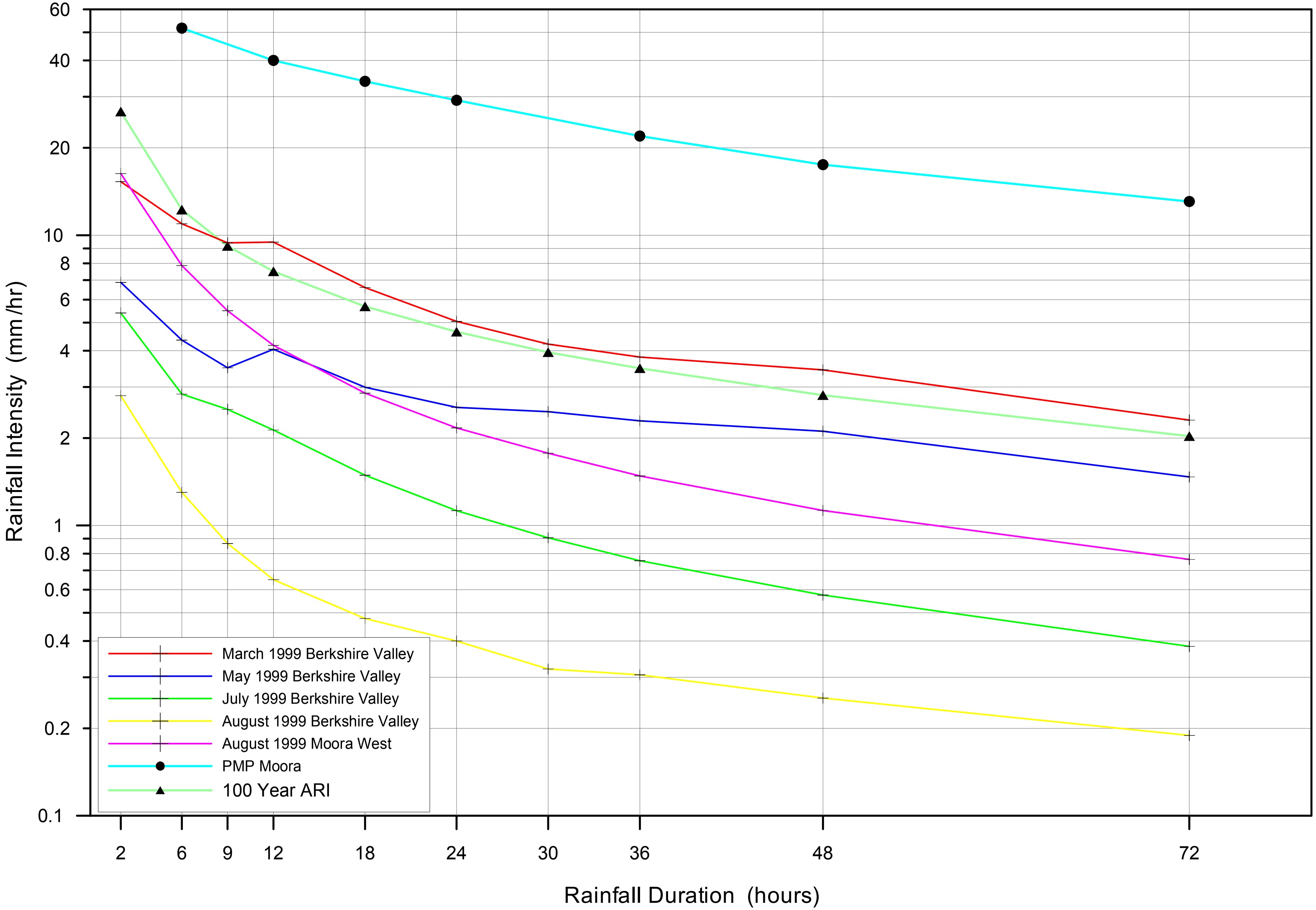

Figure C6.1 Rainfall Intensity - Frequency - Duration Curves for the Study Area

C.48 Differences in runoff generation between design and observed rainfalls were investigated by comparing rainfall excesses (i.e. total rainfall - total losses). Table C6.2 lists rainfall excesses and total losses (initial plus continuing) for the March and May 1999 event and the 100 year ARI design event for the Moore River Catchment upstream of Moora Caravan Park. The differences between the three rainfall data sets are:

- The March and May 1999 rainfall values are for periods **within** a larger storm, whereas the design rainfalls are the **total** event rainfall, and
- Differences in temporal patterns. The design rainfall pattern displays a high initial burst lasting one hour followed by a gradual reduction in rainfall intensity. In contrast, the March and May 1999 rainfall temporal patterns display long periods of steady rainfall, thus 'wetting up' the catchment, followed by a sustained period of high rainfall lasting a number of hours.

Table C6.2 Rainfall Excess for Moore River North Catchment, Upstream of Moora

Event	Rainfall Excess (mm)	Total Losses (mm)
March 1999	28.8	150
May 1999	19.4	81.9
100 year ARI	24.0	87.6

C.49 It is noted that the adopted design rainfall and temporal patterns were published in 1987 (IEAust, 1987) based on a very limited data set in Western Australia. In addition, these design rainfalls and temporal patterns did not take into account the distinct seasonal variability apparent in the study area. Revision of IFD estimates was not carried out for IEAust (1998). It is suspected that design rainfall estimates for the upper catchment of the Moore River are low, and/or the design temporal patterns for the upper catchment are incorrect.

C.50 There is considerable uncertainty surrounding the assignment of ARI's to recorded flood events at Moora. It is concluded that the March and May 1999 events were extreme events. Given the uncertainties in the design discharge estimation process, it is not possible to estimate the ARI of the March and May 1999 events with any confidence. However, based on a subjective assessment of all available information, it is estimated that the March and May 1999 flood events had ARI's of 100 - 250 years and 50 - 100 years respectively.

C7 REFERENCES

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D Appendix
DESIGN FLOOD LEVEL ESTIMATION

D1 INTRODUCTION

D.01 This Appendix describes the estimation of design flood levels in the Moora town area. The setup and calibration of the Mike 11 hydraulic model used to estimate design flood levels are described in Appendix B and are therefore not repeated herein.

D.02 This Appendix is divided into six sections:

- Section 2 describes the methodology adopted for the estimation of design flood levels.
- Section 3 presents the predicted design flood levels for design events ranging from 2 Year ARI to PMF.
- Section 4 presents the rating curve derived for Moore River at Moora Caravan Park.
- Section 5 presents the extent of flooding in Moora for the 100 year ARI flood event.
- Section 6 compares the design flood levels predicted in this study with equivalent values from the 1991 flood study.

D2 METHODOLOGY

D.03 The calibrated Moora Mike 11 model was used to estimate peak flood levels and the extent of flooding along Moore River and Yadgena Brook at Moora. Peak flood levels were estimated for the 10, 20, 50, and 100 Year ARI events and the PMF.

D.04 The Moora Mike 11 model was calibrated to the March, May and July 1999 flood events using the available data. At Moora, the March, May and July 1999 flood events had peak discharges of 500, 280 and 30 m^3/s respectively. It is likely that not all of the overflow locations along the Moore River No. 1 channel have been identified in the model calibration because of the limited data available on flooding in Moora. This may result in small inaccuracies in the predicted peak flood levels and extent of flooding.

D.05 The peak discharge for the PMF at Moora is estimated to be 6,300 m^3/s, which is more than 12 times larger than the peak discharge for the March 1999 event. The surveyed cross-sections at certain locations were not wide enough to capture the full extent of flooding for the PMF. To overcome this, the model assumes that there is a vertical wall at the end of each cross-section to contain the total discharge. Any errors in active flow areas are expected to be generally small compared with the inundated area. However there may be some floodplain storage to the north of Moora that has not been included in the model and this may affect predicted peak flood levels. As a consequence, the model may slightly over-predict the peak flood levels for the PMF along the Moore River. No peak flood levels are provided along the Yadgena Brook for the PMF due to insufficient survey of Yadgena Brook south of Walebing Road.

D3 PEAK DESIGN FLOOD LEVELS

D.06 Table D3.1 shows peak flood level estimates at model cross-sections for the 10, 20, 50 and 100 Year ARI flood events and the PMF (see Figure B2.1 of Appendix B for cross-section locations). Figures D3.1, D3.2, D3.3 and D3.4 show predicted peak flood level profiles for the five design flood events along the Moore River No. 1, 2, 3 and 4 branches respectively.

Table D3.1 Peak Design Flood Levels at the Cross-sections, Moora Mike 11 Model

Branch	Branch Chainage (km)	Peak Flood Level (m AHD)				
		10 Yr ARI	20 Yr ARI	50 Yr ARI	100 Yr ARI	PMF
MOORE 1	0.00	207.81	208.02	208.18	208.29	211.38
MOORE 1	0.32	207.57	207.81	207.97	208.09	211.08
MOORE 1	0.44	207.38	207.66	207.84	207.96	210.92
MOORE 1	0.86	206.74	207.00	207.25	207.36	210.41
MOORE 1	1.33	206.22	206.46	206.73	206.85	209.62
MOORE 1	1.58	206.00	206.22	206.46	206.59	209.16
MOORE 1	1.87	205.80	206.02	206.24	206.40	208.80
MOORE 1	1.92	205.77	206.00	206.23	206.38	208.82
MOORE 1	2.59	205.05	205.41	205.59	205.68	208.06
MOORE 1	2.63	205.06	205.40	205.58	205.66	208.00
MOORE 1	2.76	204.94	205.25	205.40	205.47	207.69
MOORE 1	3.18	204.56	204.71	204.85	204.92	207.02
MOORE 1	3.60	204.07	204.29	204.45	204.54	206.71
MOORE 1	3.72	203.94	204.19	204.36	204.46	206.65
MOORE 1	3.93	203.73	203.97	204.17	204.28	206.45
MOORE 1	4.19	203.32	203.59	203.90	204.04	206.25
MOORE 1	4.48	202.97	203.28	203.66	203.86	206.04
MOORE 1	4.49	202.98	203.28	203.67	203.88	206.33
MOORE 1	4.54	202.95	203.27	203.66	203.87	206.22
MOORE 1	4.72	202.86	203.15	203.42	203.54	206.03
MOORE 1	4.86	202.79	203.06	203.32	203.43	205.84
MOORE 1	4.89	202.73	202.99	203.24	203.34	205.68
MOORE 1	5.06	202.53	202.77	202.98	203.05	205.40
MOORE 1	5.30	202.28	202.50	202.71	202.78	205.29
MOORE 1	5.55	202.06	202.30	202.50	202.57	205.16
MOORE 1	5.57	202.06	202.29	202.49	202.56	205.17
MOORE 1	5.60	201.98	202.20	202.37	202.42	205.06
MOORE 1	5.78	201.88	202.10	202.26	202.30	204.99
MOORE 1	6.08	201.64	201.82	201.95	201.99	204.79
MOORE 1	6.56	201.17	201.25	201.37	201.46	204.48
MOORE 1	6.69	201.10	201.18	201.31	201.42	204.48
MOORE 1	6.97	200.95	201.02	201.12	201.21	204.10
MOORE 1	7.17	200.76	200.85	200.96	201.05	203.99
MOORE 1	7.34	200.67	200.76	200.87	200.94	203.83
MOORE 1	7.89	200.34	200.41	200.50	200.55	203.37
MOORE 1	8.46	199.87	199.95	200.05	200.11	202.97
MOORE 1	9.01	199.46	199.54	199.65	199.72	202.49
MOORE 1	9.81	198.75	198.86	198.99	199.08	201.74
YADGENA BROOK	0.00	207.59	207.80	207.95	208.10	Nc
YADGENA BROOK	0.31	206.72	206.93	207.09	207.25	Nc
YADGENA BROOK	0.36	206.51	206.73	206.92	207.11	Nc
YADGENA BROOK	0.76	205.58	205.87	206.06	206.22	Nc
YADGENA BROOK	1.16	204.45	204.81	205.05	205.23	Nc
YADGENA BROOK	1.74	202.71	203.05	203.31	203.51	Nc
YADGENA BROOK	1.81	202.41	202.72	203.31	203.30	Nc
YADGENA BROOK	2.20	202.29	202.59	202.78	202.95	Nc
YADGENA BROOK	2.56	201.52	201.79	201.95	202.09	Nc
YADGENA BROOK	3.02	200.88	201.11	201.26	201.39	Nc
YADGENA BROOK	3.87	200.05	200.18	200.28	200.35	Nc
YADGENA BROOK	4.53	199.23	199.31	199.41	199.46	Nc
YADGENA BROOK	4.87	198.71	198.83	198.94	199.06	Nc
COONDEROO	0.00	201.22	201.33	201.47	201.63	205.44
COONDEROO	0.44	201.22	201.33	201.47	201.63	205.38
COONDEROO	0.80	201.22	201.33	201.47	201.63	205.22
COONDEROO	0.81	201.22	201.33	201.47	201.63	205.21
MOORE 5	0.00	200.76	200.85	200.96	201.05	203.99
MOORE 5	0.26	200.40	200.55	200.72	200.83	203.99
MOORE 5	0.45	200.19	200.34	200.52	200.64	203.81
MOORE 5	0.89	199.88	200.02	200.22	200.33	203.46
MOORE 5	1.43	199.70	199.82	199.99	200.09	203.04
MOORE 5	1.91	199.42	199.53	199.68	199.77	202.55
MOORE 5	2.67	198.71	198.83	198.97	199.06	201.77
MOORE 2	0.00	207.57	207.81	207.97	208.09	211.08

'-'Denotes no flooding occurs
nc: Not calculated (see Section D2 for details)

Table D3.1 Peak Design Flood Levels at the Cross-sections, Moora Mike 11 Model - Cont'd

Branch	Branch Chainage (km)	Peak Flood Level (m AHD)				
		10 Yr ARI	20 Yr ARI	50 Yr ARI	100 Yr ARI	PMF
MOORE 2	0.57	207.04	207.23	207.33	207.41	210.36
MOORE 2	0.96	206.66	206.87	206.98	207.07	209.64
MOORE 2	1.30	206.28	206.51	206.63	206.71	208.97
MOORE 2	1.63	205.81	206.03	206.15	206.21	208.40
MOORE 2	2.01	205.47	205.63	205.71	205.73	208.04
MOORE 2	2.12	205.42	205.56	205.63	205.65	207.92
MOORE 2	2.24	205.38	205.51	205.57	205.60	207.88
MOORE 2	2.99	204.78	204.93	205.01	205.06	207.46
MOORE 2	3.42	204.19	204.35	204.48	204.65	207.05
MOORE 2	3.72	203.87	204.00	204.21	204.42	206.71
MOORE 2	3.97	203.72	203.84	203.98	204.19	206.42
MOORE 2	4.21	203.65	203.75	203.86	204.07	206.39
MOORE 2	4.25	203.33	203.50	203.73	204.03	206.33
MOORE 2	4.40	203.28	203.43	203.63	203.80	206.11
MOORE 2	4.58	203.23	203.36	203.50	203.61	205.99
MOORE 2	4.60	202.87	203.03	203.24	203.40	205.53
MOORE 2	4.76	202.67	202.82	203.04	203.22	205.51
MOORE 2	5.18	201.94	202.22	202.53	202.76	205.40
MOORE 2	5.35	201.70	201.95	202.29	202.54	205.34
MOORE 2	5.73	201.57	201.80	202.09	202.30	205.24
MOORE 2	6.02	201.39	201.56	201.88	202.03	205.14
MOORE 2	6.31	201.22	201.33	201.51	201.68	205.03
MOORE 2	6.48	201.21	201.31	201.46	201.62	205.02
MOORE 2	6.65	201.20	201.30	201.44	201.59	205.01
MOORE 2	6.70	201.12	201.22	201.35	201.50	204.91
MOORE 2	7.15	201.10	201.18	201.31	201.42	204.48
MOORE 3	0.00	205.42	205.56	205.63	205.65	207.92
MOORE 3	0.59	204.51	204.61	204.66	204.69	207.35
MOORE 3	0.83	204.35	204.44	204.49	204.51	207.00
MOORE 3	1.13	204.11	204.31	204.37	204.39	206.46
MOORE 3	1.18	203.80	203.84	203.85	203.85	206.30
MOORE 3	1.77	202.68	202.91	202.93	202.94	205.73
MOORE 3	2.22	202.30	202.37	202.39	202.40	205.56
MOORE 3	2.60	202.15	202.18	202.20	202.20	205.44
MOORE 3	2.87	201.98	202.03	202.05	202.08	205.36
MOORE 3	3.34	201.44	201.55	201.62	201.80	205.21
MOORE 3	3.65	201.22	201.33	201.47	201.63	205.21
MOORE 3	3.91	201.21	201.31	201.46	201.62	205.02
M2-M1	0.00	207.04	207.23	207.33	207.41	210.36
M2-M1	0.14	206.91	207.10	207.20	207.29	210.25
M2-M1	0.51	206.44	206.60	206.70	206.78	209.55
M2-M1	0.83	206.20	206.33	206.43	206.54	209.11
M2-M1	1.20	205.78	206.01	206.23	206.39	208.82
M2-M1	1.23	205.77	206.00	206.23	206.38	208.82
M2-M1	1.23	205.77	206.00	206.23	206.38	208.82
ISBISTER	0.00	206.44	206.60	206.70	206.78	209.55
ISBISTER	0.40	205.81	206.01	206.12	206.24	209.02
ISBISTER	0.71	205.61	205.80	205.91	206.05	208.49
ISBISTER	1.11	204.99	205.14	205.34	205.49	208.00
ISBISTER	1.41	204.20	204.45	204.94	205.14	207.61
ISBISTER	1.76	204.17	204.37	204.73	204.88	207.13
ISBISTER	1.93	204.12	204.30	204.66	204.81	207.03
ISBISTER	2.24	203.97	204.12	204.41	204.51	206.70
ISBISTER	2.48	203.92	204.01	204.15	204.19	206.34
ISBISTER	2.73	203.92	204.01	204.13	204.16	206.31
ISBISTER	3.06	-	203.08	203.31	203.44	205.85
ISBISTER	3.26	-	202.50	202.89	203.04	205.68
ISBISTER	3.48	-	202.30	202.70	202.86	205.42
ISBISTER	4.00	-	201.63	202.09	202.17	205.06
ISBISTER	4.34	-	201.47	201.77	201.80	204.93
ISBISTER	4.85	-	201.18	201.31	201.42	204.48
MOORE 4	0.00	205.06	205.40	205.58	205.66	208.00
MOORE 4	0.13	204.99	205.28	205.42	205.49	207.59
MOORE 4	0.51	204.56	204.72	204.85	204.92	206.98
MOORE 4	0.95	203.95	204.12	204.27	204.35	206.60
MOORE 4	0.97	203.95	204.12	204.27	204.34	206.59

'-' Denotes no flooding occurs
nc: Not calculated (see Section D2 for details)

Table D3.1 Peak Design Flood Levels at the Cross-sections, Moora Mike 11 Model - Cont'd

Branch	Branch Chainage (km)	Peak Flood Level (m AHD)				
		10 Yr ARI	20 Yr ARI	50 Yr ARI	100 Yr ARI	PMF
MOORE 4	1.61	203.87	204.02	204.13	204.15	206.18
MOORE 4	1.84	203.71	203.85	203.95	204.00	205.97
MOORE 4	2.02	203.50	203.63	203.74	203.80	205.69
MOORE 4	2.13	203.40	203.51	203.60	203.66	205.51
MOORE 4	2.36	203.12	203.25	203.36	203.42	205.33
MOORE 4	2.37	203.11	203.24	203.35	203.40	205.29
MOORE 4	2.96	201.98	202.06	202.18	202.24	204.61
MOORE 4	3.16	201.55	201.64	201.76	201.82	204.44
MOORE 4	3.58	200.93	200.98	201.07	201.12	203.92
MOORE 4	3.95	199.88	200.02	200.22	200.33	203.46
CLINCH ST	0.00	203.12	203.25	203.36	203.42	205.33
CLINCH ST	0.37	202.85	202.93	203.01	203.05	204.92
CLINCH ST	0.60	202.30	202.42	202.50	202.52	204.62
CLINCH ST	0.79	201.66	201.77	201.83	201.86	204.52
CLINCH ST	1.27	201.05	201.17	201.31	201.37	204.32
CLINCH ST	1.53	200.58	200.77	200.93	200.99	204.04
CLINCH ST	1.57	200.40	200.55	200.72	200.83	203.99
CAMERON ST	0.00	203.95	204.12	204.27	204.35	206.60
CAMERON ST	0.33	203.69	203.86	203.97	204.05	206.41
CAMERON ST	0.58	203.18	203.30	203.69	203.90	206.15
CAMERON ST	0.71	202.97	203.28	203.66	203.86	206.04
M1M4-1	0.00	204.56	204.71	204.85	204.92	207.02
M1M4-1	0.10	204.56	204.72	204.85	204.92	206.98
WALEBING RD	0.00	-	-	206.92	207.11	nc
WALEBING RD	0.36	-	-	206.49	206.69	nc
WALEBING RD	0.73	-	-	206.10	206.23	nc
WALEBING RD	1.28	-	-	204.94	205.06	nc
WALEBING RD	1.91	-	-	203.89	204.04	nc
WALEBING RD	2.22	-	-	203.74	203.84	nc
WALEBING RD	2.66	-	-	203.61	203.66	nc
BINDOON RD	0.00	-	-	-	-	nc
BINDOON RD	0.45	-	-	-	-	nc
BINDOON RD	0.88	-	-	-	-	nc
BINDOON RD	1.45	-	-	-	-	nc
BINDOON RD	1.77	-	-	-	-	nc
BINDOON RD	2.21	-	-	-	-	nc

'-' Denotes no flooding occurs
nc: Not calculated (see Section D2 for details)

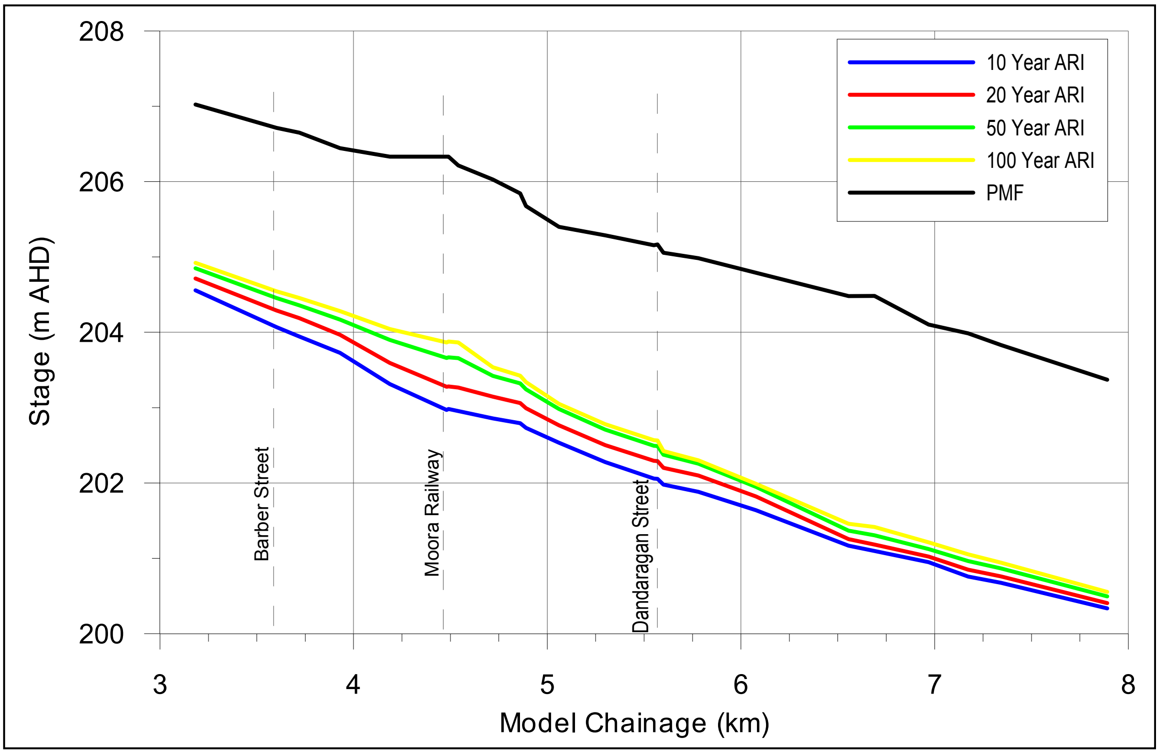

Figure D3.1 Peak Flood Level Profiles for the 10, 20, 50 and 100 Year ARI Flood Events and the PMF, Moore River No. 1 Branch

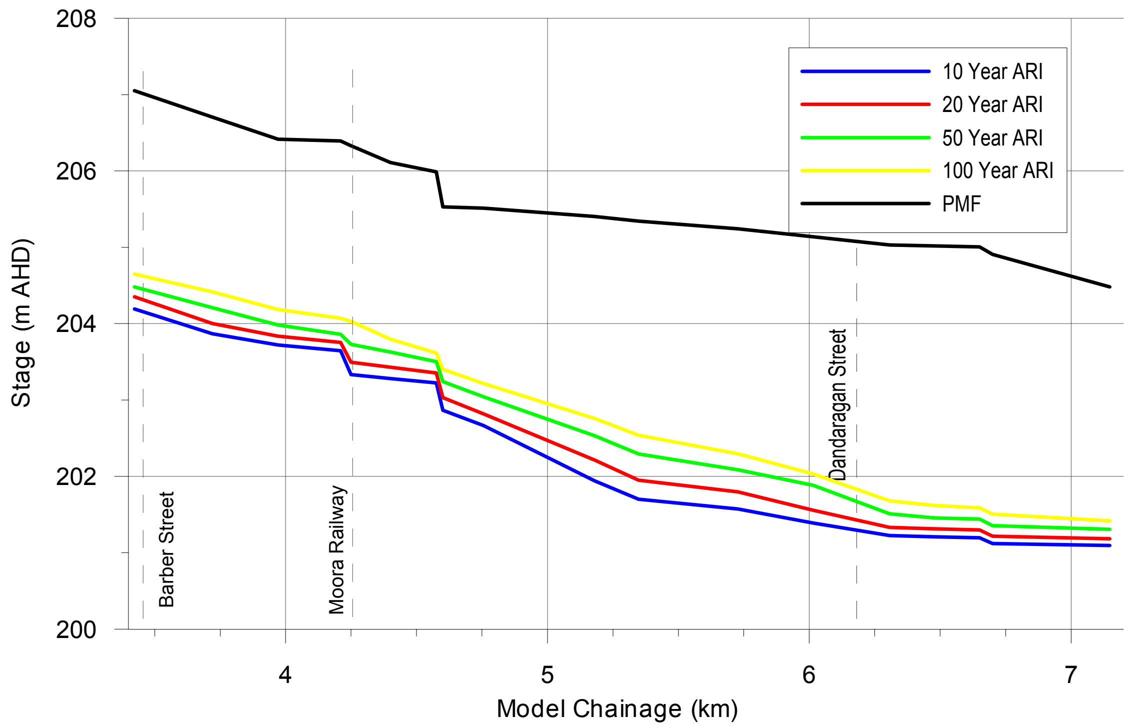

Figure D3.2 Peak Flood Level Profiles for the 10, 20, 50 and 100 Year ARI Flood Events and the PMF, Moore River No. 2 Branch

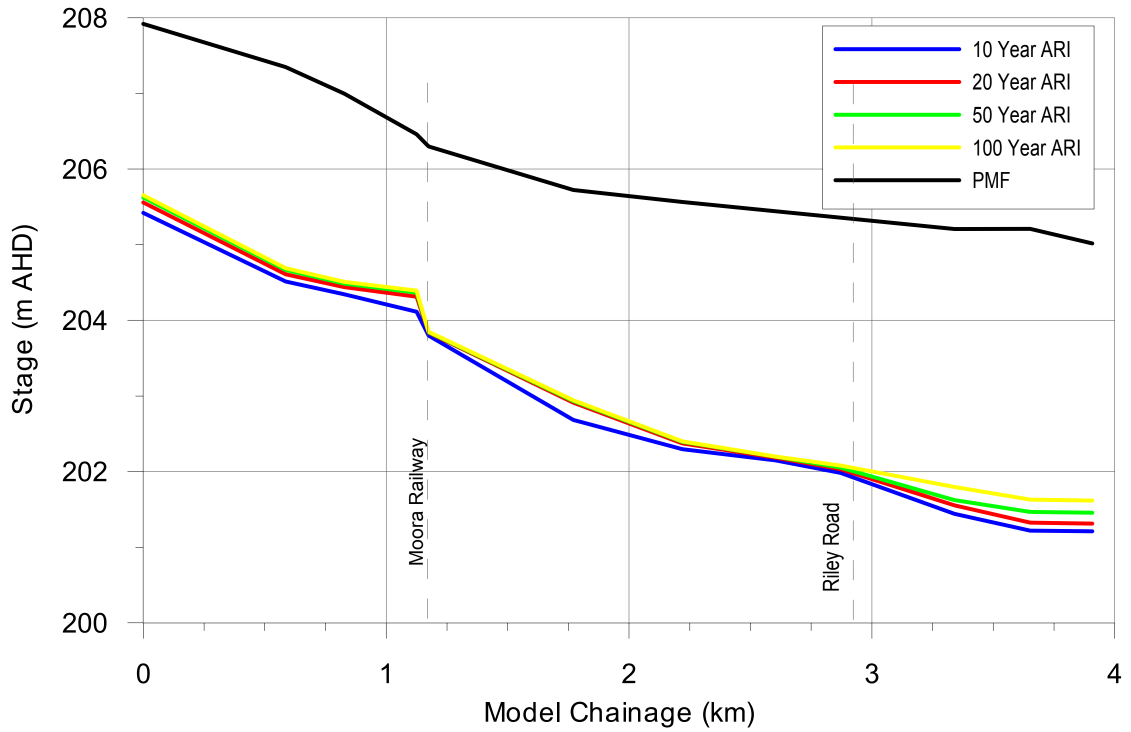

Figure D3.3 Peak Flood Level Profiles for the 10, 20, 50 and 100 Year ARI Flood Events and the PMF, Moore River No 3 Branch

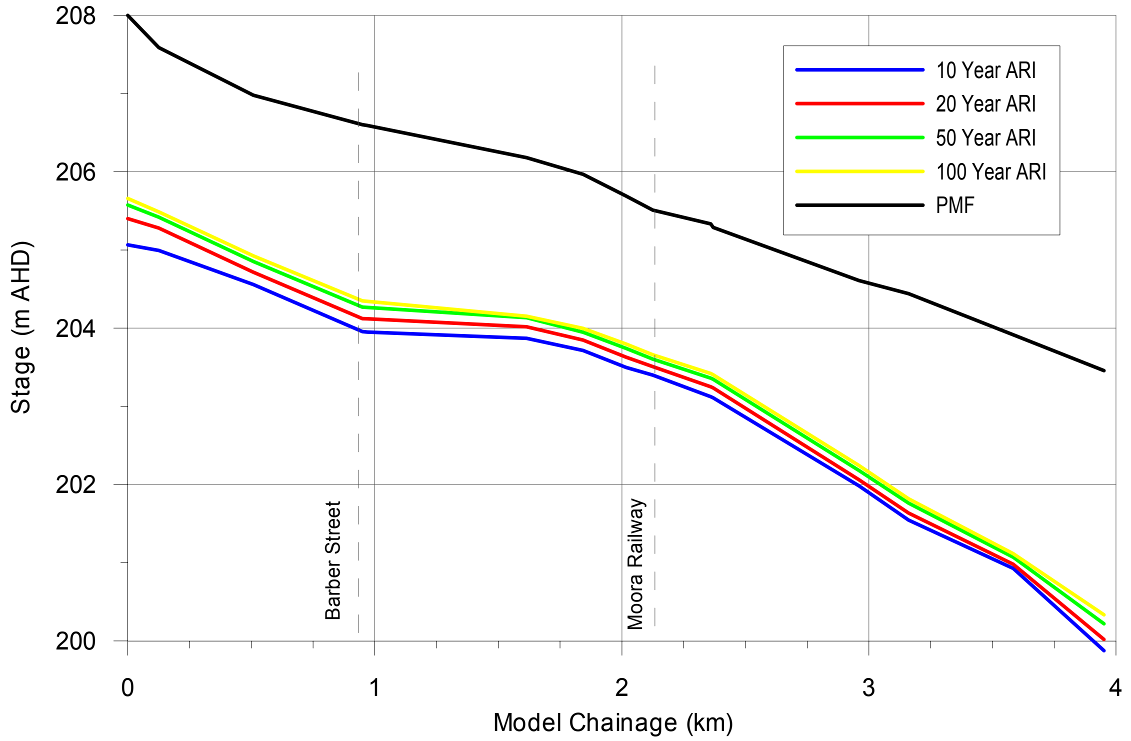

Figure D3.4 Peak Flood Level Profiles for the 10, 20, 50 and 100 Year ARI Flood Events and the PMF, Moore River No. 4 Branch

D4 RATING CURVE FOR MOORE RIVER AT MOORA CARAVAN PARK

D.07 Figure D4.1 shows the estimated high flow rating curve for the Moore River at the Moora Caravan Park gauging station (GS 617010). The curve was derived from peak discharges and flood levels estimated by the Mike 11 model for the March, May and July 1999 events, and the design flood events. The peak discharge for each event was estimated by combining the peak discharges along all branches of the Moore River at the Moora Railway line.

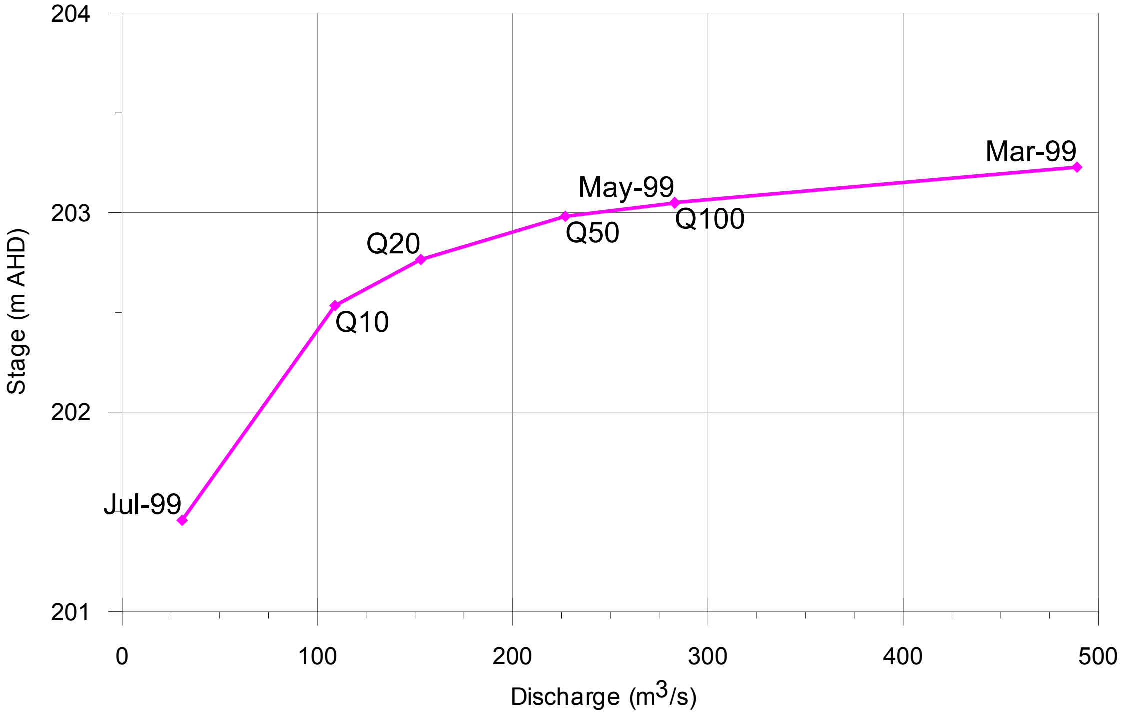

Figure D4.1 Estimated High Flow Rating Curve, Moore River at Moora Caravan Park (GS 617010)

D5 EXTENT OF FLOODING

D.08 Figure D5.1 shows the estimated extent of flooding and flood contours for the 100 Year ARI design event. Note that this design flood has not inundated part of Moora between the Moore River No. 2 and 3 Branches. There may be some local water from along the Midlands Highway or from the Coonderoo that may contribute to flooding in this area as explained in the Hydraulic Model Calibration Report (see Appendix B).

D6 COMPARISON WITH 1991 FLOOD STUDY

D.09 The peak flood level estimated in the GHD(1991) flood study for Moore River at the Moora Caravan Park (617010) for the 100 Year ARI flood is 202.63 m AHD. The 100 Year ARI peak flood level estimated in this study is some 0.42 m higher at 203.05 m AHD. It is noted that the 100 Year ARI discharge in the 1991 flood study is more or less fully contained within the Moore River No. 1 and No. 2 Branches. Most of the town is flooded at the higher 100 year ARI design flood level predicted in this study.

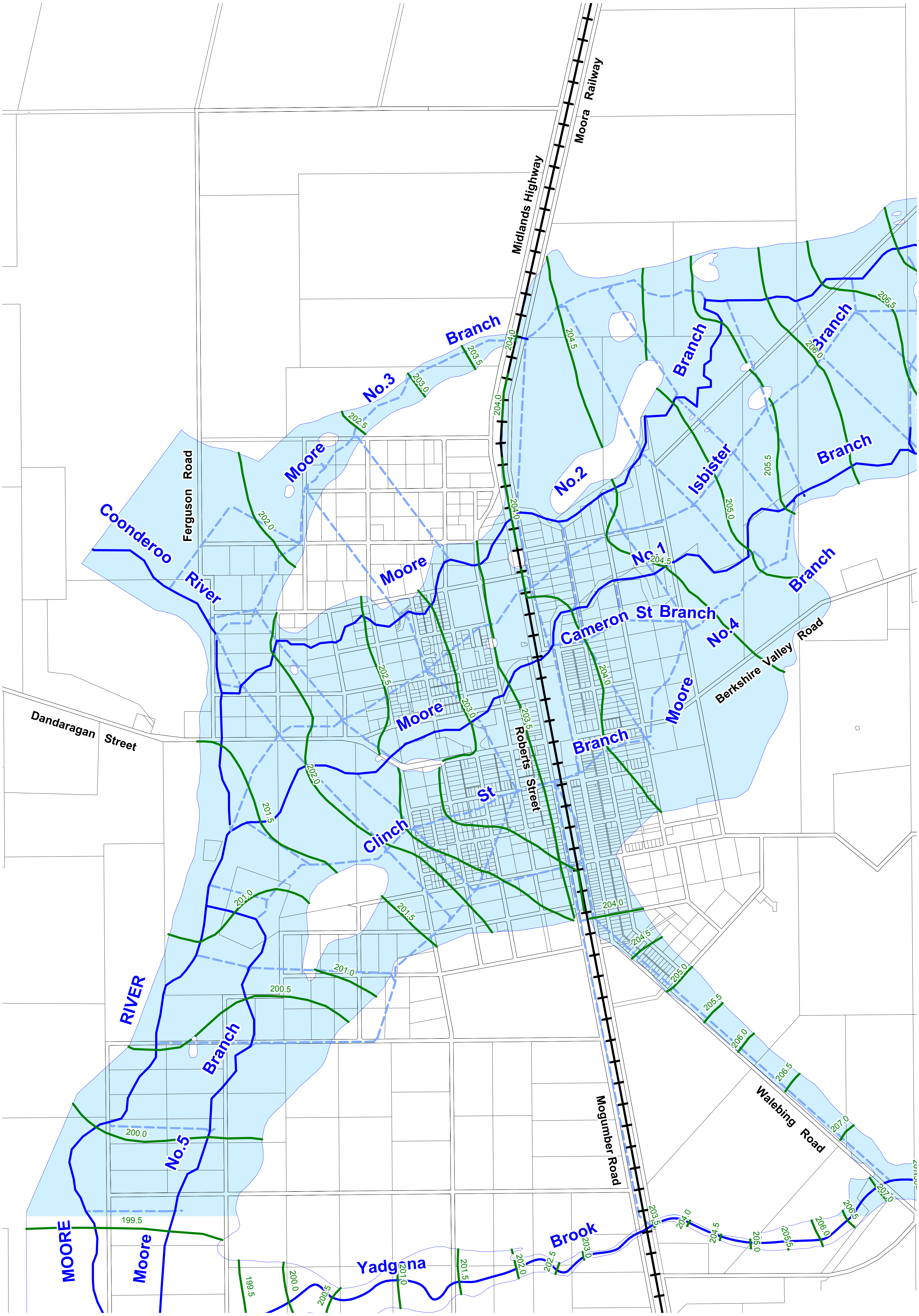

Figure D5.1 Extent of Flooding for the 100 Year ARI Design Flood Event, Moora Mike 11 Model

E Appendix
ENVIRONMENTAL ASSESSMENT OF FLOOD MITIGATION OPTIONS

E1 INTRODUCTION

E1.1 Background

E.01 This report is the result of an investigation to identify the potential environmental impacts resulting from the implementation of proposed structural floodplain management options developed by Water Studies Pty Ltd for the Moore River near the Mid West town of Moora.

E.02 Water Studies identified six structural mitigation options:

- Option 1 – Levee across the Moore River No. 4 Channel Bifurcation;
- Option 2 - Detention Basin at Longpool – 1 km upstream of gauging station;
- Option 3 - Detention Basin at 1 km upstream of Kitchin Bridge;
- Option 4 – Northern Diversion Drain and Levee;
- Option 5 – Southern Diversion Drain and Levee; and
- Option 6 – Widening of the Moore River No. 1 and No. 2 main channels.

E.02 Section 2 of this report describes the existing natural and physical environment in the immediate vicinity of the Moora townsite, while the potential environmental impacts of each of the six options are assessed in Section 3.

E1.2 Scope of Work

E.03 ATA Environmental undertook an inspection of the six floodplain mitigation options during a site visit to the area on the 29 June, 2000. The environmental issues associated with each option were investigated and an assessment made on the likely environmental impact that each of the options would have. The environmental impacts primarily focussed on the biological components most likely to be affected (i.e. flora and fauna).

E.04 In order to determine the most favourable option, each of the six options were ranked on the basis of the likely extent of their environmental impact. Once the most favourable option is determined, any further detailed studies that may be required relating to that option can be identified.

E2 EXISTING ENVIRONMENT

E2.1 Climate

E.05 Moora has a mild temperate climate, with dry warm summer and mild moist winters that are largely determined by the towns inland location and topographic feature. Average daily maximum temperature range from 33.8°C in January and February to 27.0°C in July, while average daily minimum temperatures rage from 18.1°C in February to 6.3°C in July. Average annual rainfall for Moora is 461 mm, spread over an average of 90 days with the majority falling in winter between May and July (WA Meteorological Records).

E2.2 Topography and Soils

E.06 The study area lies on the undulating Victoria Plains. Although the Darling Range Fault line passes along the western edge of Moora, it is barely visible as a topographic feature. The change in elevation over the Shire of Moora is less than 150 metres.

E.07 The northern arm of the Moore River System is characterised by alluvial and colluvial sands, silts and clays. A soil type known as Coomberdale Chert is also quite apparent to the north. The soils mapped for the Moora Flood Study area show a strong reliance in interpretation of landforms. (Stoneham, 1990; Northcote *et. al*, 1967). Soil units of major landforms such as drainage lines and lateritic uplands are analogous with certain surface geology units. The dominant soil type of the study area are sandy soils with mottled clayey subsoils. Yellow duplex soils are predominant over areas including gneissic outcrops, while small residual hills of quartzite with shallow earthy gravelly sands occur north of Moora. Duplex soils with yellow clayey subsoils predominate in the major trunk valleys of the Moore River.

E2.3 Drainage

E.08 The various branches of the Moore River are the principal drainage lines in the study area. As is the case with most drainage patterns in the south west of Western Australia, they essentially drain from east to west. However, on occasions, as is the case with the Moore system in the western part of the study area, they may drain southward through part of their trunk valleys. Drainage is usually seasonal following winter rain and many tributaries are dry during summer.

E.09 There are four branches of the Moore River in the study area: Moore River No.1, No.2, main channels and Moore River No.3 and No. 4 branches. The main branch of the river (Moore River North) arises approximately 15km to the northeast of Miling and joins with Dungaroo Creek approximately 22km to the northeast of Moora in the Berkshire Valley. From there, it bifurcates into the Moore River No.1 and No.2 channels approximately 1.4 km due north of the Berkshire Valley Road. Moore River No.1 main channel, is approximately 6.25km in length and flows through the centre of the Moore townsite, before linking up with the Coonderoo River on the western side of the town, south of Dandaragan Road.

E.10 After the Moore River No. 2 channel splits off from the No.1 branch, it flows to the north west of Barber Road, passing to the north of the town centre before joining with the Coonderoo River just north of Dandaragan Road. In total, it is approximately 6 km in length.

E.11 Moore River No.3 branch is a minor flow which branches off from Moore River No.2 just north of Barber Road and Moora townsite for a distance of approximately 3.3km before joining with the Coonderoo River near Clarke Road.

E.12 Moore River No. 4 branch splits off from Moore River No.1 main channel approximately 0.5km north of Berkshire Valley Road and flows for approximately 3.6km to the south of the Moore No.1 channel. Remnants of the channel are visible between Barber, Ranfurley and Melbourne Streets.

E2.4 Vegetation and Flora

E.13 The study area is dominated by different combination of Wandoo (*Eucalyptus wandoo*), York Gum (*Eucalyptus loxophleba* spp. *loxophleba*) Flooded Gum *(Eucalyptus rudis*) and Salmon Gum *(Eucalyptus salmonophloia*) woodland. The area falls with the Walebing Vegetation System (Avon Botanical District) (Beard, 1979). The landscape is described as undulating and hilly with open Wandoo woodlands on the summits of hills and upper slopes, merging downslope into York Gum, which in turn merges with Salmon Gum where there are extensive flats. Flooded Gum and Swamp Sheoak (*Casuarina obesa*) appears on creek and river banks, while *Halosarcia* sp. dominated samphire flats occur on many of the valley floors.

E2.5 Land Use

E.14 The Study Area primarily consists of farmland on private property, with minor areas of Crown reserves and Vacant Crown Land. The Moora townsite is located in the western part of the study area. It is dissected by the Mogumber Road/Midlands Highway and the Midlands Railway, with trunk roads leading to Walebing, Miling and Bindi Bindi to the east, Dandaragan to the west, Watheroo and Geraldton to the north and Gingin and Perth to the south.

E.15 The long history of agriculture settlement in the area has meant that the vast majority of the private land has been cleared of native vegetation in favour of crops and pasture.

E3 ENVIRONMENTAL ISSUES

E.16 The following environmental issues were identified for each of the six flood mitigation options:

E3.1 Option 1 – Levee Across the Moore River No. 4 Channel Bifurcation

E.17 Moore River No. 4 Channel is an old flood channel of the Moore River which flows through Moora to the south of Moore River No. 1 channel. An old levee across Moore River No. 4 channel had previously been constructed to prevent overflows from Moore River No. 1 into this channel. During the March 1999 flood event this levee was both overtopped and outflanked and as a consequence was severely damaged. This option proposes that the levee be reconstructed at the same level as the original to prevent nuisance flooding in the Berkshire Valley Road area. Scour protection would be provided to prevent the levee from eroding if overtopped again.

E.18 Moore River No. 4 Channel is fringed by York Gum (*Eucalyptus loxophleba* ssp. *loxophleba*) dominated open woodland with scattered Wandoo (*Eucalyptus wandoo*). Given the existence of the old levee prior to the 1999 floods, there is unlikely to be any major impact on vegetation as a result of the construction of a new levee.

E3.2 Option 2 - Detention Basin at Longpool – 1 km Upstream of Gauging Station

E.19 This option in the upper catchment involves the construction of a 4 m thick wall, 1370 metres in length (including a spillway length of 50 m, which is 7 m above the height of the stream), with a low flow culvert. The basin has been designed to store a volume of approximately 20,800 ML. The proposed basin is dominated by scattered Swamp Sheoak (*Casuarina obesa*), with occasional York Gum, Jam (*Acacia acuminata*), *Hakea preissii* and Paperbark (*Melaleuca rhaphiophylla*). There is little understorey remaining due to heavy grazing pressures on the area from livestock, although intermittent Bluebush (*Maireana* sp.) occurs on the fringes of the area. The highly saline drainage line is fringed by Swamp Sheoak.

E.20 The construction of a detention basin wall at the Longpool site will require the clearing of several Swamp Sheoak trees, and inundation, at peak flow, of an area of 8.5 km^2. Associated with the loss of these trees would be the subsequent removal of avifauna nesting habitats as well as the loss other vertebrate or invertebrate fauna which use the trees as habitats. Inundation will affect some significant York Gum trees and other species intolerant to waterlogging. The drainage line in the vicinity of the basin had, as a result of the 1999 flood events, become severely eroded, and the roots of fringing Sheoaks were exposed and undermined to an extent where they may topple into the drainage line with further erosion. Additionally, the detention of water in the basin may result in the silting of an area upstream of the dam wall as well as possible erosion downstream.

E.21 During a flood event similar to the 1999 event, it is predicted that water would remain within the basin for a period of up to 14 days. Positive short-term benefits of this retention include the creation of a temporary freshwater wetland and associated ecosystem that is likely to attract a variety of waterfowl etc to the area.

E3.3 Option 3 - Detention Basin at Kitchin Bridge

E.22 This option in the upper catchment involves the construction of a 4 m thick wall, 1550 metres in length (including a spillway length of 50 m, which is 8 m above the height of the stream), with a low flow culvert designed to store a volume of approximately 16,200 ML. Vegetation in this location is predominantly of an open woodland of York Gum and Swamp Sheoak with scattered Jam, Wandoo, Flooded Gum and *Hakea preissii*, with a degraded, weed infested understorey. The drainage line (which is relatively saline) is fringed with *Halosarcia* sp. dominated samphire. A sizeable samphire flat occurs approximately a hundred metres to the northeast of the drainage line.

E.23 The construction of a detention basin at this site upstream of Kitchin Bridge will necessitate the clearing of several York Gum trees. Associated with the loss of these trees would be the subsequent loss of nesting habitats for birds and habitats for vertebrate or invertebrate fauna. Should a storm of similar intensity to the 1999 flood events occur, an area of $5.8km^2$ of water will be detained in the basin. Detention of water in the basin will result in a period of inundation, with associated waterlogging, probably resulting in the deaths of all waterlogging intolerant trees to the east of the basin wall. Subsequently, there will be a deleterious impact on vertebrate and invertebrate fauna that utilise the vegetation in this area as habitat. Additionally, there will be some erosion of the drainage line resulting is the exposure of tree roots and probable subsequent deaths. Other than noise result generated during the construction of the wall, avifauna noted from the area (which included Port Lincoln Parrots, Magpies, Magpie Larks and Pink and Grey Galah's) are unlikely to be adversely impacted as a result of this option. The detention of water in the basin may result in the silting of an area upstream of the dam wall as well as possible erosion downstream.

E.24 During a flood event similar to the 1999 event, it is estimated that water would remain within the basin for a period of 10-14 days. Positive short-term benefits of this retention of water include the potential for creation of a temporary freshwater wetland and associated ecosystem that is likely to attract a variety of waterfowl etc to the area.

E3.4 Option 4 – Northern Diversion Drain and Levee

E.25 This option involves the diversion of upstream floodwaters to the north of Moora via the Coonderoo River into the Coonderoo Lakes System. The construction of a levee system has also been designed to reduce flood risk in Moora. The length of the proposed drain and levee is 4500m and has been designed to cater for a 10-15 year ARI. Several low flow culverts through the levee are also proposed

E.26 The drain channel and levee construction will potentially result in the disturbance to and removal of a substantial stand of York Gum trees within the Carrick Street and Ralston Road road reserves. Associated with the loss of these trees would be the subsequent loss of avifauna nesting habitats and other vertebrate or invertebrate fauna which use them as habitats. Additionally vegetation occurring to the west and south of the proposed levee and drain system may suffer periods of inundation and potential waterlogging and death during a flood event, while increased flow velocity along the Coondaroo River may result in bank erosion and subsequent loss of riparian vegetation and associated fauna habitat.

E3.5 Option 5 – Southern Diversion Drain and Levee

E.27 The southern drain and diversion levee option involves the diversion of upstream floodwaters to the south to rejoin the Moore River downstream of Moora. It is a considerably more extensive drain and levee system than the northern diversion option (6,200 m in length).

E.28 Between the high school and Mogumber Road, the proposed diversion drain will be up to 30 metres wide and 3 metres in depth. Along with the removal existing houses along Saleeba and Atbara Roads to allow this option to be constructed, a remnant of high quality Wandoo dominate low woodland with an understorey of *Allocasuarina campestris* heath will be negatively impacted upon. This area is potentially an important refuge for vertebrate fauna and will be detrimentally affected as a result of any clearing of vegetation. Additionally, this option is likely to result in the removal of good quality York Gum woodland in the Barber Street road reserve and several York and Salmon Gum trees where the drain intersects Cooper Road.

E3.6 Option 6 – Widening of the Moore River No. 1 and 2 Main Channels

E.29 This option proposes the widening of Moore River No. 1 Main Channel by 20m and Moore River No.2 channel by 10m commencing at Barber Road through to where both channels join (at Coonderoo River).

E.30 Moore River No. 1 is fringed by a moderately diverse array of flora, particularly in the area adjoining the Causeway crossing on Gardiner Street. This area is vegetated with a tall open woodland dominated by York Gum and Flooded Gum scattered with the occasional Swamp Sheoak, with an understorey consisting of *Melaleuca viminea*, *Acacia saligna,* Jam, *Mesomelaena* sp., *Halosarcia* sp. The fringing vegetation further upstream is less diverse and more degraded as the River runs through land that has been cleared for agriculture, and consists of York and Flooded Gum along with numerous Flooded Gum seedlings. An area on the southern side of the channel near the channel crossing at Barber Road has been used as a refuse site for old car bodies, agricultural machinery and hydrocarbon drums. The vegetation fringing Moore River No. 2 channel is comprised predominantly of York Gum with occasional Wandoo and no native species in the understorey.

E.31 Widening Moore River No. 1 channel by 20 metres and the Moore River No. 2 channel by 10 metres from Barber St through to their junction with the Coonderoo River will result in the removal of the majority of the fringing riverine trees. Disturbance to the refuse/dump site near the Barber St crossing has the potential to release hydrocarbons and other contaminants into the river system.

E4 RANKING ENVIRONMENTAL IMPACT

E.32 Table E4.1 shows the ranking assigned to each structural mitigation option.

Table E4.1 Ranking of Environmental Impacts, Moora Structural Mitigation Options

Score	Option 1	Option 2	Option 3	Option 4	Option 5	Option 6
Impact on Flora and Veget-ation	2	6	8	7	8	9
Impacts on Fauna	2	3	7	6	7	8
Other Impacts (including increased salinity, erosion	2	6	6	5	6	6

(NB. Ranking: 10 = highest environmental impact; 1 = lowest environmental impact. The ranking takes into account any potential positive environmental impacts).

E5 CONCLUSIONS

E.33 Option 6 (Widening of Moore River No. 1 and No.2 Channels) is likely to have the most significant detrimental environmental impact of any of the options, while Option 1 is likely to have the least potential for adverse impact on the environment.

E.34 There may be some short term positive impacts resulting from the implementation of either of Options 2 or 3, including the creation of a temporary freshwater wetland for waterbirds as a result of detention of water within the basins for approximately 10-14 days.

E6 REFERENCES

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Appendix F
DETAILED RESULTS ON THE IMPACT OF STRUCTURAL FLOOD MITIGATION OPTIONS ON FLOOD LEVELS AT MOORA

Table F.1 Reduction in 5 Year ARI Design Flood Levels at the Various Cross-Sections for the Six Flood Mitigation Options, Moora Mike 11 Model

Branch Name	Branch Chainage (km)	Existing Design Flood Level (m AHD)	Reduction in 5 Year ARI Design Flood Level (m AHD)					
			Option 1	Option 2	Option 3	Option 4	Option 5	Option 6
MOORE 1	0.00	207.60	0.00	-0.27	-0.15	0.00	0.00	-0.36
MOORE 1	0.32	207.35	0.00	-0.28	-0.15	0.01	0.00	-0.48
MOORE 1	0.44	207.15	0.00	-0.28	-0.15	0.01	0.00	-0.46
MOORE 1	0.86	206.51	0.01	-0.28	-0.14	0.03	0.00	-0.30
MOORE 1	1.33	206.02	0.01	-0.27	-0.13	0.07	0.01	-0.08
MOORE 1	1.58	205.82	0.02	-0.26	-0.12	0.10	0.01	-0.12
MOORE 1	1.87	205.61	0.04	-0.28	-0.12	0.17	0.01	-0.09
MOORE 1	1.92	205.58	0.04	-0.29	-0.12	0.18	0.01	-0.07
MOORE 1	2.59	204.82	0.13	-0.39	-0.04	0.69	0.06	-0.59
MOORE 1	2.63	204.85	0.12	-0.41	-0.05	0.66	0.06	-0.64
MOORE 1	2.76	204.74	0.11	-0.44	-0.05	0.72	0.09	-0.71
MOORE 1	3.18	204.40	0.09	-0.47	-0.05	-0.21	0.22	-0.95
MOORE 1	3.60	203.85	0.10	-0.30	-0.06	-0.22	0.67	-0.83
MOORE 1	3.72	203.71	0.11	-0.23	-0.06	-0.21	-0.25	-0.78
MOORE 1	3.93	203.50	0.10	-0.24	-0.06	-0.21	-0.25	-0.78
MOORE 1	4.19	203.09	0.09	-0.23	-0.06	-0.21	-0.25	-0.58
MOORE 1	4.48	202.73	0.08	-0.21	-0.07	-0.19	-0.23	-0.41
MOORE 1	4.49	202.74	0.09	-0.22	-0.07	-0.20	-0.23	-0.47
MOORE 1	4.54	202.70	0.09	-0.22	-0.07	-0.20	-0.24	-0.48
MOORE 1	4.72	202.62	0.08	-0.22	-0.08	-0.20	-0.23	-0.53
MOORE 1	4.86	202.57	0.08	-0.21	-0.07	-0.19	-0.23	-0.53
MOORE 1	4.89	202.51	0.08	-0.21	-0.07	-0.19	-0.23	-0.57
MOORE 1	5.06	202.33	0.07	-0.18	-0.07	-0.16	-0.20	-0.53
MOORE 1	5.30	202.07	0.08	-0.20	-0.08	-0.17	-0.22	-0.45
MOORE 1	5.55	201.83	0.08	-0.23	-0.09	-0.18	-0.25	-0.35
MOORE 1	5.57	201.83	0.08	-0.23	-0.09	-0.18	-0.25	-0.34
MOORE 1	5.60	201.76	0.08	-0.23	-0.09	-0.17	-0.24	-0.37
MOORE 1	5.78	201.67	0.08	-0.22	-0.09	-0.16	-0.24	-0.35
MOORE 1	6.08	201.45	0.07	-0.19	-0.08	-0.11	-0.20	-0.27
MOORE 1	6.56	201.02	0.03	-0.11	-0.04	0.03	-0.12	-0.09
MOORE 1	6.69	200.95	0.03	-0.11	-0.04	0.05	-0.13	-0.07
MOORE 1	6.97	200.82	0.03	-0.12	-0.04	0.05	-0.13	-0.04
MOORE 1	7.17	200.63	0.02	-0.08	-0.03	0.04	-0.09	0.04
MOORE 1	7.34	200.56	0.02	-0.07	-0.02	0.04	-0.07	0.06
MOORE 1	7.89	200.24	0.01	-0.07	-0.02	0.03	-0.07	0.05
MOORE 1	8.46	199.74	0.02	-0.13	-0.03	0.05	-0.14	0.08
MOORE 1	9.01	199.31	0.03	-0.14	-0.04	0.06	-0.14	0.09
MOORE 1	9.81	198.62	0.02	-0.07	-0.02	0.04	0.02	0.05
MOORE 1	9.91	198.59	0.02	-0.07	-0.02	0.04	0.04	0.05
MOORE 1	10.11	198.41	0.02	-0.07	-0.02	0.04	0.04	0.05
MOORE 1	10.66	197.85	0.02	-0.06	-0.02	0.03	0.03	0.05
YADGENA BROOK	0.00	207.27	0.00	0.00	0.00	0.00	0.00	0.00
YADGENA BROOK	0.31	206.49	0.01	0.01	0.01	0.01	0.01	0.01
YADGENA BROOK	0.36	206.31	0.03	0.03	0.03	0.03	0.03	0.03
YADGENA BROOK	0.76	205.14	0.07	0.07	0.07	0.07	0.07	0.07
YADGENA BROOK	1.16	204.01	0.07	0.07	0.07	0.07	0.07	0.07
YADGENA BROOK	1.74	202.38	0.05	0.06	0.06	0.05	0.05	0.05
YADGENA BROOK	1.81	202.09	0.05	0.05	0.05	0.05	0.05	0.05
YADGENA BROOK	2.20	201.96	0.06	0.06	0.06	0.06	0.06	0.06
YADGENA BROOK	2.56	201.20	0.05	0.06	0.06	0.05	0.05	0.05
YADGENA BROOK	3.02	200.57	0.06	0.06	0.06	0.06	0.06	0.06
YADGENA BROOK	3.87	199.73	0.06	0.06	0.06	0.06	0.06	0.06
YADGENA BROOK	4.53	199.14	0.02	0.04	0.04	0.02	0.02	0.01
YADGENA BROOK	4.87	198.59	0.02	-0.07	-0.02	0.04	0.04	0.05
COONDEROO	0.00	201.04	0.02	-0.12	-0.04	0.15	-0.14	-0.11
COONDEROO	0.44	201.04	0.02	-0.12	-0.04	0.15	-0.14	-0.11
COONDEROO	0.80	201.04	0.02	-0.12	-0.05	0.15	-0.14	-0.11
COONDEROO	0.81	201.04	0.02	-0.12	-0.05	0.15	-0.14	-0.11

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Table F.1 Reduction in 5 Year ARI Design Flood Levels at the Various Cross-Sections for the Six Flood Mitigation Options, Moora Mike 11 Model - Cont'd

Branch Name	Branch Chainage (km)	Existing Design Flood Level (m AHD)	Reduction in 5 Year ARI Design Flood Level (m AHD)					
			Option 1	Option 2	Option 3	Option 4	Option 5	Option 6
MOORE 5	0.00	200.63	0.02	-0.08	-0.03	0.04	-0.09	0.04
MOORE 5	0.26	200.25	0.02	-0.07	-0.03	0.04	-0.07	0.03
MOORE 5	0.45	200.04	0.01	-0.07	-0.03	0.03	-0.05	0.03
MOORE 5	0.89	199.74	0.01	-0.06	-0.02	0.02	0.02	0.02
MOORE 5	1.43	199.60	0.00	-0.04	-0.02	0.01	0.09	0.01
MOORE 5	1.91	199.33	0.01	-0.04	-0.01	0.01	0.07	0.02
MOORE 5	2.67	198.59	0.02	-0.07	-0.02	0.04	0.04	0.05
MOORE 2	0.00	207.35	0.00	-0.28	-0.15	0.01	0.00	-0.48
MOORE 2	0.57	206.86	0.00	-0.21	-0.11	0.00	0.00	-0.70
MOORE 2	0.96	206.48	0.00	-0.21	-0.10	0.00	0.00	-0.67
MOORE 2	1.30	206.08	0.00	-0.29	-0.13	0.00	0.01	-0.69
MOORE 2	1.63	205.62	0.00	-0.25	-0.11	0.01	0.06	-0.34
MOORE 2	2.01	205.31	0.00	-0.23	-0.10	0.03	0.14	-0.62
MOORE 2	2.12	205.28	0.00	-0.23	-0.10	0.03	0.15	-0.98
MOORE 2	2.24	205.24	0.00	-0.22	-0.09	0.04	0.13	-1.13
MOORE 2	2.99	204.66	0.00	-0.21	-0.09	0.49	0.35	-1.13
MOORE 2	3.42	204.05	0.00	-0.18	-0.09	-0.21	0.85	-0.77
MOORE 2	3.72	203.76	0.00	-0.17	-0.08	-0.20	-0.22	-0.58
MOORE 2	3.97	203.63	0.00	-0.23	-0.10	-0.27	-0.30	-0.55
MOORE 2	4.21	203.57	0.00	-0.28	-0.12	-0.33	-0.37	-0.52
MOORE 2	4.25	203.11	-0.01	-0.27	-0.15	-0.28	-0.34	-0.81
MOORE 2	4.40	203.07	-0.01	-0.26	-0.15	-0.29	-0.34	-0.81
MOORE 2	4.58	202.96	-0.02	-0.28	-0.19	-0.30	-0.33	-0.76
MOORE 2	4.60	202.75	-0.01	-0.15	-0.07	-0.17	-0.20	-0.64
MOORE 2	4.76	202.55	0.00	-0.15	-0.06	-0.16	-0.19	-0.62
MOORE 2	5.18	201.74	0.01	-0.19	-0.09	-0.18	-0.25	-0.63
MOORE 2	5.35	201.51	0.01	-0.18	-0.08	-0.12	-0.23	-0.49
MOORE 2	5.73	201.39	0.01	-0.18	-0.08	-0.08	-0.23	-0.39
MOORE 2	6.02	201.21	0.02	-0.15	-0.06	0.01	-0.19	-0.26
MOORE 2	6.31	201.05	0.02	-0.13	-0.05	0.11	-0.15	-0.12
MOORE 2	6.48	201.03	0.02	-0.12	-0.05	0.12	-0.14	-0.11
MOORE 2	6.65	201.02	0.02	-0.12	-0.05	0.11	-0.14	-0.10
MOORE 2	6.70	200.97	0.02	-0.11	-0.04	0.09	-0.13	-0.09
MOORE 2	7.15	200.95	0.03	-0.11	-0.04	0.05	-0.13	-0.07
MOORE 3	0.00	205.28	0.00	-0.23	-0.10	0.03	0.15	-
MOORE 3	0.59	204.44	0.00	-0.06	-0.02	0.03	-	-
MOORE 3	0.83	204.25	0.00	-0.12	-0.05	0.17	-	-
MOORE 3	1.13	203.85	0.00	-0.35	-0.16	-0.30	-	-
MOORE 3	1.18	203.72	0.00	-0.39	-0.15	-0.24	-	-
MOORE 3	1.77	202.04	0.00	-0.72	-0.25	0.97	-	-
MOORE 3	2.22	201.96	0.00	-0.68	-0.23	0.58	-	-
MOORE 3	2.60	201.74	0.00	-0.69	-0.21	0.58	-	-
MOORE 3	2.87	201.30	0.00	-0.34	-0.11	0.86	-	-
MOORE 3	3.34	201.06	0.02	-0.14	-0.05	0.65	-	-0.14
MOORE 3	3.65	201.04	0.02	-0.12	-0.05	0.15	-0.14	-0.11
MOORE 3	3.91	201.03	0.02	-0.12	-0.05	0.12	-0.14	-0.11
M2-M1	0.00	206.86	0.00	-0.21	-0.11	0.00	0.00	-
M2-M1	0.14	206.73	0.00	-0.24	-0.12	0.01	0.00	-
M2-M1	0.51	206.16	0.00	-0.53	-0.30	0.03	0.00	-
M2-M1	0.83	205.89	0.01	-0.49	-0.26	0.08	0.01	-
M2-M1	1.20	205.59	0.04	-0.29	-0.12	0.18	0.01	-0.08
M2-M1	1.23	205.58	0.04	-0.29	-0.12	0.18	0.01	-0.07
ISBISTER	0.00	206.16	0.00	-0.53	-0.30	0.03	0.00	-
ISBISTER	0.40	205.71	0.01	-0.57	-0.23	0.03	0.01	-
ISBISTER	0.71	205.48	0.00	-0.81	-0.32	0.13	0.05	-
ISBISTER	1.11	204.65	0.00	-0.47	-0.21	0.71	0.35	-
ISBISTER	1.41	204.15	0.00	-0.13	-0.03	1.20	0.66	-
ISBISTER	1.76	204.13	0.00	-0.12	-0.03	-	0.69	-
ISBISTER	1.93	204.09	0.00	-0.10	-0.02	-	0.72	-

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Table F.1 Reduction in 5 Year ARI Design Flood Levels at the Various Cross-Sections for the Six Flood Mitigation Options, Moora Mike 11 Model - Cont'd

Branch Name	Branch Chainage (km)	Existing Design Flood Level (m AHD)	Reduction in 5 Year ARI Design Flood Level (m AHD)					
			Option 1	Option 2	Option 3	Option 4	Option 5	Option 6
ISBISTER	2.24	203.96	0.00	-0.04	-0.01	-	-	-
ISBISTER	2.48	203.91	0.00	-0.02	-0.01	-	-	-
ISBISTER	2.73	203.91	0.00	-0.02	-0.01	-	-	-
ISBISTER	3.06	-	-	-	-	-	-	-
ISBISTER	3.26	-	-	-	-	-	-	-
ISBISTER	3.48	-	-	-	-	-	-	-
ISBISTER	4.00	-	-	-	-	-	-	-
ISBISTER	4.34	-	-	-	-	-	-	-
ISBISTER	4.85	-	-	-	-	-	-	-
MOORE 4	0.00	204.85	-	-	-	0.66	0.06	-
MOORE 4	0.13	204.81	-	-	-	0.69	0.03	-
MOORE 4	0.51	204.40	-	-	-	-	0.22	-
MOORE 4	0.95	203.84	-	-	-	-	-	-
MOORE 4	0.97	203.84	-	-	-	-	-	-
MOORE 4	1.61	203.80	-	-	-	-	-	-
MOORE 4	1.84	203.61	-	-	-	-	-	-
MOORE 4	2.02	203.38	-	-	-	-	-	-
MOORE 4	2.13	203.31	-	-	-	-	-	-
MOORE 4	2.36	203.03	-	-	-	-	-	-
MOORE 4	2.37	203.02	-	-	-	-	-	-
MOORE 4	2.96	201.79	-	-	-	-	-	-
MOORE 4	3.16	201.10	-	-	-	-	-	-
MOORE 4	3.58	200.38	-	-	-	-	-	-
MOORE 4	3.95	199.74	-	-	-	-	-	-
CLINCH ST	0.00	203.03	-	-	-	-	-	-
CLINCH ST	0.37	202.78	-	-	-	-	-	-
CLINCH ST	0.60	202.16	-	-	-	-	-	-
CLINCH ST	0.79	201.41	-	-	-	-	-	-
CLINCH ST	1.27	200.94	-	-	-	-	-	-
CLINCH ST	1.53	200.41	-	-	-	-	-	-
CLINCH ST	1.57	200.25	-	-	-	-	-	-
CAMERON ST	0.00	203.84	-	-	-	-	-	-
CAMERON ST	0.33	203.67	-	-	-	-	-	-
CAMERON ST	0.58	202.96	-	-	-	-	-	-
CAMERON ST	0.71	202.73	-	-	-	-	-	-
M1M4-1	0.00	204.40	-	-	-	-	-	-
M1M4-1	0.10	204.40	-	-	-	-	-	-
WALEBING RD	0.00	-	-	-	-	-	-	-
WALEBING RD	0.36	-	-	-	-	-	-	-
WALEBING RD	0.73	-	-	-	-	-	-	-
WALEBING RD	1.28	-	-	-	-	-	-	-
WALEBING RD	1.91	-	-	-	-	-	-	-
WALEBING RD	2.22	-	-	-	-	-	-	-
WALEBING RD	2.66	-	-	-	-	-	-	-
BINDOON RD	0.00	-	-	-	-	-	-	-
BINDOON RD	0.45	-	-	-	-	-	-	-
BINDOON RD	0.88	-	-	-	-	-	-	-
BINDOON RD	1.45	-	-	-	-	-	-	-
BINDOON RD	1.77	-	-	-	-	-	-	-
BINDOON RD	2.21	-	-	-	-	-	-	-

'-'Denotes no flooding occurs

Table F.2 Reduction in 10 Year ARI Design Flood Levels at the Cross-Sections for the Six Flood Mitigation Options, Moora Mike 11 Model

Branch Name	Branch Chainage (km)	Existing Design Flood Level (m AHD)	Reduction in 10 Year ARI Design Flood Level (m AHD)					
			Option 1	Option 2	Option 3	Option 4	Option 5	Option 6
MOORE 1	0.00	207.81	0.00	-0.34	-0.21	0.00	0.00	-0.33
MOORE 1	0.32	207.57	0.00	-0.36	-0.23	0.00	0.00	-0.46
MOORE 1	0.44	207.38	0.00	-0.37	-0.24	0.00	0.00	-0.45
MOORE 1	0.86	206.74	0.01	-0.36	-0.23	0.02	0.00	-0.28
MOORE 1	1.33	206.22	0.02	-0.32	-0.19	0.04	0.01	-0.06
MOORE 1	1.58	206.00	0.03	-0.29	-0.16	0.07	0.01	-0.08
MOORE 1	1.87	205.80	0.06	-0.30	-0.16	0.11	0.02	-0.07
MOORE 1	1.92	205.77	0.06	-0.30	-0.16	0.12	0.02	-0.06
MOORE 1	2.59	205.05	0.27	-0.25	-0.10	0.56	0.09	-0.42
MOORE 1	2.63	205.06	0.25	-0.25	-0.10	0.55	0.08	-0.41
MOORE 1	2.76	204.94	0.22	-0.24	-0.09	0.63	0.12	-0.48
MOORE 1	3.18	204.56	0.07	-0.20	-0.08	-0.32	0.27	-0.81
MOORE 1	3.60	204.07	0.10	-0.27	-0.12	-0.41	0.69	-0.77
MOORE 1	3.72	203.94	0.11	-0.29	-0.13	-0.43	-0.41	-0.72
MOORE 1	3.93	203.73	0.10	-0.28	-0.13	-0.43	-0.40	-0.72
MOORE 1	4.19	203.32	0.09	-0.27	-0.14	-0.42	-0.40	-0.52
MOORE 1	4.48	202.97	0.09	-0.29	-0.15	-0.41	-0.40	-0.36
MOORE 1	4.49	202.98	0.09	-0.29	-0.15	-0.41	-0.40	-0.40
MOORE 1	4.54	202.95	0.09	-0.30	-0.15	-0.43	-0.42	-0.42
MOORE 1	4.72	202.86	0.09	-0.29	-0.15	-0.41	-0.40	-0.49
MOORE 1	4.86	202.79	0.08	-0.28	-0.14	-0.39	-0.38	-0.47
MOORE 1	4.89	202.73	0.08	-0.27	-0.13	-0.39	-0.38	-0.50
MOORE 1	5.06	202.53	0.07	-0.25	-0.12	-0.34	-0.34	-0.46
MOORE 1	5.30	202.28	0.06	-0.26	-0.13	-0.33	-0.36	-0.38
MOORE 1	5.55	202.06	0.05	-0.29	-0.14	-0.34	-0.41	-0.31
MOORE 1	5.57	202.06	0.05	-0.29	-0.14	-0.33	-0.41	-0.31
MOORE 1	5.60	201.98	0.04	-0.28	-0.13	-0.32	-0.39	-0.33
MOORE 1	5.78	201.88	0.03	-0.27	-0.13	-0.30	-0.39	-0.32
MOORE 1	6.08	201.64	0.03	-0.24	-0.11	-0.21	-0.33	-0.24
MOORE 1	6.56	201.17	0.02	-0.14	-0.07	0.02	-0.21	-0.08
MOORE 1	6.69	201.10	0.02	-0.12	-0.06	0.07	-0.19	-0.06
MOORE 1	6.97	200.95	0.02	-0.11	-0.05	0.05	-0.19	-0.04
MOORE 1	7.17	200.76	0.02	-0.11	-0.06	0.06	-0.16	0.04
MOORE 1	7.34	200.67	0.02	-0.11	-0.06	0.06	-0.15	0.06
MOORE 1	7.89	200.34	0.01	-0.09	-0.04	0.05	-0.13	0.05
MOORE 1	8.46	199.87	0.02	-0.11	-0.05	0.05	-0.17	0.05
MOORE 1	9.01	199.46	0.01	-0.12	-0.05	0.05	-0.17	0.05
MOORE 1	9.81	198.75	0.00	-0.12	-0.07	0.04	0.01	0.03
MOORE 1	9.91	198.71	0.00	-0.11	-0.07	0.03	0.03	0.03
MOORE 1	10.11	198.55	0.00	-0.12	-0.07	0.04	0.03	0.03
MOORE 1	10.66	198.00	0.00	-0.14	-0.09	0.04	0.03	0.03
YADGENA BROOK	0.00	207.59	0.00	0.00	0.00	0.00	0.00	0.00
YADGENA BROOK	0.31	206.72	0.01	0.01	0.01	0.01	0.01	0.01
YADGENA BROOK	0.36	206.51	0.03	0.03	0.03	0.03	0.03	0.03
YADGENA BROOK	0.76	205.58	0.04	0.04	0.04	0.04	0.04	0.04
YADGENA BROOK	1.16	204.45	0.06	0.06	0.06	0.06	0.06	0.06
YADGENA BROOK	1.74	202.71	0.05	0.05	0.05	0.05	0.05	0.05
YADGENA BROOK	1.81	202.41	0.04	0.03	0.03	0.03	0.03	0.04
YADGENA BROOK	2.20	202.29	0.04	0.04	0.04	0.04	0.04	0.04
YADGENA BROOK	2.56	201.52	0.04	0.04	0.04	0.04	0.04	0.04
YADGENA BROOK	3.02	200.88	0.03	0.03	0.03	0.03	0.03	0.03
YADGENA BROOK	3.87	200.05	0.01	0.02	0.02	0.01	0.01	0.01
YADGENA BROOK	4.53	199.23	0.00	0.03	0.03	0.00	0.00	0.01
YADGENA BROOK	4.87	198.71	0.00	-0.11	-0.07	0.03	0.03	0.03
COONDEROO	0.00	201.22	0.01	-0.14	-0.07	0.31	-0.23	-0.10
COONDEROO	0.44	201.22	0.01	-0.14	-0.07	0.31	-0.23	-0.10
COONDEROO	0.80	201.22	0.01	-0.14	-0.07	0.31	-0.23	-0.10
COONDEROO	0.81	201.22	0.01	-0.14	-0.07	0.31	-0.23	-0.10

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Table F.2 Reduction in 10 Year ARI Design Flood Levels at the Various Cross-Sections for the Six Flood Mitigation Options, Moora Mike 11 Model - Cont'd

Branch Name	Branch Chainage (km)	Existing Design Flood Level (m AHD)	Reduction in 10 Year ARI Design Flood Level (m AHD)					
			Option 1	Option 2	Option 3	Option 4	Option 5	Option 6
MOORE 5	0.00	200.76	0.02	-0.11	-0.06	0.06	-0.16	0.04
MOORE 5	0.26	200.40	-0.02	-0.14	-0.09	0.02	-0.16	-0.01
MOORE 5	0.45	200.19	-0.02	-0.13	-0.09	0.02	-0.11	-0.01
MOORE 5	0.89	199.88	-0.03	-0.13	-0.10	0.00	0.00	-0.03
MOORE 5	1.43	199.70	-0.03	-0.10	-0.07	0.00	0.14	-0.02
MOORE 5	1.91	199.42	-0.02	-0.09	-0.06	0.00	0.11	-0.01
MOORE 5	2.67	198.71	0.00	-0.11	-0.07	0.03	0.03	0.03
MOORE 2	0.00	207.57	0.00	-0.36	-0.23	0.00	0.00	-0.46
MOORE 2	0.57	207.04	0.00	-0.27	-0.18	0.00	0.00	-0.46
MOORE 2	0.96	206.66	0.00	-0.27	-0.18	0.00	0.00	-0.49
MOORE 2	1.30	206.28	0.00	-0.31	-0.20	0.00	0.01	-0.56
MOORE 2	1.63	205.81	0.00	-0.29	-0.19	0.02	0.05	-0.30
MOORE 2	2.01	205.47	0.00	-0.25	-0.16	0.06	0.14	-0.53
MOORE 2	2.12	205.42	0.00	-0.23	-0.15	0.07	0.16	-0.71
MOORE 2	2.24	205.38	0.00	-0.22	-0.14	0.09	0.15	-0.75
MOORE 2	2.99	204.78	0.00	-0.20	-0.12	0.55	0.34	-0.74
MOORE 2	3.42	204.19	0.00	-0.22	-0.14	-0.32	0.79	-0.43
MOORE 2	3.72	203.87	0.00	-0.18	-0.11	-0.28	-0.31	-0.19
MOORE 2	3.97	203.72	0.00	-0.18	-0.10	-0.31	-0.36	-0.11
MOORE 2	4.21	203.65	0.00	-0.18	-0.09	-0.35	-0.41	-0.06
MOORE 2	4.25	203.33	0.00	-0.36	-0.27	-0.45	-0.54	-0.50
MOORE 2	4.40	203.28	0.00	-0.35	-0.26	-0.46	-0.53	-0.51
MOORE 2	4.58	203.23	0.00	-0.44	-0.35	-0.53	-0.57	-0.53
MOORE 2	4.60	202.87	-0.01	-0.18	-0.11	-0.26	-0.29	-0.37
MOORE 2	4.76	202.67	0.00	-0.17	-0.11	-0.23	-0.29	-0.38
MOORE 2	5.18	201.94	0.00	-0.26	-0.17	-0.23	-0.41	-0.46
MOORE 2	5.35	201.70	0.00	-0.24	-0.16	-0.12	-0.38	-0.35
MOORE 2	5.73	201.57	0.00	-0.22	-0.14	-0.04	-0.35	-0.28
MOORE 2	6.02	201.39	0.01	-0.18	-0.10	0.10	-0.29	-0.19
MOORE 2	6.31	201.22	0.02	-0.14	-0.07	0.23	-0.23	-0.10
MOORE 2	6.48	201.21	0.01	-0.14	-0.07	0.25	-0.23	-0.09
MOORE 2	6.65	201.20	0.02	-0.13	-0.06	0.22	-0.22	-0.09
MOORE 2	6.70	201.12	0.02	-0.12	-0.06	0.16	-0.20	-0.07
MOORE 2	7.15	201.10	0.02	-0.12	-0.06	0.07	-0.19	-0.06
MOORE 3	0.00	205.42	0.00	-0.23	-0.15	0.07	0.16	-0.71
MOORE 3	0.59	204.51	0.00	-0.10	-0.08	0.14	-3.64	-0.52
MOORE 3	0.83	204.35	0.00	-0.15	-0.10	0.23	-4.24	-0.68
MOORE 3	1.13	204.11	0.00	-0.41	-0.27	-0.28	-4.11	-0.78
MOORE 3	1.18	203.80	0.00	-0.21	-0.08	-0.03	-3.80	-0.70
MOORE 3	1.77	202.68	0.00	-0.52	-0.28	0.51	-1.69	-1.83
MOORE 3	2.22	202.30	0.00	-0.25	-0.08	0.41	-1.30	-1.44
MOORE 3	2.60	202.15	0.00	-0.30	-0.04	0.35	-1.16	-1.26
MOORE 3	2.87	201.98	0.00	-0.41	-0.05	0.32	-0.99	-0.93
MOORE 3	3.34	201.44	0.01	-0.29	-0.11	0.39	-0.45	-0.33
MOORE 3	3.65	201.22	0.01	-0.14	-0.07	0.31	-0.23	-0.10
MOORE 3	3.91	201.21	0.01	-0.14	-0.07	0.25	-0.23	-0.09
M2-M1	0.00	207.04	0.00	-0.27	-0.18	0.00	0.00	-
M2-M1	0.14	206.91	0.00	-0.30	-0.19	0.00	0.00	-
M2-M1	0.51	206.44	0.00	-0.55	-0.28	-0.01	0.00	-
M2-M1	0.83	206.20	0.01	-0.54	-0.29	0.02	0.00	-
M2-M1	1.20	205.78	0.06	-0.30	-0.16	0.12	0.02	-0.06
M2-M1	1.23	205.77	0.06	-0.29	-0.16	0.12	0.02	-0.06
ISBISTER	0.00	206.44	0.00	-0.55	-0.28	-0.01	0.00	-
ISBISTER	0.40	205.81	-0.01	-0.30	-0.07	-0.03	-0.01	-
ISBISTER	0.71	205.61	0.00	-0.39	-0.10	0.06	0.00	-
ISBISTER	1.11	204.99	-0.04	-0.53	-0.33	0.51	0.03	-
ISBISTER	1.41	204.20	0.00	-0.07	-0.04	1.29	0.71	-
ISBISTER	1.76	204.17	0.00	-0.06	-0.04	-	0.74	-
ISBISTER	1.93	204.12	0.00	-0.05	-0.03	-	0.79	-

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Table F.2 Reduction in 10 Year ARI Design Flood Levels at the Various Cross-Sections for the Six Flood Mitigation Options, Moora Mike 11 Model - Cont'd

Branch Name	Branch Chainage (km)	Existing Design Flood Level (m AHD)	Reduction in 10 Year ARI Design Flood Level (m AHD)					
			Option 1	Option 2	Option 3	Option 4	Option 5	Option 6
ISBISTER	2.24	203.97	0.00	-0.02	-0.01	-	-	-
ISBISTER	2.48	203.92	0.00	-0.02	-0.01	-	-	-
ISBISTER	2.73	203.92	0.00	-0.02	-0.01	-	-	-
ISBISTER	3.06	-	-	-	-	-	-	-
ISBISTER	3.26	-	-	-	-	-	-	-
ISBISTER	3.48	-	-	-	-	-	-	-
ISBISTER	4.00	-	-	-	-	-	-	-
ISBISTER	4.34	-	-	-	-	-	-	-
ISBISTER	4.85	-	-	-	-	-	-	-
MOORE 4	0.00	205.06	-	-	-	0.55	0.08	-
MOORE 4	0.13	204.99	-	-	-	0.62	0.04	-
MOORE 4	0.51	204.56	-	-	-	-2.75	0.26	-
MOORE 4	0.95	203.95	-	-	-	-	-	-
MOORE 4	0.97	203.95	-	-	-	-	-	-
MOORE 4	1.61	203.87	-	-	-	-	-	-
MOORE 4	1.84	203.71	-	-	-	-	-	-
MOORE 4	2.02	203.50	-	-	-	-	-	-
MOORE 4	2.13	203.40	-	-	-	-	-	-
MOORE 4	2.36	203.12	-	-	-	-	-	-
MOORE 4	2.37	203.11	-	-	-	-	-	-
MOORE 4	2.96	201.98	-	-	-	-	-	-
MOORE 4	3.16	201.55	-	-	-	-	-	-
MOORE 4	3.58	200.93	-	-	-	-	-	-
MOORE 4	3.95	199.88	-	-	-	-	-	-
CLINCH ST	0.00	203.12	-	-	-	-	-	-
CLINCH ST	0.37	202.85	-	-	-	-	-	-
CLINCH ST	0.60	202.30	-	-	-	-	-	-
CLINCH ST	0.79	201.66	-	-	-	-	-	-
CLINCH ST	1.27	201.05	-	-	-	-	-	-
CLINCH ST	1.53	200.58	-	-	-	-	-	-
CLINCH ST	1.57	200.40	-	-	-	-	-	-
CAMERON ST	0.00	203.95	-	-	-	-	-	-
CAMERON ST	0.33	203.69	-	-	-	-	-	-
CAMERON ST	0.58	203.18	-	-	-	-	-	-
CAMERON ST	0.71	202.97	-	-	-	-	-	-
M1M4-1	0.00	204.56	-	-	-	-	-	-
M1M4-1	0.10	204.56	-	-	-	-	-	-
WALEBING RD	0.00	-	-	-	-	-	-	-
WALEBING RD	0.36	-	-	-	-	-	-	-
WALEBING RD	0.73	-	-	-	-	-	-	-
WALEBING RD	1.28	-	-	-	-	-	-	-
WALEBING RD	1.91	-	-	-	-	-	-	-
WALEBING RD	2.22	-	-	-	-	-	-	-
WALEBING RD	2.66	-	-	-	-	-	-	-
BINDOON RD	0.00	-	-	-	-	-	-	-
BINDOON RD	0.45	-	-	-	-	-	-	-
BINDOON RD	0.88	-	-	-	-	-	-	-
BINDOON RD	1.45	-	-	-	-	-	-	-
BINDOON RD	1.77	-	-	-	-	-	-	-
BINDOON RD	2.21	-	-	-	-	-	-	-

'-'Denotes no flooding occurs

Table F.3 Reduction in 20 Year ARI Design Flood Levels at the Various Cross-Sections for the Six Flood Mitigation Options, Moora Mike 11 Model

Branch Name	Branch Chainage (km)	Existing Design Flood Level (m AHD)	Reduction in 20 Year ARI Design Flood Level (m AHD)					
			Option 1	Option 2	Option 3	Option 4	Option 5	Option 6
MOORE 1	0.00	208.02	0.00	-0.39	-0.31	0.00	0.00	-0.21
MOORE 1	0.32	207.81	0.00	-0.43	-0.35	0.00	0.00	-0.36
MOORE 1	0.44	207.66	0.00	-0.48	-0.39	0.00	0.00	-0.39
MOORE 1	0.86	207.00	0.00	-0.45	-0.36	0.00	0.00	-0.22
MOORE 1	1.33	206.46	0.00	-0.40	-0.33	0.01	0.00	0.00
MOORE 1	1.58	206.22	0.01	-0.35	-0.28	0.03	0.00	0.03
MOORE 1	1.87	206.02	0.01	-0.35	-0.27	0.05	0.00	0.04
MOORE 1	1.92	206.00	0.01	-0.35	-0.28	0.06	0.00	0.05
MOORE 1	2.59	205.41	0.09	-0.42	-0.32	0.33	0.03	-0.03
MOORE 1	2.63	205.40	0.09	-0.39	-0.29	0.34	0.03	-0.02
MOORE 1	2.76	205.25	0.07	-0.37	-0.28	0.45	0.07	-0.06
MOORE 1	3.18	204.71	0.02	-0.20	-0.15	-0.44	0.32	-0.45
MOORE 1	3.60	204.29	0.03	-0.30	-0.21	-0.59	0.69	-0.63
MOORE 1	3.72	204.19	0.03	-0.33	-0.23	-0.63	-0.58	-0.62
MOORE 1	3.93	203.97	0.04	-0.33	-0.22	-0.62	-0.57	-0.61
MOORE 1	4.19	203.59	0.08	-0.37	-0.27	-0.66	-0.61	-0.45
MOORE 1	4.48	203.28	0.08	-0.41	-0.31	-0.68	-0.65	-0.31
MOORE 1	4.49	203.28	0.08	-0.41	-0.30	-0.67	-0.64	-0.33
MOORE 1	4.54	203.27	0.08	-0.42	-0.32	-0.70	-0.67	-0.34
MOORE 1	4.72	203.15	0.06	-0.39	-0.29	-0.66	-0.63	-0.45
MOORE 1	4.86	203.06	0.06	-0.36	-0.27	-0.62	-0.60	-0.42
MOORE 1	4.89	202.99	0.06	-0.36	-0.26	-0.61	-0.59	-0.44
MOORE 1	5.06	202.77	0.05	-0.32	-0.24	-0.53	-0.52	-0.39
MOORE 1	5.30	202.50	0.04	-0.32	-0.24	-0.50	-0.54	-0.31
MOORE 1	5.55	202.30	0.04	-0.34	-0.26	-0.49	-0.59	-0.26
MOORE 1	5.57	202.29	0.03	-0.34	-0.26	-0.49	-0.58	-0.25
MOORE 1	5.60	202.20	0.03	-0.32	-0.24	-0.46	-0.56	-0.28
MOORE 1	5.78	202.10	0.02	-0.31	-0.24	-0.43	-0.55	-0.27
MOORE 1	6.08	201.82	0.02	-0.26	-0.20	-0.30	-0.47	-0.20
MOORE 1	6.56	201.25	0.02	-0.13	-0.08	0.05	-0.27	-0.03
MOORE 1	6.69	201.18	0.01	-0.12	-0.07	0.11	-0.26	-0.02
MOORE 1	6.97	201.02	0.01	-0.09	-0.06	0.09	-0.23	0.01
MOORE 1	7.17	200.85	0.01	-0.12	-0.08	0.09	-0.23	0.06
MOORE 1	7.34	200.76	0.01	-0.12	-0.08	0.08	-0.22	0.07
MOORE 1	7.89	200.41	0.01	-0.10	-0.06	0.07	-0.18	0.06
MOORE 1	8.46	199.95	0.01	-0.11	-0.07	0.08	-0.22	0.07
MOORE 1	9.01	199.54	0.00	-0.11	-0.08	0.07	-0.21	0.06
MOORE 1	9.81	198.86	-0.01	-0.15	-0.11	0.04	0.00	0.02
MOORE 1	9.91	198.83	-0.01	-0.16	-0.12	0.04	0.03	0.01
MOORE 1	10.11	198.66	-0.01	-0.16	-0.12	0.03	0.03	0.01
MOORE 1	10.66	198.09	-0.01	-0.16	-0.11	0.03	0.02	0.01
YADGENA BROOK	0.00	207.80	0.00	0.00	0.00	0.00	0.00	0.00
YADGENA BROOK	0.31	206.93	0.01	0.01	0.01	0.01	0.01	0.01
YADGENA BROOK	0.36	206.73	0.03	0.03	0.03	0.03	0.03	0.03
YADGENA BROOK	0.76	205.87	0.03	0.03	0.03	0.03	0.03	0.03
YADGENA BROOK	1.16	204.81	0.04	0.04	0.04	0.04	0.04	0.04
YADGENA BROOK	1.74	203.05	0.05	0.05	0.05	0.05	0.05	0.05
YADGENA BROOK	1.81	202.72	0.04	0.02	0.02	0.02	0.02	0.04
YADGENA BROOK	2.20	202.59	0.03	0.03	0.03	0.03	0.03	0.03
YADGENA BROOK	2.56	201.79	0.02	0.03	0.03	0.02	0.02	0.02
YADGENA BROOK	3.02	201.11	0.02	0.02	0.02	0.02	0.02	0.02
YADGENA BROOK	3.87	200.18	0.02	0.02	0.02	0.02	0.02	0.02
YADGENA BROOK	4.53	199.31	0.01	0.03	0.03	0.01	0.01	0.01
YADGENA BROOK	4.87	198.83	-0.01	-0.16	-0.12	0.04	0.03	0.01
COONDEROO	0.00	201.33	0.01	-0.13	-0.09	0.46	-0.30	-0.06
COONDEROO	0.44	201.33	0.01	-0.13	-0.09	0.46	-0.30	-0.06
COONDEROO	0.80	201.33	0.01	-0.13	-0.09	0.46	-0.30	-0.06
COONDEROO	0.81	201.33	0.01	-0.13	-0.09	0.46	-0.30	-0.06

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Table F.3 Reduction in 20 Year ARI Design Flood Levels at the Various Cross-Sections for the Six Flood Mitigation Options, Moora Mike 11 Model - Cont'd

Branch Name	Branch Chainage (km)	Existing Design Flood Level (m AHD)	Reduction in 20 Year ARI Design Flood Level (m AHD)					
			Option 1	Option 2	Option 3	Option 4	Option 5	Option 6
MOORE 5	0.00	200.85	0.01	-0.12	-0.08	0.09	-0.23	0.06
MOORE 5	0.26	200.55	-0.02	-0.21	-0.18	0.04	-0.27	-0.04
MOORE 5	0.45	200.34	-0.02	-0.21	-0.18	0.04	-0.19	-0.03
MOORE 5	0.89	200.02	-0.04	-0.22	-0.19	0.00	-0.02	-0.06
MOORE 5	1.43	199.82	-0.03	-0.18	-0.15	0.00	0.16	-0.05
MOORE 5	1.91	199.53	-0.03	-0.16	-0.13	0.01	0.12	-0.04
MOORE 5	2.67	198.83	-0.01	-0.16	-0.12	0.04	0.03	0.01
MOORE 2	0.00	207.81	0.00	-0.43	-0.35	0.00	0.00	-0.36
MOORE 2	0.57	207.23	0.00	-0.34	-0.28	0.00	0.00	-0.32
MOORE 2	0.96	206.87	0.00	-0.37	-0.30	0.00	0.00	-0.35
MOORE 2	1.30	206.51	0.00	-0.40	-0.33	0.00	0.00	-0.44
MOORE 2	1.63	206.03	0.00	-0.39	-0.31	0.01	0.03	-0.33
MOORE 2	2.01	205.63	0.00	-0.30	-0.24	0.05	0.11	-0.31
MOORE 2	2.12	205.56	0.00	-0.26	-0.21	0.07	0.14	-0.35
MOORE 2	2.24	205.51	0.00	-0.25	-0.20	0.10	0.14	-0.37
MOORE 2	2.99	204.93	0.00	-0.25	-0.21	0.56	0.32	-0.48
MOORE 2	3.42	204.35	0.00	-0.28	-0.22	-0.45	0.79	-0.31
MOORE 2	3.72	204.00	0.00	-0.23	-0.19	-0.38	-0.41	-0.10
MOORE 2	3.97	203.84	0.00	-0.19	-0.16	-0.39	-0.43	-0.05
MOORE 2	4.21	203.75	0.00	-0.17	-0.15	-0.40	-0.45	-0.01
MOORE 2	4.25	203.50	0.00	-0.40	-0.31	-0.59	-0.67	-0.09
MOORE 2	4.40	203.43	0.00	-0.38	-0.29	-0.58	-0.64	-0.09
MOORE 2	4.58	203.36	0.00	-0.45	-0.34	-0.63	-0.66	-0.06
MOORE 2	4.60	203.03	-0.02	-0.26	-0.22	-0.39	-0.42	-0.21
MOORE 2	4.76	202.82	0.00	-0.24	-0.19	-0.34	-0.41	-0.20
MOORE 2	5.18	202.22	0.00	-0.42	-0.35	-0.35	-0.65	-0.37
MOORE 2	5.35	201.95	0.00	-0.38	-0.31	-0.18	-0.59	-0.27
MOORE 2	5.73	201.80	0.00	-0.33	-0.27	-0.06	-0.54	-0.21
MOORE 2	6.02	201.56	0.01	-0.23	-0.18	0.16	-0.42	-0.15
MOORE 2	6.31	201.33	0.01	-0.14	-0.09	0.36	-0.31	-0.06
MOORE 2	6.48	201.31	0.01	-0.13	-0.08	0.37	-0.30	-0.05
MOORE 2	6.65	201.30	0.01	-0.13	-0.08	0.33	-0.30	-0.05
MOORE 2	6.70	201.22	0.01	-0.12	-0.08	0.29	-0.27	-0.04
MOORE 2	7.15	201.18	0.01	-0.12	-0.07	0.11	-0.26	-0.02
MOORE 3	0.00	205.56	0.00	-0.26	-0.21	0.07	0.14	-0.35
MOORE 3	0.59	204.61	0.00	-0.17	-0.14	0.27	-3.74	-0.16
MOORE 3	0.83	204.44	0.00	-0.18	-0.14	0.29	-4.33	-0.23
MOORE 3	1.13	204.31	0.00	-0.44	-0.34	-0.16	-4.31	-0.58
MOORE 3	1.18	203.84	0.00	-0.10	-0.07	0.23	-3.84	-0.20
MOORE 3	1.77	202.91	0.00	-0.43	-0.29	0.47	-1.89	-
MOORE 3	2.22	202.37	0.00	-0.13	-0.09	0.51	-1.35	-
MOORE 3	2.60	202.18	0.00	-0.07	-0.05	0.52	-1.16	-
MOORE 3	2.87	202.03	0.00	-0.08	-0.06	0.45	-1.01	-
MOORE 3	3.34	201.55	0.00	-0.18	-0.12	0.39	-0.53	-
MOORE 3	3.65	201.33	0.01	-0.13	-0.09	0.46	-0.30	-0.06
MOORE 3	3.91	201.31	0.01	-0.13	-0.08	0.37	-0.30	-0.05
M2-M1	0.00	207.23	0.00	-0.34	-0.28	0.00	0.00	-0.32
M2-M1	0.14	207.10	0.00	-0.34	-0.27	0.00	0.00	-0.32
M2-M1	0.51	206.60	0.00	-0.38	-0.26	0.00	0.00	-0.33
M2-M1	0.83	206.33	0.00	-0.36	-0.20	0.00	0.00	-0.19
M2-M1	1.20	206.01	0.01	-0.36	-0.28	0.06	0.00	0.04
M2-M1	1.23	206.00	0.01	-0.35	-0.28	0.06	0.00	0.05
ISBISTER	0.00	206.60	0.00	-0.38	-0.26	0.00	0.00	-0.33
ISBISTER	0.40	206.01	0.00	-0.27	-0.25	-0.01	0.00	-0.26
ISBISTER	0.71	205.80	0.00	-0.27	-0.25	0.07	0.01	-0.26
ISBISTER	1.11	205.14	0.04	-0.40	-0.30	0.52	0.11	-0.36
ISBISTER	1.41	204.45	0.15	-0.28	-0.27	1.19	0.65	-0.28
ISBISTER	1.76	204.37	0.11	-0.23	-0.22	-	0.72	-0.23
ISBISTER	1.93	204.30	0.10	-0.20	-0.19	-	0.80	-0.20

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Table F.3 Reduction in 20 Year ARI Design Flood Levels at the Various Cross-Sections for the Six Flood Mitigation Options, Moora Mike 11 Model - Cont'd

Branch Name	Branch Chainage (km)	Existing Design Flood Level (m AHD)	Reduction in 20 Year ARI Design Flood Level (m AHD)					
			Option 1	Option 2	Option 3	Option 4	Option 5	Option 6
ISBISTER	2.24	204.12	0.10	-0.15	-0.15	-	-	-0.15
ISBISTER	2.48	204.01	0.05	-0.10	-0.10	-	-	-0.10
ISBISTER	2.73	204.01	0.05	-0.10	-0.10	-	-	-0.10
ISBISTER	3.06	-	-	-	-	-	-	-
ISBISTER	3.26	-	-	-	-	-	-	-
ISBISTER	3.48	-	-	-	-	-	-	-
ISBISTER	4.00	-	-	-	-	-	-	-
ISBISTER	4.34	-	-	-	-	-	-	-
ISBISTER	4.85	-	-	-	-	-	-	-
MOORE 4	0.00	205.40	0.09	-	-	0.34	0.03	-
MOORE 4	0.13	205.28	-0.14	-	-	0.47	0.01	-
MOORE 4	0.51	204.72	-0.07	-	-	-	0.29	-
MOORE 4	0.95	204.12	-0.08	-	-	-	0.89	-
MOORE 4	0.97	204.12	-0.08	-	-	-	-	-
MOORE 4	1.61	204.02	-0.03	-	-	-	-	-
MOORE 4	1.84	203.85	-0.07	-	-	-	-	-
MOORE 4	2.02	203.63	-0.06	-	-	-	-	-
MOORE 4	2.13	203.51	-0.07	-	-	-	-	-
MOORE 4	2.36	203.25	-0.09	-	-	-	-	-
MOORE 4	2.37	203.24	-0.09	-	-	-	-	-
MOORE 4	2.96	202.06	-0.06	-	-	-	-	-
MOORE 4	3.16	201.64	-0.07	-	-	-	-	-
MOORE 4	3.58	200.98	-0.04	-	-	-	-	-
MOORE 4	3.95	200.02	-0.04	-	-	-	-	-
CLINCH ST	0.00	203.25	-0.09	-	-	-	-	-
CLINCH ST	0.37	202.93	-0.06	-	-	-	-	-
CLINCH ST	0.60	202.42	-0.07	-	-	-	-	-
CLINCH ST	0.79	201.77	-0.05	-	-	-	-	-
CLINCH ST	1.27	201.17	-0.04	-	-	-	-	-
CLINCH ST	1.53	200.77	-0.04	-	-	-	-	-
CLINCH ST	1.57	200.55	-0.02	-	-	-	-	-
CAMERON ST	0.00	204.12	-0.08	-	-	-	-	-
CAMERON ST	0.33	203.86	-0.06	-	-	-	-	-
CAMERON ST	0.58	203.30	0.06	-	-	-	-	-
CAMERON ST	0.71	203.28	0.08	-	-	-	-	-
M1M4-1	0.00	204.71	0.02	-	-	-	-	-
M1M4-1	0.10	204.72	-0.07	-	-	-	-	-
WALEBING RD	0.00	-	-	-	-	-	-	-
WALEBING RD	0.36	-	-	-	-	-	-	-
WALEBING RD	0.73	-	-	-	-	-	-	-
WALEBING RD	1.28	-	-	-	-	-	-	-
WALEBING RD	1.91	-	-	-	-	-	-	-
WALEBING RD	2.22	-	-	-	-	-	-	-
WALEBING RD	2.66	-	-	-	-	-	-	-
BINDOON RD	0.00	-	-	-	-	-	-	-
BINDOON RD	0.45	-	-	-	-	-	-	-
BINDOON RD	0.88	-	-	-	-	-	-	-
BINDOON RD	1.45	-	-	-	-	-	-	-
BINDOON RD	1.77	-	-	-	-	-	-	-
BINDOON RD	2.21	-	-	-	-	-	-	-

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Table F.4 Reduction in 50 Year ARI Design Flood Levels at the Various Cross-Sections for the Six Flood Mitigation Options, Moora Mike 11 Model

Branch Name	Branch Chainage (km)	Existing Design Flood Level (m AHD)	Reduction in 50 Year ARI Design Flood Level (m AHD)					
			Option 1	Option 2	Option 3	Option 4	Option 5	Option 6
MOORE 1	0.00	208.18	0.00	-0.39	-0.38	0.00	0.00	-0.10
MOORE 1	0.32	207.97	0.00	-0.41	-0.41	0.00	0.00	-0.21
MOORE 1	0.44	207.84	0.00	-0.48	-0.47	0.00	0.00	-0.26
MOORE 1	0.86	207.25	0.00	-0.52	-0.51	0.00	0.00	-0.20
MOORE 1	1.33	206.73	0.00	-0.50	-0.49	0.01	0.00	-0.05
MOORE 1	1.58	206.46	0.00	-0.44	-0.43	0.03	0.00	0.00
MOORE 1	1.87	206.24	0.01	-0.40	-0.39	0.07	0.00	0.03
MOORE 1	1.92	206.23	0.01	-0.41	-0.40	0.07	0.00	0.04
MOORE 1	2.59	205.59	0.05	-0.30	-0.27	0.34	0.04	-0.01
MOORE 1	2.63	205.58	0.05	-0.29	-0.26	0.36	0.05	0.01
MOORE 1	2.76	205.40	0.03	-0.26	-0.24	0.48	0.11	-0.02
MOORE 1	3.18	204.86	-0.02	-0.23	-0.23	-0.54	0.47	-0.27
MOORE 1	3.60	204.45	0.00	-0.28	-0.28	-0.70	0.83	-0.39
MOORE 1	3.72	204.36	0.00	-0.30	-0.30	-0.74	-0.66	-0.37
MOORE 1	3.93	204.17	0.01	-0.34	-0.33	-0.76	-0.69	-0.35
MOORE 1	4.19	203.90	0.01	-0.49	-0.48	-0.90	-0.84	-0.24
MOORE 1	4.48	203.66	0.01	-0.60	-0.59	-1.01	-0.96	-0.14
MOORE 1	4.49	203.67	0.01	-0.60	-0.59	-1.00	-0.95	-0.16
MOORE 1	4.54	203.66	0.01	-0.61	-0.60	-1.03	-0.98	-0.15
MOORE 1	4.72	203.42	0.00	-0.47	-0.46	-0.88	-0.84	-0.28
MOORE 1	4.86	203.32	0.00	-0.44	-0.43	-0.82	-0.78	-0.24
MOORE 1	4.89	203.24	0.00	-0.43	-0.42	-0.80	-0.76	-0.26
MOORE 1	5.06	202.98	0.00	-0.38	-0.37	-0.69	-0.68	-0.20
MOORE 1	5.30	202.71	0.00	-0.37	-0.36	-0.64	-0.68	-0.09
MOORE 1	5.55	202.50	-0.01	-0.38	-0.37	-0.61	-0.71	-0.02
MOORE 1	5.57	202.49	-0.01	-0.38	-0.36	-0.61	-0.71	-0.02
MOORE 1	5.60	202.37	-0.01	-0.35	-0.33	-0.55	-0.66	-0.15
MOORE 1	5.78	202.26	-0.01	-0.33	-0.31	-0.50	-0.63	-0.13
MOORE 1	6.08	201.95	0.00	-0.27	-0.25	-0.31	-0.54	-0.09
MOORE 1	6.56	201.37	0.01	-0.16	-0.14	0.11	-0.34	-0.02
MOORE 1	6.69	201.31	0.01	-0.16	-0.15	0.16	-0.34	-0.02
MOORE 1	6.97	201.12	0.01	-0.13	-0.12	0.13	-0.29	0.01
MOORE 1	7.17	200.96	0.01	-0.16	-0.14	0.12	-0.32	0.05
MOORE 1	7.34	200.87	0.01	-0.15	-0.13	0.10	-0.30	0.05
MOORE 1	7.89	200.50	0.00	-0.12	-0.11	0.07	-0.24	0.04
MOORE 1	8.46	200.05	0.00	-0.15	-0.14	0.07	-0.25	0.04
MOORE 1	9.01	199.65	0.00	-0.15	-0.14	0.06	-0.16	0.03
MOORE 1	9.81	198.99	0.00	-0.13	-0.14	0.05	0.04	0.00
MOORE 1	9.91	198.97	0.00	-0.13	-0.14	0.05	0.05	-0.01
MOORE 1	10.11	198.80	0.00	-0.13	-0.14	0.06	0.06	-0.01
MOORE 1	10.66	198.24	0.00	-0.13	-0.14	0.06	0.06	-0.01
YADGENA BROOK	0.00	207.95	0.00	0.00	0.00	0.00	0.00	0.00
YADGENA BROOK	0.31	207.09	0.02	0.02	0.02	0.01	0.01	0.01
YADGENA BROOK	0.36	206.92	0.03	0.03	0.03	0.03	0.03	0.03
YADGENA BROOK	0.76	206.06	0.03	0.03	0.03	0.02	0.02	0.02
YADGENA BROOK	1.16	205.05	0.03	0.03	0.03	0.03	0.03	0.03
YADGENA BROOK	1.74	203.31	0.04	0.04	0.04	0.03	0.03	0.03
YADGENA BROOK	1.81	203.02	0.05	0.01	0.01	0.00	0.00	0.04
YADGENA BROOK	2.20	202.78	0.03	0.03	0.03	0.02	0.02	0.02
YADGENA BROOK	2.56	201.95	0.03	0.03	0.03	0.02	0.02	0.02
YADGENA BROOK	3.02	201.26	0.02	0.02	0.02	0.02	0.02	0.02
YADGENA BROOK	3.87	200.28	0.01	0.02	0.02	0.01	0.01	0.01
YADGENA BROOK	4.53	199.41	0.01	0.05	0.05	0.00	0.00	0.00
YADGENA BROOK	4.87	198.97	0.00	-0.13	-0.14	0.05	0.05	-0.01
COONDEROO	0.00	201.47	0.02	-0.19	-0.17	0.55	-0.41	-0.06
COONDEROO	0.44	201.47	0.02	-0.19	-0.17	0.55	-0.41	-0.06
COONDEROO	0.80	201.47	0.02	-0.19	-0.17	0.55	-0.41	-0.06
COONDEROO	0.81	201.47	0.02	-0.19	-0.17	0.54	-0.41	-0.06

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Table F.4 Reduction in 50 Year ARI Design Flood Levels at the Various Cross-Sections for the Six Flood Mitigation Options, Moora Mike 11 Model - Cont'd

Branch Name	Branch Chainage (km)	Existing Design Flood Level (m AHD)	Reduction in 50 Year ARI Design Flood Level (m AHD)					
			Option 1	Option 2	Option 3	Option 4	Option 5	Option 6
MOORE 5	0.00	200.96	0.01	-0.16	-0.14	0.12	-0.32	0.05
MOORE 5	0.26	200.72	0.00	-0.31	-0.29	0.08	-0.37	-0.02
MOORE 5	0.45	200.52	0.00	-0.32	-0.30	0.08	-0.26	-0.02
MOORE 5	0.89	200.22	-0.01	-0.35	-0.33	0.03	-0.04	-0.05
MOORE 5	1.43	199.99	-0.01	-0.30	-0.28	0.03	0.18	-0.04
MOORE 5	1.91	199.68	0.00	-0.25	-0.25	0.03	0.13	-0.03
MOORE 5	2.67	198.97	0.00	-0.13	-0.14	0.05	0.05	-0.01
MOORE 2	0.00	207.97	0.00	-0.41	-0.41	0.00	0.00	-0.21
MOORE 2	0.57	207.33	0.00	-0.30	-0.30	0.00	0.00	-0.14
MOORE 2	0.96	206.98	0.00	-0.34	-0.34	0.00	0.00	-0.17
MOORE 2	1.30	206.63	0.00	-0.37	-0.36	0.00	0.00	-0.23
MOORE 2	1.63	206.15	0.00	-0.36	-0.35	0.02	0.02	-0.17
MOORE 2	2.01	205.71	0.00	-0.25	-0.24	0.11	0.09	-0.10
MOORE 2	2.12	205.63	0.00	-0.22	-0.21	0.15	0.14	-0.12
MOORE 2	2.24	205.57	0.00	-0.20	-0.20	0.18	0.15	-0.14
MOORE 2	2.99	205.01	0.00	-0.24	-0.24	0.63	0.44	-0.28
MOORE 2	3.42	204.48	0.02	-0.30	-0.30	-0.56	0.93	-0.25
MOORE 2	3.72	204.21	0.04	-0.35	-0.35	-0.56	-0.55	-0.18
MOORE 2	3.97	203.98	0.03	-0.27	-0.27	-0.49	-0.48	-0.11
MOORE 2	4.21	203.86	0.02	-0.23	-0.23	-0.45	-0.44	-0.05
MOORE 2	4.25	203.73	0.04	-0.47	-0.46	-0.78	-0.84	-0.16
MOORE 2	4.40	203.63	0.02	-0.42	-0.41	-0.74	-0.78	-0.15
MOORE 2	4.58	203.50	0.01	-0.37	-0.36	-0.74	-0.76	-0.08
MOORE 2	4.60	203.24	0.02	-0.39	-0.39	-0.57	-0.59	-0.23
MOORE 2	4.76	203.04	0.03	-0.38	-0.38	-0.51	-0.59	-0.23
MOORE 2	5.18	202.53	0.04	-0.59	-0.58	-0.45	-0.91	-0.38
MOORE 2	5.35	202.29	0.04	-0.59	-0.58	-0.25	-0.88	-0.33
MOORE 2	5.73	202.09	0.04	-0.50	-0.49	-0.07	-0.78	-0.24
MOORE 2	6.02	201.88	0.03	-0.46	-0.44	0.13	-0.70	-0.26
MOORE 2	6.31	201.51	0.02	-0.23	-0.21	0.42	-0.45	-0.09
MOORE 2	6.48	201.46	0.02	-0.19	-0.17	0.45	-0.41	-0.06
MOORE 2	6.65	201.44	0.02	-0.18	-0.17	0.39	-0.40	-0.05
MOORE 2	6.70	201.35	0.01	-0.18	-0.16	0.40	-0.37	-0.05
MOORE 2	7.15	201.31	0.01	-0.16	-0.15	0.16	-0.34	-0.02
MOORE 3	0.00	205.63	0.00	-0.22	-0.21	0.15	0.14	-0.12
MOORE 3	0.59	204.66	0.00	-0.16	-0.16	0.45	-	-0.10
MOORE 3	0.83	204.49	0.00	-0.14	-0.14	0.42	-	-0.09
MOORE 3	1.13	204.37	0.00	-0.25	-0.24	0.07	-	-0.12
MOORE 3	1.18	203.85	0.00	-0.05	-0.04	0.49	-	-0.02
MOORE 3	1.77	202.93	0.00	-0.14	-0.10	0.63	-	-0.21
MOORE 3	2.22	202.39	0.00	-0.07	-0.06	0.69	-	-0.09
MOORE 3	2.60	202.20	0.00	-0.04	-0.04	0.76	-	-0.05
MOORE 3	2.87	202.05	0.00	-0.05	-0.04	0.64	-	-0.05
MOORE 3	3.34	201.62	0.01	-0.13	-0.12	0.46	-	-0.09
MOORE 3	3.65	201.47	0.02	-0.19	-0.17	0.54	-0.41	-0.06
MOORE 3	3.91	201.46	0.02	-0.19	-0.17	0.45	-0.41	-0.06
M2-M1	0.00	207.33	0.00	-0.30	-0.30	0.00	0.00	-0.14
M2-M1	0.14	207.20	0.00	-0.30	-0.29	0.00	0.00	-0.14
M2-M1	0.51	206.70	0.00	-0.27	-0.27	0.00	0.00	-0.14
M2-M1	0.83	206.43	0.00	-0.23	-0.22	0.03	0.00	-0.04
M2-M1	1.20	206.23	0.01	-0.41	-0.40	0.07	0.00	0.03
M2-M1	1.23	206.23	0.01	-0.41	-0.39	0.07	0.00	0.04
ISBISTER	0.00	206.70	0.00	-0.27	-0.27	0.00	0.00	-0.14
ISBISTER	0.40	206.12	0.00	-0.35	-0.32	0.00	0.00	-0.16
ISBISTER	0.71	205.91	0.01	-0.34	-0.32	0.09	0.03	-0.13
ISBISTER	1.11	205.34	0.05	-0.47	-0.45	0.51	0.16	-0.07
ISBISTER	1.41	204.94	0.09	-0.75	-0.75	0.89	0.48	-0.13
ISBISTER	1.76	204.73	0.04	-0.58	-0.57	-	0.66	-0.16
ISBISTER	1.93	204.66	0.04	-0.55	-0.55	-	0.73	-0.16

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Table F.4 Reduction in 50 Year ARI Design Flood Levels at the Various Cross-Sections for the Six Flood Mitigation Options, Moora Mike 11 Model - Cont'd

Branch Name	Branch Chainage (km)	Existing Design Flood Level (m AHD)	Reduction in 50 Year ARI Design Flood Level (m AHD)					
			Option 1	Option 2	Option 3	Option 4	Option 5	Option 6
ISBISTER	2.24	204.41	0.01	-0.44	-0.44	-	-	-0.11
ISBISTER	2.48	204.15	0.01	-0.23	-0.23	-	-	-0.05
ISBISTER	2.73	204.13	0.00	-0.21	-0.21	-	-	-0.03
ISBISTER	3.06	203.31	-203.31	-	-	-	-	-
ISBISTER	3.26	202.89	-202.89	-	-	-	-	-
ISBISTER	3.48	202.70	-202.70	-	-	-	-	-
ISBISTER	4.00	202.09	-202.09	-	-	-	-	-
ISBISTER	4.34	201.77	-201.77	-	-	0.12	-	-
ISBISTER	4.85	201.31	-201.31	-	-	0.16	-	-
MOORE 4	0.00	205.58	0.05	-	-0.26	0.36	0.05	0.01
MOORE 4	0.13	205.42	-0.22	-	-0.88	0.51	0.09	-0.15
MOORE 4	0.51	204.86	-0.03	-	-0.58	-	0.44	-0.17
MOORE 4	0.95	204.27	-0.01	-	-0.48	-	1.03	-0.16
MOORE 4	0.97	204.27	-0.01	-	-0.47	-	-	-0.16
MOORE 4	1.61	204.13	0.04	-	-0.37	-	-	-0.10
MOORE 4	1.84	203.95	0.00	-	-0.38	-	-	-0.11
MOORE 4	2.02	203.74	0.00	-	-0.46	-	-	-0.13
MOORE 4	2.13	203.61	-0.02	-	-0.48	-	-	-0.12
MOORE 4	2.36	203.36	-0.03	-	-0.39	-	-	-0.14
MOORE 4	2.37	203.35	-0.03	-	-0.39	-	-	-0.14
MOORE 4	2.96	202.18	-0.03	-	-0.63	-	-	-0.15
MOORE 4	3.16	201.77	-0.03	-	-0.86	-	-	-0.16
MOORE 4	3.58	201.07	-0.02	-	-0.76	-	-	-0.11
MOORE 4	3.95	200.22	-0.01	-	-0.33	-	-	-0.05
CLINCH ST	0.00	203.36	-0.03	-	-0.39	-	-	-0.14
CLINCH ST	0.37	203.01	-0.02	-	-0.27	-	-	-0.10
CLINCH ST	0.60	202.50	-0.01	-	-0.39	-	-	-0.09
CLINCH ST	0.79	201.83	-0.01	-	-0.57	-	-	-0.07
CLINCH ST	1.27	201.31	0.00	-0.32	-0.30	-	-	-0.08
CLINCH ST	1.53	200.93	0.00	-0.39	-0.36	-	-	-0.07
CLINCH ST	1.57	200.72	0.00	-0.31	-0.29	-	-	-0.02
CAMERON ST	0.00	204.27	-0.01	-0.72	-0.48	-	-	-0.16
CAMERON ST	0.33	203.97	0.00	-0.65	-0.48	-	-	-0.09
CAMERON ST	0.58	203.69	0.01	-0.57	-0.52	-	-	-0.16
CAMERON ST	0.71	203.66	0.01	-0.60	-0.59	-	-	-0.14
M1M4-1	0.00	204.86	-0.02	-0.23	-0.23	-	0.47	-0.27
M1M4-1	0.10	204.86	-0.03	-0.86	-0.58	-	0.44	-0.17
WALEBING RD	0.00	206.92	-	-	-	-	-	-
WALEBING RD	0.36	206.49	-	-	-	-	-	-
WALEBING RD	0.73	206.10	-	-	-	-	-	-
WALEBING RD	1.28	204.94	-	-	-	-	-	-
WALEBING RD	1.91	203.89	-	-	-	-	-	-
WALEBING RD	2.22	203.74	-	-	-	-	-	-
WALEBING RD	2.66	203.61	-	-	-	-	-	-
BINDOON RD	0.00	203.31	-	-	-	-	-	-
BINDOON RD	0.45	203.56	-	-	-	-	-	-
BINDOON RD	0.88	203.57	-	-	-	-	-	-
BINDOON RD	1.45	203.57	-	-	-	-	-	-
BINDOON RD	1.77	203.57	-	-	-	-	-	-
BINDOON RD	2.21	203.61	-	-	-	-	-	-

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Table F.5 Reduction in 100 Year ARI Design Flood Levels at the Various Cross-Sections for the Six Flood Mitigation Options, Moora Mike 11 Model

Branch Name	Branch Chainage (km)	Existing Design Flood Level (m AHD)	Reduction in 100 Year ARI Design Flood Level (m AHD)					
			Option 1	Option 2	Option 3	Option 4	Option 5	Option 6
MOORE 1	0.00	208.29	0.00	-0.38	-0.44	0.00	0.00	-0.09
MOORE 1	0.32	208.09	0.00	-0.41	-0.47	0.00	0.00	-0.19
MOORE 1	0.44	207.96	0.00	-0.46	-0.53	0.00	0.00	-0.22
MOORE 1	0.86	207.36	0.00	-0.51	-0.56	0.00	0.00	-0.17
MOORE 1	1.33	206.85	0.00	-0.51	-0.56	0.01	0.00	-0.04
MOORE 1	1.58	206.59	0.00	-0.47	-0.51	0.03	0.01	0.00
MOORE 1	1.87	206.40	0.00	-0.46	-0.49	0.07	0.01	0.03
MOORE 1	1.92	206.38	0.00	-0.46	-0.50	0.07	0.01	0.04
MOORE 1	2.59	205.68	0.04	-0.25	-0.28	0.37	0.10	-0.01
MOORE 1	2.63	205.66	0.04	-0.24	-0.27	0.39	0.11	0.01
MOORE 1	2.76	205.47	0.03	-0.20	-0.23	0.53	0.21	0.00
MOORE 1	3.18	204.92	0.00	-0.23	-0.26	-0.57	0.59	-0.15
MOORE 1	3.60	204.54	0.01	-0.28	-0.33	-0.76	0.94	-0.20
MOORE 1	3.72	204.46	0.01	-0.30	-0.35	-0.81	-0.71	-0.18
MOORE 1	3.93	204.28	0.01	-0.35	-0.40	-0.84	-0.75	-0.16
MOORE 1	4.19	204.04	0.01	-0.50	-0.57	-1.01	-0.94	-0.06
MOORE 1	4.48	203.86	0.00	-0.66	-0.72	-1.16	-1.12	0.02
MOORE 1	4.49	203.88	0.00	-0.66	-0.73	-1.17	-1.12	0.01
MOORE 1	4.54	203.87	0.00	-0.67	-0.74	-1.19	-1.15	0.01
MOORE 1	4.72	203.54	0.00	-0.45	-0.51	-0.94	-0.91	-0.19
MOORE 1	4.86	203.43	0.00	-0.42	-0.47	-0.88	-0.84	-0.15
MOORE 1	4.89	203.34	0.00	-0.40	-0.45	-0.85	-0.82	-0.16
MOORE 1	5.06	203.05	0.00	-0.33	-0.38	-0.71	-0.71	-0.08
MOORE 1	5.30	202.78	0.00	-0.33	-0.36	-0.64	-0.70	0.02
MOORE 1	5.55	202.57	0.00	-0.32	-0.35	-0.58	-0.73	0.09
MOORE 1	5.57	202.56	0.00	-0.32	-0.35	-0.57	-0.73	0.10
MOORE 1	5.60	202.42	0.00	-0.27	-0.29	-0.50	-0.66	-0.07
MOORE 1	5.78	202.30	0.00	-0.25	-0.27	-0.43	-0.62	-0.06
MOORE 1	6.08	201.99	0.00	-0.21	-0.22	-0.23	-0.53	-0.03
MOORE 1	6.56	201.46	0.00	-0.21	-0.22	0.13	-0.41	-0.01
MOORE 1	6.69	201.42	0.00	-0.23	-0.24	0.18	-0.43	-0.02
MOORE 1	6.97	201.21	0.00	-0.19	-0.19	0.14	-0.35	0.01
MOORE 1	7.17	201.05	0.00	-0.21	-0.21	0.12	-0.38	0.05
MOORE 1	7.34	200.94	0.00	-0.19	-0.19	0.10	-0.35	0.05
MOORE 1	7.89	200.55	0.00	-0.15	-0.15	0.07	-0.27	0.04
MOORE 1	8.46	200.11	0.00	-0.17	-0.18	0.06	-0.21	0.03
MOORE 1	9.01	199.72	0.00	-0.19	-0.19	0.05	-0.10	0.02
MOORE 1	9.81	199.08	0.00	-0.22	-0.25	0.05	0.04	0.01
MOORE 1	9.91	199.06	0.00	-0.22	-0.27	0.05	0.05	0.01
MOORE 1	10.11	198.89	0.00	-0.23	-0.27	0.05	0.05	0.01
MOORE 1	10.66	198.33	0.00	-0.23	-0.27	0.05	0.04	0.01
YADGENA BROOK	0.00	208.10	0.00	0.00	0.00	0.00	0.00	0.00
YADGENA BROOK	0.31	207.25	0.03	0.03	0.03	0.03	0.03	0.03
YADGENA BROOK	0.36	207.11	0.06	0.06	0.06	0.06	0.06	0.06
YADGENA BROOK	0.76	206.22	0.05	0.05	0.05	0.05	0.05	0.05
YADGENA BROOK	1.16	205.23	0.06	0.06	0.06	0.06	0.06	0.06
YADGENA BROOK	1.74	203.51	0.06	0.07	0.07	0.06	0.06	0.06
YADGENA BROOK	1.81	203.30	0.08	0.04	0.04	0.03	0.03	0.08
YADGENA BROOK	2.20	202.95	0.05	0.05	0.05	0.05	0.05	0.05
YADGENA BROOK	2.56	202.09	0.04	0.04	0.04	0.04	0.04	0.04
YADGENA BROOK	3.02	201.39	0.04	0.04	0.04	0.04	0.04	0.04
YADGENA BROOK	3.87	200.35	0.02	0.03	0.02	0.02	0.02	0.02
YADGENA BROOK	4.53	199.46	0.02	0.04	0.04	0.02	0.02	0.02
YADGENA BROOK	4.87	199.06	0.00	-0.22	-0.27	0.05	0.05	0.01
COONDEROO	0.00	201.63	0.01	-0.30	-0.30	0.57	-0.54	-0.10
COONDEROO	0.44	201.63	0.01	-0.30	-0.30	0.57	-0.54	-0.10
COONDEROO	0.80	201.63	0.01	-0.30	-0.30	0.55	-0.55	-0.10
COONDEROO	0.81	201.63	0.01	-0.30	-0.30	0.55	-0.55	-0.10

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Table F.5 Variation in 100 Year ARI Design Flood Levels at the Cross-Sections for the Six Flood Mitigation Options, Moora Mike 11 Model - Cont'd

Branch Name	Branch Chainage (km)	Existing Design Flood Level (m AHD)	Reduction in 100 Year ARI Design Flood Level (m AHD)					
			Option 1	Option 2	Option 3	Option 4	Option 5	Option 6
MOORE 5	0.00	201.05	0.00	-0.21	-0.21	0.12	-0.38	0.05
MOORE 5	0.26	200.83	0.00	-0.34	-0.35	0.10	-0.43	0.01
MOORE 5	0.45	200.64	0.00	-0.36	-0.37	0.09	-0.31	0.01
MOORE 5	0.89	200.33	-0.01	-0.39	-0.40	0.05	-0.06	-0.01
MOORE 5	1.43	200.09	-0.01	-0.34	-0.34	0.05	0.17	0.00
MOORE 5	1.91	199.77	0.00	-0.29	-0.30	0.05	0.11	0.00
MOORE 5	2.67	199.06	0.00	-0.22	-0.27	0.05	0.05	0.01
MOORE 2	0.00	208.09	0.00	-0.41	-0.47	0.00	0.00	-0.19
MOORE 2	0.57	207.41	0.00	-0.28	-0.33	0.00	0.00	-0.11
MOORE 2	0.96	207.07	0.00	-0.32	-0.37	0.00	0.00	-0.13
MOORE 2	1.30	206.71	0.00	-0.33	-0.39	0.00	0.00	-0.16
MOORE 2	1.63	206.21	0.00	-0.30	-0.36	0.02	0.01	-0.10
MOORE 2	2.01	205.73	0.00	-0.19	-0.23	0.19	0.12	-0.04
MOORE 2	2.12	205.65	0.00	-0.16	-0.20	0.21	0.17	-0.07
MOORE 2	2.24	205.60	0.00	-0.16	-0.19	0.25	0.19	-0.08
MOORE 2	2.99	205.06	0.01	-0.21	-0.25	0.69	0.57	-0.19
MOORE 2	3.42	204.65	0.04	-0.38	-0.42	-0.72	0.96	-0.23
MOORE 2	3.72	204.42	0.03	-0.49	-0.52	-0.76	-0.73	-0.19
MOORE 2	3.97	204.19	0.03	-0.42	-0.45	-0.67	-0.65	-0.16
MOORE 2	4.21	204.07	0.03	-0.39	-0.41	-0.63	-0.62	-0.14
MOORE 2	4.25	204.03	0.04	-0.65	-0.70	-1.06	-1.09	-0.19
MOORE 2	4.40	203.80	0.02	-0.47	-0.52	-0.89	-0.90	-0.13
MOORE 2	4.58	203.61	0.01	-0.36	-0.41	-0.83	-0.83	-0.06
MOORE 2	4.60	203.40	0.01	-0.48	-0.52	-0.71	-0.71	-0.18
MOORE 2	4.76	203.22	0.02	-0.48	-0.52	-0.65	-0.72	-0.19
MOORE 2	5.18	202.76	0.02	-0.68	-0.75	-0.55	-1.09	-0.29
MOORE 2	5.35	202.54	0.02	-0.71	-0.77	-0.36	-1.09	-0.27
MOORE 2	5.73	202.30	0.02	-0.60	-0.65	-0.13	-0.95	-0.19
MOORE 2	6.02	202.03	0.01	-0.53	-0.55	0.13	-0.82	-0.16
MOORE 2	6.31	201.68	0.01	-0.35	-0.35	0.42	-0.59	-0.11
MOORE 2	6.48	201.62	0.01	-0.30	-0.30	0.45	-0.54	-0.09
MOORE 2	6.65	201.59	0.01	-0.29	-0.29	0.41	-0.52	-0.08
MOORE 2	6.70	201.50	0.01	-0.29	-0.29	0.41	-0.50	-0.07
MOORE 2	7.15	201.42	0.00	-0.23	-0.24	0.18	-0.43	-0.02
MOORE 3	0.00	205.65	0.00	-0.16	-0.20	0.21	0.17	-0.07
MOORE 3	0.59	204.69	0.00	-0.14	-0.16	0.56	-	-0.06
MOORE 3	0.83	204.51	0.00	-0.12	-0.14	0.52	-	-0.05
MOORE 3	1.13	204.39	0.00	-0.15	-0.19	0.26	-	-0.05
MOORE 3	1.18	203.85	0.00	-0.03	-0.04	0.64	-	-0.01
MOORE 3	1.77	202.94	0.00	-0.04	-0.05	0.72	-	-0.03
MOORE 3	2.22	202.40	0.00	-0.04	-0.05	0.84	-	-0.03
MOORE 3	2.60	202.20	0.00	-0.02	-0.03	0.94	-	-0.02
MOORE 3	2.87	202.08	0.00	-0.05	-0.06	0.76	-	-0.04
MOORE 3	3.34	201.80	0.01	-0.25	-0.26	0.43	-	-0.16
MOORE 3	3.65	201.63	0.01	-0.30	-0.30	0.55	-0.55	-0.10
MOORE 3	3.91	201.62	0.01	-0.30	-0.30	0.45	-0.54	-0.09
M2-M1	0.00	207.41	0.00	-0.28	-0.33	0.00	0.00	-0.11
M2-M1	0.14	207.29	0.00	-0.29	-0.34	0.00	0.00	-0.12
M2-M1	0.51	206.78	0.00	-0.27	-0.32	0.01	0.00	-0.09
M2-M1	0.83	206.54	0.00	-0.27	-0.30	0.03	0.00	-0.01
M2-M1	1.20	206.39	0.00	-0.46	-0.50	0.07	0.01	0.03
M2-M1	1.23	206.38	0.00	-0.46	-0.50	0.07	0.01	0.04
ISBISTER	0.00	206.78	0.00	-0.27	-0.32	0.01	0.00	-0.09
ISBISTER	0.40	206.24	0.00	-0.34	-0.40	0.04	0.00	-0.09
ISBISTER	0.71	206.05	0.01	-0.36	-0.41	0.11	0.04	-0.08
ISBISTER	1.11	205.49	0.04	-0.46	-0.48	0.49	0.21	-0.05
ISBISTER	1.41	205.14	0.05	-0.81	-0.90	0.80	0.48	-0.06
ISBISTER	1.76	204.88	0.03	-0.60	-0.67	-	0.71	-0.09
ISBISTER	1.93	204.81	0.03	-0.59	-0.66	-	0.78	-0.09

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Table F.5 Reduction in 100 Year ARI Design Flood Levels at the Various Cross-Sections for the Six Flood Mitigation Options, Moora Mike 11 Model - Cont'd

Branch Name	Branch Chainage (km)	Existing Design Flood Level (m AHD)	Reduction in 100 Year ARI Design Flood Level (m AHD)					
			Option 1	Option 2	Option 3	Option 4	Option 5	Option 6
ISBISTER	2.24	204.51	0.02	-0.47	-0.52	-	-	-0.09
ISBISTER	2.48	204.19	0.01	-0.22	-0.26	-	-	-0.04
ISBISTER	2.73	204.16	0.00	-0.19	-0.22	-	-	-0.02
ISBISTER	3.06	203.44	0.00	-	-	-	-	-0.15
ISBISTER	3.26	203.04	0.01	-	-	-	-	-0.16
ISBISTER	3.48	202.86	0.01	-	-	-	-	-0.19
ISBISTER	4.00	202.17	0.00	-	-	-0.04	-	-0.11
ISBISTER	4.34	201.80	0.00	-	-	0.26	-	-0.03
ISBISTER	4.85	201.42	0.00	-	-	0.18	-	-0.02
MOORE 4	0.00	205.66	0.04	-0.24	-0.27	0.39	0.11	0.01
MOORE 4	0.13	205.49	-0.18	-0.61	-0.70	0.56	0.18	-0.19
MOORE 4	0.51	204.92	-0.04	-0.42	-0.46	-	0.56	-0.15
MOORE 4	0.95	204.35	-0.02	-0.42	-0.45	-	1.14	-0.17
MOORE 4	0.97	204.34	-0.02	-0.41	-0.45	-	-	-0.17
MOORE 4	1.61	204.15	0.04	-0.27	-0.30	-	-	-0.06
MOORE 4	1.84	204.00	-0.01	-0.30	-0.33	-	-	-0.10
MOORE 4	2.02	203.80	-0.01	-0.32	-0.35	-	-	-0.09
MOORE 4	2.13	203.66	-0.02	-0.29	-0.31	-	-	-0.10
MOORE 4	2.36	203.42	-0.02	-0.34	-0.36	-	-	-0.12
MOORE 4	2.37	203.40	-0.02	-0.34	-0.36	-	-	-0.11
MOORE 4	2.96	202.24	-0.02	-0.42	-0.43	-	-	-0.13
MOORE 4	3.16	201.82	-0.02	-0.35	-0.37	-	-	-0.13
MOORE 4	3.58	201.12	-0.01	-0.20	-0.22	-	-	-0.09
MOORE 4	3.95	200.33	-0.01	-0.39	-0.40	-	-	-0.01
CLINCH ST	0.00	203.42	-0.02	-0.34	-0.36	-	-	-0.12
CLINCH ST	0.37	203.05	-0.01	-0.23	-0.24	-	-	-0.08
CLINCH ST	0.60	202.52	-0.01	-0.25	-0.26	-	-	-0.06
CLINCH ST	0.79	201.86	0.00	-0.22	-0.24	-	-	-0.04
CLINCH ST	1.27	201.37	0.00	-0.29	-0.30	-	-	-0.02
CLINCH ST	1.53	200.99	0.00	-0.32	-0.33	-	-	-0.01
CLINCH ST	1.57	200.83	0.00	-0.34	-0.35	-	-	0.01
CAMERON ST	0.00	204.35	-0.02	-0.42	-0.45	-	-	-0.17
CAMERON ST	0.33	204.05	-0.01	-0.44	-0.49	-	-	-0.04
CAMERON ST	0.58	203.90	0.00	-0.67	-0.70	-	-	-0.01
CAMERON ST	0.71	203.86	0.00	-0.66	-0.72	-	-	0.02
M1M4-1	0.00	204.92	0.00	-0.23	-0.26	-0.57	0.59	-0.15
M1M4-1	0.10	204.92	-0.04	-0.42	-0.46	-	0.56	-0.15
WALEBING RD	0.00	207.11	-	-	-	-	-	-
WALEBING RD	0.36	206.69	-	-	-	-	-	-
WALEBING RD	0.73	206.23	-	-	-	-	-	-
WALEBING RD	1.28	205.06	-	-	-	-	-	-
WALEBING RD	1.91	204.04	-	-	-	-	-	-
WALEBING RD	2.22	203.84	-	-	-	-	-	-
WALEBING RD	2.66	203.66	-	-	-	-	-	-
BINDOON RD	0.00	-	-	-	-	-	-	-
BINDOON RD	0.45	-	-	-	-	-	-	-
BINDOON RD	0.88	-	-	-	-	-	-	-
BINDOON RD	1.45	-	-	-	-	-	-	-
BINDOON RD	1.77	-	-	-	-	-	-	-
BINDOON RD	2.21	-	-	-	-	-	-	-

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Table F.6 Reduction in March 1999 Flood Levels at the Various Cross-Sections for the Six Flood Mitigation Options, Moora Mike 11 Model

Branch Name	Branch Chainage (km)	Existing Design Flood Level (m AHD)	Reduction in March 1999 Flood Level (m AHD)					
			Option 1	Option 2	Option 3	Option 4	Option 5	Option 6
MOORE 1	0.00	208.61	0.00	-0.62	-0.60	0.00	0.00	-0.09
MOORE 1	0.32	208.39	0.00	-0.60	-0.58	0.00	0.00	-0.14
MOORE 1	0.44	208.25	0.00	-0.63	-0.60	0.00	0.00	-0.15
MOORE 1	0.86	207.68	0.00	-0.71	-0.68	0.00	0.00	-0.12
MOORE 1	1.33	207.15	0.00	-0.72	-0.70	0.01	0.00	-0.03
MOORE 1	1.58	206.88	0.00	-0.68	-0.65	0.03	0.02	0.00
MOORE 1	1.87	206.69	0.00	-0.68	-0.66	0.06	0.03	0.02
MOORE 1	1.92	206.68	0.00	-0.69	-0.67	0.06	0.04	0.02
MOORE 1	2.59	205.86	0.02	-0.38	-0.37	0.48	0.40	-0.03
MOORE 1	2.63	205.84	0.02	-0.36	-0.35	0.50	0.42	-0.01
MOORE 1	2.76	205.61	0.01	-0.30	-0.29	0.68	0.59	0.00
MOORE 1	3.18	205.11	0.00	-0.38	-0.37	-0.70	0.99	-0.08
MOORE 1	3.60	204.78	0.00	-0.47	-0.46	-0.92	1.30	-0.12
MOORE 1	3.72	204.71	0.00	-0.50	-0.49	-0.99	-0.85	-0.10
MOORE 1	3.93	204.55	0.00	-0.56	-0.54	-1.03	-0.91	-0.09
MOORE 1	4.19	204.38	0.00	-0.73	-0.69	-1.23	-1.16	-0.03
MOORE 1	4.48	204.24	0.00	-0.91	-0.86	-1.38	-1.38	0.01
MOORE 1	4.49	204.26	0.00	-0.93	-0.88	-1.39	-1.39	0.00
MOORE 1	4.54	204.25	0.00	-0.93	-0.88	-1.40	-1.41	0.01
MOORE 1	4.72	203.81	0.00	-0.62	-0.59	-1.05	-1.06	-0.15
MOORE 1	4.86	203.68	0.00	-0.58	-0.54	-0.96	-0.98	-0.10
MOORE 1	4.89	203.58	0.00	-0.55	-0.51	-0.92	-0.95	-0.18
MOORE 1	5.06	203.23	0.00	-0.43	-0.40	-0.69	-0.78	-0.07
MOORE 1	5.30	202.97	0.00	-0.44	-0.41	-0.56	-0.79	0.02
MOORE 1	5.55	202.78	0.00	-0.47	-0.43	-0.46	-0.83	0.08
MOORE 1	5.57	202.78	0.00	-0.47	-0.43	-0.46	-0.83	0.08
MOORE 1	5.60	202.54	0.00	-0.32	-0.29	-0.29	-0.67	-0.06
MOORE 1	5.78	202.40	0.00	-0.29	-0.26	-0.19	-0.62	-0.04
MOORE 1	6.08	202.07	0.00	-0.23	-0.20	0.06	-0.52	0.00
MOORE 1	6.56	201.62	0.00	-0.34	-0.31	0.28	-0.52	0.00
MOORE 1	6.69	201.60	0.00	-0.38	-0.35	0.31	-0.56	-0.02
MOORE 1	6.97	201.36	0.00	-0.32	-0.29	0.23	-0.46	0.02
MOORE 1	7.17	201.21	0.00	-0.33	-0.30	0.22	-0.47	0.05
MOORE 1	7.34	201.07	0.00	-0.29	-0.26	0.18	-0.42	0.05
MOORE 1	7.89	200.66	0.00	-0.23	-0.21	0.13	-0.28	0.03
MOORE 1	8.46	200.26	0.00	-0.28	-0.25	0.13	-0.10	0.02
MOORE 1	9.01	199.89	0.00	-0.32	-0.30	0.12	0.01	0.02
MOORE 1	9.81	199.26	0.00	-0.28	-0.31	0.11	0.07	0.02
MOORE 1	9.91	199.24	0.00	-0.28	-0.31	0.11	0.07	0.02
MOORE 1	10.11	199.07	0.00	-0.28	-0.31	0.11	0.07	0.02
MOORE 1	10.66	198.51	0.00	-0.28	-0.31	0.11	0.08	0.02
YADGENA BROOK	0.00	208.29	0.02	0.02	0.02	0.02	0.02	0.02
YADGENA BROOK	0.31	207.46	0.14	0.14	0.14	0.14	0.14	0.14
YADGENA BROOK	0.36	207.35	0.19	0.19	0.19	0.19	0.19	0.19
YADGENA BROOK	0.76	206.43	0.11	0.11	0.11	0.11	0.11	0.11
YADGENA BROOK	1.16	205.48	0.14	0.15	0.15	0.14	0.14	0.14
YADGENA BROOK	1.74	203.78	0.19	0.21	0.21	0.19	0.19	0.19
YADGENA BROOK	1.81	203.60	0.19	0.16	0.16	0.14	0.14	0.19
YADGENA BROOK	2.20	203.15	0.13	0.14	0.14	0.13	0.13	0.13
YADGENA BROOK	2.56	202.24	0.11	0.10	0.10	0.10	0.10	0.11
YADGENA BROOK	3.02	201.54	0.11	0.11	0.11	0.11	0.11	0.11
YADGENA BROOK	3.87	200.43	0.06	0.07	0.06	0.06	0.06	0.06
YADGENA BROOK	4.53	199.52	0.04	0.08	0.07	0.04	0.04	0.05
YADGENA BROOK	4.87	199.24	0.00	-0.28	-0.31	0.11	0.07	0.02
COONDEROO	0.00	201.90	0.00	-0.53	-0.49	0.68	-0.77	-0.10
COONDEROO	0.44	201.90	0.00	-0.53	-0.49	0.68	-0.77	-0.10
COONDEROO	0.80	201.90	0.01	-0.53	-0.49	0.64	-0.77	-0.10
COONDEROO	0.81	201.90	0.01	-0.53	-0.49	0.64	-0.77	-0.10

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Table F.6 Reduction in March 1999 Flood Levels at the Various Cross-Sections for the Six Flood Mitigation Options, Moora Mike 11 Model - Cont'd

Branch Name	Branch Chainage (km)	Existing Design Flood Level (m AHD)	Reduction in March 1999 Flood Level (m AHD)					
			Option 1	Option 2	Option 3	Option 4	Option 5	Option 6
MOORE 5	0.00	201.21	0.00	-0.33	-0.30	0.22	-0.47	0.05
MOORE 5	0.26	201.06	0.00	-0.50	-0.44	0.22	-0.50	0.02
MOORE 5	0.45	200.88	0.00	-0.53	-0.48	0.18	-0.36	0.02
MOORE 5	0.89	200.61	0.00	-0.59	-0.54	0.12	-0.11	0.01
MOORE 5	1.43	200.33	0.00	-0.51	-0.46	0.11	0.16	0.01
MOORE 5	1.91	199.97	0.00	-0.43	-0.40	0.11	0.11	0.01
MOORE 5	2.67	199.24	0.00	-0.28	-0.31	0.11	0.07	0.02
MOORE 2	0.00	208.39	0.00	-0.60	-0.58	0.00	0.00	-0.14
MOORE 2	0.57	207.65	0.00	-0.45	-0.43	0.00	0.00	-0.10
MOORE 2	0.96	207.26	0.00	-0.42	-0.40	0.00	0.00	-0.06
MOORE 2	1.30	206.83	0.00	-0.35	-0.33	0.01	0.01	-0.06
MOORE 2	1.63	206.39	0.00	-0.38	-0.36	0.09	0.08	-0.04
MOORE 2	2.01	205.88	0.01	-0.26	-0.25	0.33	0.39	-0.02
MOORE 2	2.12	205.80	0.01	-0.26	-0.25	0.36	0.46	-0.04
MOORE 2	2.24	205.75	0.00	-0.26	-0.25	0.39	0.50	-0.05
MOORE 2	2.99	205.37	0.01	-0.45	-0.44	0.64	0.83	-0.07
MOORE 2	3.42	205.03	0.01	-0.69	-0.68	-1.06	1.15	-0.08
MOORE 2	3.72	204.75	0.00	-0.76	-0.75	-1.05	-0.99	-0.06
MOORE 2	3.97	204.54	0.00	-0.71	-0.70	-0.98	-0.91	-0.05
MOORE 2	4.21	204.48	0.00	-0.74	-0.73	-1.00	-0.94	-0.05
MOORE 2	4.25	204.44	0.00	-0.96	-0.94	-1.40	-1.39	-0.05
MOORE 2	4.40	203.93	0.00	-0.53	-0.50	-0.96	-0.93	-0.06
MOORE 2	4.58	203.72	0.00	-0.39	-0.37	-0.86	-0.84	-0.01
MOORE 2	4.60	203.56	0.00	-0.54	-0.52	-0.80	-0.81	-0.10
MOORE 2	4.76	203.38	0.01	-0.56	-0.53	-0.70	-0.82	-0.10
MOORE 2	5.18	203.04	0.00	-0.81	-0.75	-0.51	-1.27	-0.18
MOORE 2	5.35	202.83	0.00	-0.86	-0.81	-0.32	-1.29	-0.17
MOORE 2	5.73	202.57	0.00	-0.76	-0.71	-0.07	-1.14	-0.14
MOORE 2	6.02	202.23	0.00	-0.63	-0.57	0.28	-0.94	-0.10
MOORE 2	6.31	201.92	0.00	-0.54	-0.49	0.53	-0.77	-0.09
MOORE 2	6.48	201.86	0.00	-0.50	-0.46	0.55	-0.73	-0.08
MOORE 2	6.65	201.81	0.00	-0.47	-0.43	0.53	-0.69	-0.06
MOORE 2	6.70	201.76	0.00	-0.50	-0.47	0.50	-0.70	-0.08
MOORE 2	7.15	201.60	0.00	-0.38	-0.35	0.31	-0.56	-0.02
MOORE 3	0.00	205.80	0.01	-0.26	-0.25	0.36	0.46	-0.04
MOORE 3	0.59	204.88	0.01	-0.28	-0.27	0.82	-	-0.08
MOORE 3	0.83	204.74	0.01	-0.31	-0.30	0.81	-	-0.07
MOORE 3	1.13	204.61	0.01	-0.30	-0.29	0.78	-	-0.06
MOORE 3	1.18	203.99	0.01	-0.16	-0.15	0.83	-	-0.07
MOORE 3	1.77	203.13	0.02	-0.21	-0.21	0.84	-	-0.11
MOORE 3	2.22	202.58	0.02	-0.20	-0.20	1.08	-	-0.11
MOORE 3	2.60	202.37	0.02	-0.18	-0.18	1.21	-	-0.12
MOORE 3	2.87	202.25	0.02	-0.22	-0.21	0.92	-	-0.11
MOORE 3	3.34	202.04	0.00	-0.46	-0.44	0.53	-	-0.11
MOORE 3	3.65	201.90	0.01	-0.53	-0.49	0.64	-0.77	-0.10
MOORE 3	3.91	201.86	0.00	-0.50	-0.46	0.55	-0.73	-0.08
M2-M1	0.00	207.65	0.00	-0.45	-0.43	0.00	0.00	-0.10
M2-M1	0.14	207.55	0.00	-0.48	-0.46	0.00	0.00	-0.10
M2-M1	0.51	207.09	0.00	-0.51	-0.49	0.01	0.00	-0.05
M2-M1	0.83	206.81	0.00	-0.50	-0.48	0.03	0.02	0.00
M2-M1	1.20	206.68	0.00	-0.68	-0.66	0.06	0.04	0.02
M2-M1	1.23	206.68	0.00	-0.69	-0.67	0.06	0.04	0.02
ISBISTER	0.00	207.09	0.00	-0.51	-0.49	0.01	0.00	-0.05
ISBISTER	0.40	206.70	0.00	-0.71	-0.69	0.04	0.02	-0.06
ISBISTER	0.71	206.43	0.00	-0.65	-0.64	0.09	0.07	-0.03
ISBISTER	1.11	205.79	0.02	-0.63	-0.61	0.49	0.44	-0.03
ISBISTER	1.41	205.46	0.02	-0.92	-0.86	0.78	0.73	-0.04
ISBISTER	1.76	205.14	0.01	-0.71	-0.66	-	1.02	-0.06

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Table F.6 Reduction in March 1999 Flood Levels at the Various Cross-Sections for the Six Flood Mitigation Options, Moora Mike 11 Model - Cont'd

Branch Name	Branch Chainage (km)	Existing Design Flood Level (m AHD)	Reduction in March 1999 Flood Level (m AHD)					
			Option 1	Option 2	Option 3	Option 4	Option 5	Option 6
ISBISTER	1.93	205.07	0.01	-0.97	-0.72	-	1.09	-0.06
ISBISTER	2.24	204.77	0.00	-0.81	-0.59	-	-	-0.08
ISBISTER	2.48	204.47	0.00	-0.56	-0.43	-	-	-0.04
ISBISTER	2.73	204.41	0.00	-0.50	-0.37	-	-	-0.04
ISBISTER	3.06	203.63	-0.01	-2.56	-2.41	-	-	-0.09
ISBISTER	3.26	203.29	-0.01	-2.22	-2.08	-	-	-0.13
ISBISTER	3.48	203.11	0.00	-2.04	-1.89	-	-	-0.13
ISBISTER	4.00	202.32	0.00	-1.25	-1.10	0.13	-	-0.07
ISBISTER	4.34	201.97	0.00	-0.90	-0.75	0.40	-	-0.07
ISBISTER	4.85	201.60	0.00	-0.53	-0.38	0.31	-	-0.02
MOORE 4	0.00	205.84	0.02	-0.83	-0.36	0.50	0.42	-0.01
MOORE 4	0.13	205.65	-0.11	-2.10	-0.57	0.69	0.54	-0.16
MOORE 4	0.51	205.10	-0.03	-2.06	-0.47	-	0.97	-0.10
MOORE 4	0.95	204.64	-0.01	-1.81	-0.60	-	1.43	-0.12
MOORE 4	0.97	204.64	-0.01	-1.80	-0.60	-	-	-0.12
MOORE 4	1.61	204.37	-0.01	-1.76	-0.40	-	-	-0.09
MOORE 4	1.84	204.24	-0.01	-2.16	-0.45	-	-	-0.08
MOORE 4	2.02	204.08	-0.01	-2.35	-0.51	-	-	-0.05
MOORE 4	2.13	203.92	-0.01	-2.26	-0.48	-	-	-0.05
MOORE 4	2.36	203.69	-0.01	-2.05	-0.52	-	-	-0.05
MOORE 4	2.37	203.66	-0.01	-2.11	-0.51	-	-	-0.05
MOORE 4	2.96	202.49	-0.01	-2.14	-0.48	-	-	-0.05
MOORE 4	3.16	202.04	-0.01	-2.10	-0.47	-	-	-0.04
MOORE 4	3.58	201.29	-0.01	-1.44	-0.34	-	-	-0.03
MOORE 4	3.95	200.61	0.00	-0.81	-0.59	-	-	0.01
CLINCH ST	0.00	203.69	-0.01	-2.05	-0.52	-	-	-0.05
CLINCH ST	0.37	203.23	0.00	-1.59	-0.35	-	-	-0.03
CLINCH ST	0.60	202.64	0.00	-0.98	-0.27	-	-	-0.02
CLINCH ST	0.79	201.97	0.00	-1.05	-0.23	-	-	-0.02
CLINCH ST	1.27	201.53	0.00	-0.89	-0.37	-	-	-0.01
CLINCH ST	1.53	201.15	0.00	-0.76	-0.38	-	-	0.01
CLINCH ST	1.57	201.06	0.00	-0.72	-0.50	-	-	0.02
CAMERON ST	0.00	204.64	-0.01	-1.81	-0.60	-	-	-0.12
CAMERON ST	0.33	204.51	0.00	-1.70	-0.71	-	-	-0.09
CAMERON ST	0.58	204.29	0.00	-1.47	-0.95	-	-	-0.02
CAMERON ST	0.71	204.24	0.00	-1.37	-0.91	-	-	0.01
M1M4-1	0.00	205.11	0.00	-0.59	-0.38	-0.70	0.99	-0.08
M1M4-1	0.10	205.10	-0.03	-2.06	-0.47	-	0.97	-0.10
WALEBING RD	0.00	207.35	-	-	-	-	-	-
WALEBING RD	0.36	206.94	-	-	-	-	-	-
WALEBING RD	0.73	206.44	-	-	-	-	-	-
WALEBING RD	1.28	205.25	-	-	-	-	-	-
WALEBING RD	1.91	204.19	-	-	-	-	-	-
WALEBING RD	2.22	203.96	-	-	-	-	-	-
WALEBING RD	2.66	203.92	-	-	-	-	-	-
BINDOON RD	0.00	203.60	-	-	-	-	-	-
BINDOON RD	0.45	203.62	-	-	-	-	-	-
BINDOON RD	0.88	203.62	-	-	-	-	-	-
BINDOON RD	1.45	203.63	-	-	-	-	-	-
BINDOON RD	1.77	203.63	-	-	-	-	-	-
BINDOON RD	2.21	203.92	-	-	-	-	-	-

'-'Denotes no flooding occurs