WATER RESOURCES COMMISSION

ROCKHAMPTON FLOOD MANAGEMENT STUDY

PHASE 2 REPORT

VOLUME 2
REPORT

CMPS&F

CAMP SCOTT FURPHY PTY LTD
ACN 004 939 548

NOVEMBER 1992

Carpenter

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ROCKHAMPTON FLOOD MANAGEMENT STUDY

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NOTE: Appendices unchanged from Phase 1 not included.

GLOSSARY OF TECHNICAL TERMS USED IN THIS REPORT

Term	Abbreviation	Meaning
Afflux		The increase in water level caused by the introduction of a constriction, such as a bridge, into a stream or channel.
Australian Height Datum	AHD	National Mapping datum used throughout Australia. Australia wide average of mean sea level.
Annual exceedance probability	AEP	The probability (chance) of an event (eg. flood of a given size) being equalled or exceeded in each and every year, usually expressed as a percentage.
Average recurrence interval	ARI	The reciprocal of AEP – the average period between exceedances of an event of a given magnitude, usually in years eg. 100 year ARI is equivalent to 1% AEP. This term is often misinterpreted as the actual period between exceedances rather than the average period.
Benefit–cost ratio	BCR	The ratio between economic benefits of a proposal scheme and its cost, both expressed in terms of net present value. A BCR of 1 or greater demonstrates economic viability. This is rarely achieved with flood mitigation schemes, which typically have a BCR of 0.4 – 0.7. These schemes are justified on the basis of social and other intangible ie. non monetary benefits.
Direct Flood Damage		That loss or damage caused by the physical contact of floodwaters with buildings and their contents or with other property.
Indirect Flood Damage		That loss or damage consequent upon direct flood damages. Caused by the interruption/disruption of economic or social activities as a result of direct flood damage.
Floodplain		The portion of a river valley, which is covered with water when the river overflows during floods.

Term	Abbreviation	Meaning
Levee		Embankment structure designed to protect property from damage by floodwaters by excluding flood waters from the protected area. These are usually earth embankments but may include sections of retaining walls and spillway structures.
Mean annual damage or Average annual damage	MAD AAD	The long term mean (average) of annual flood damages taking into account the probability distribution of flood magnitude and the resulting damage caused.
Net Present Value	NPV	The difference between the sum of the present value of benefits and the sum of the present value of costs. The present value of a stream of costs/benefits spread over time is their equivalent value should they be expended at the present time ie. the value of a benefit or a cost in the future discounted to a base date.
Probable Maximum Precipitation	PMP	The depth of precipitation (rainfall) which for a given area and duration can be reached but not exceeded under known meteorological conditions.
Probable Maximum Flood	PMF	The flood produced as a result of a catchment experiencing probable maximum precipitation (rainfall). Usually taken as the highest of such floods resulting from PMP of a range of durations.

SECTION 1

1. INTRODUCTION

1.1 BACKGROUND

Rockhampton is the largest urban centre in Central Queensland. The city is built on the banks of the Fitzroy River just north of the Tropic of Capricorn and some 55 to 60 km from the Fitzroy River mouth at Keppel Bay.

The Fitzroy River at Rockhampton has a catchment area of 140,200 km^2 and the Fitzroy River basin is one of the largest on the east coast of Australia.

The general location of the catchment is shown in Figure 1-1.

The basin can experience heavy rainfall, particularly in the summer months (December - March) from a variety of atmospheric conditions and synoptic processes. Major floods in the Fitzroy River are usually associated with tropical cyclones or easterly trough lows. The northern most part of the catchment inland from Sarina receives the highest rainfalls.

The Fitzroy River at Rockhampton and adjacent areas and townships have a long and well documented history of flooding. Flood records at Rockhampton date back to 1859. The worst flood since that date was in 1918 when flood levels in Rockhampton reached 10.11 m at the City gauge (8.65 m AHD). The second highest flood peak was 9.40 m gauge height (7.95 m AHD) in 1954.

Rockhampton again suffered major flooding in January 1991 due to rainfalls from Cyclone 'Joy'. The peak flood level reached 9.30 m gauge height (7.85 m AHD) in the recent flood, but due to changes in floodplain characteristics since 1954, the relativity of the 1954 and 1991 floods cannot be directly compared. In discharge terms, the 1954 and 1991 floods were almost identical with peak flows (at Yaamba) of 15,000 m^3/s compared to 18,000 m^3/s in 1918.

Major flood flows cause flooding from Yaamba to downstream of Rockhampton, and a major breakout occurs upstream of Rockhampton at the Pink Lily meander. This breakout flow can result in flooding and closure of Rockhampton Airport, the Bruce and Capricorn Highways, and the North Coast Railway. Also, the Bruce Highway and North Coast railway are cut by floodwaters at fairly high frequency at the Alligator Creek crossing near Yaamba. In the 1991 flood, all of these links were cut for about two weeks, effectively isolating Rockhampton from the outside world. This disruption to all major traffic routes in and out of Rockhampton results in large indirect flood damages not only in Rockhampton but throughout the Queensland coast. About 160 properties were inundated above floor level and 1200 to below floor level in the 1991 flood, with significant direct flood damages.

The aim of this Study is to consider all aspects of current flood management and options for future flood management in order to make recommendations aimed at reducing the impact, both tangible and intangible, of future floods.

The Study has been funded under the Federal Water Resources Assistance Program (FWRAP) and the Study reports have been prepared to facilitate application for further FWRAP funding for the recommended works.

1.2 PHASE 2

Phase 2 of the Rockhampton Flood Management Study comprises the detailed investigation of those options short listed from Phase 1, together with the formulation of final recommendations.

The following flood management measures, shortlisted for further study in Phase 1, were investigated in further detail in Phase 2:

- construction of levees to protect the flood liable areas of Port Curtis, Depot Hill and lower Central Business District (CBD);
- construction of levees to protect Rockhampton airport together with consideration of the effect of the proposed runway extension;
- selection of a control level for initiation of floodplain flows at Pink Lily;
- raising the flood immunity of the Yeppen highway/railway crossing by a combination of raising and bridge widening;
- the preparation of flood maps for a range of flood magnitudes for the urban area of Rockhampton.

Further to the round of community consultation following the publication of the Phase 1 Report, the following were added to the issues to be investigated in Phase 2:

- development of a major floodway from Pink Lily to Gavial Creek. This had been dismissed in Phase 1 on the grounds of costs and environmental impacts but has been reconsidered after being raised by members of the community;
- the effect of the Capricorn Highway on flood levels in the Fairybower area.

Fundamental to investigation of all the above was the development of a comprehensive hydraulic model to represent both existing floodplain conditions and to enable the impacts of the various measures to be studied. The hydraulic model studies formed the major component of Phase 2.

Other activities in Phase 2 included:

- preparation of new base mapping in the North Rockhampton area where Phase 1 studies had shown significant anomalies in the available contour maps;
- refinement of concept level designs for the above range of flood mitigation measures;

- refinement of cost-benefit analysis for the above;

- delineation of floodway areas and advice regarding preparation of Local Authority Floodplain Management Policies.

1.3 COMMUNITY CONSULTATION

The Phase 1 Report was published in April 1992 and a series of public meetings were held in early May 1992 to elicit the community response. These were attended by a total of 53 residents. The opportunity for further written submissions was also made at this time. Only 2 written submissions were received.

Public response to the Phase 1 report was generally positive. A summary of comments received is given below. Notes from the public meetings are given in Appendix B.

There was general support for the proposed non-structural measures, namely upgrading of the flood warning system, the installation of flood markers, provision of a recorded telephone service, flood preparedness leaflets/telephone directory entries.

There was general agreement that further consideration to upgrading the flood immunity of the Yeppen Crossing was warranted.

There was concern expressed in regard to levees, particularly property resumption impacts and flood level impact upstream. The positive effect on property values within the protected area and the potential for development of land currently liable to flooding were recognised.

Fairybower/Gracemere residents were vocal in their adverse reaction against levees both around Port Curtis/Depot Hill and the airport. Their view was that they had been disadvantaged by previous works eg. the Fitzroy River Barrage and Yeppen crossing and did not want to be further disadvantaged. Furthermore they are against contributing (by way of rates/charges) to any works which will disadvantage them.

The main issues raised which require attention are summarised below:

- **Alligator Creek Crossing**

  The Department of Transport (DOT) was requested to provide design information regarding the proposed new Alligator Creek crossing both to the Consultant and to community representatives ie. basis of design, design discharge, design afflux, assumed tailwater conditions, design drawings to show bridge length crest RL's.

  This information was subsequently provided by briefing the relevant Livingstone Shire Councillors. This has not been considered further in the study.

  The action of the Department of Transport in including upgrading of the Alligator Creek crossing in its current work program is recognised as a significant contribution to improving access northwards of Rockhampton.

- **Flood Warning Information**

  It was suggested that the proposed recorded telephone messages include Tartrus, Riverslea, The Gap, Yaamba, new floodway gauge and Rockhampton levels; the messages to be run continuously so that a repeat of the message is available. These suggestions are supported.

- **Levees**

  The following points were raised in regard to the consideration of levee options:

  - levees to be considered on an easement rather than a resumption basis where practicable;
  - local drainage within levee systems;
  - source of levee material;
  - scour protection requirements;
  - effect on flow distribution, flood levels and velocities (from hydraulic model studies).

- **Capricorn Highway**

  The hydraulic model should consider the effects on flood levels of the Capricorn Highway. The question as to why the highway is raised above general ground level was raised, as a low level crossing is acceptable because of the existence of an alternative flood free route.

- **Major Floodway Pink Lily - Midgee**

  This was raised in written submissions subsequent to the meeting as well as at one of the meetings. In summary, these submissions suggested:

  1) building a navigable canal from Lion Creek round to the Woolwash (Gavial Creek) to provide flood mitigation and a tourist facility. As proposed this would have only a small flood mitigation capacity;

  2) Construction of a floodway from Pink Lily via Murray and Yeppen Lagoons, through Yeppen crossing to the woolwash, together with levees around the airport and Depot Hill/Port Curtis/lower CBD.

  This option had been discounted previously on cost and environmental grounds. However, as a major floodway has the potential to provide substantial flood mitigation, and because of the submissions outlined above, this was given consideration in Phase 2.

A further round of community consultation will take place following publication of the Phase 2 Report.

1.4 FEDERAL WATER RESOURCES ASSISTANCE PROGRAM

This Study has been funded under the Floodplain Management Sub-Program of the Federal Water Resources Assistance Program (FWRAP).

The Study Reports have been prepared so as to facilitate submission for funding of the recommended flood management measures under this scheme.

The terms and conditions governing FWRAP funding include the following:

- completed flood maps delineating floodways and flood fringe areas are made available to the public;
- land use controls and building regulations are in operation at the local government level to prevent unwise development in identified floodways, and that all new development in flood fringe areas is above the designated flood, or is flood proofed.

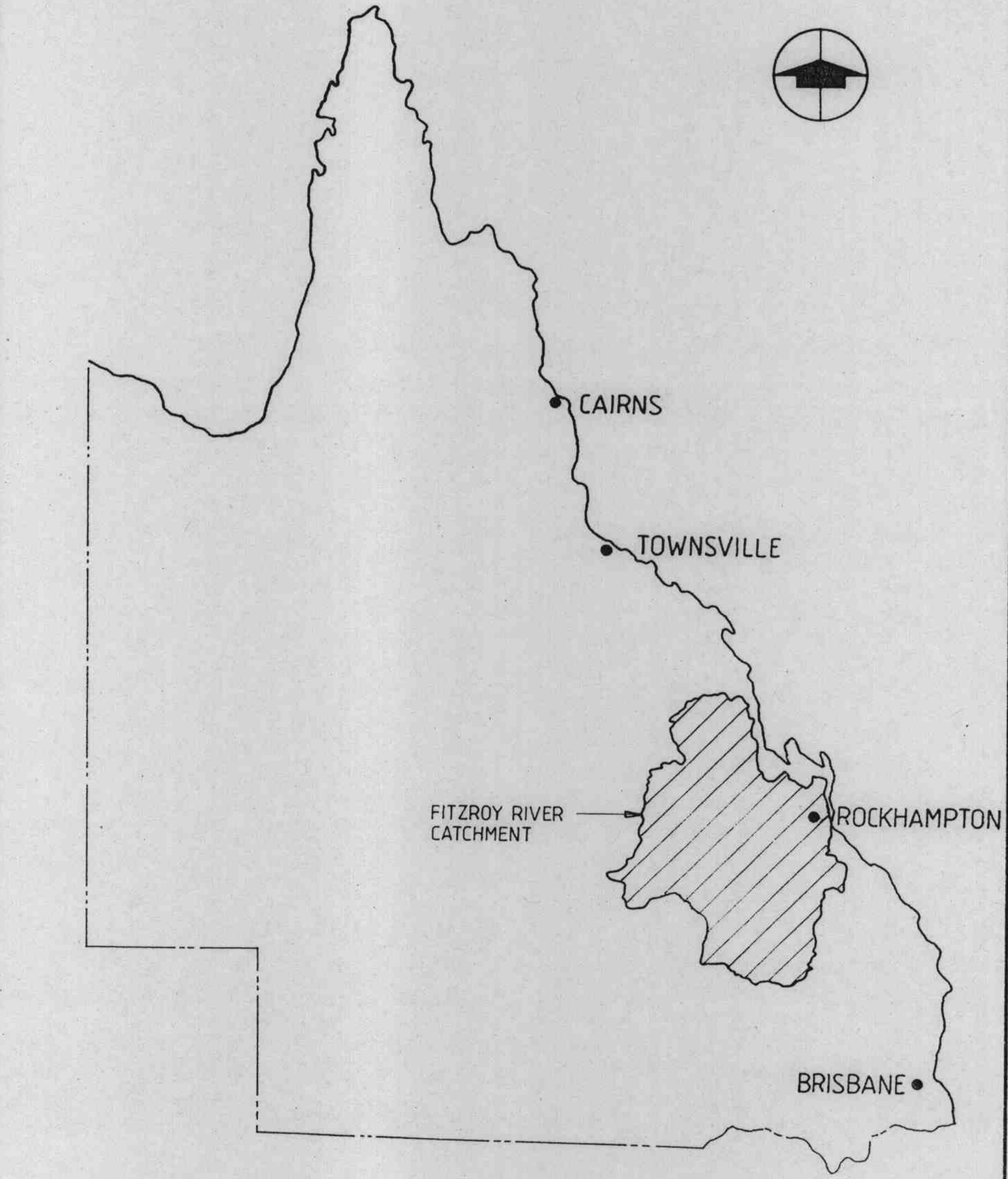
These conditions have been taken into account in formulating the recommendations presented in this Report.

The final report will be prepared to comply with the requirements for FWRAP funding applications. It should be noted that submissions for funding under FWRAP are considered on their merits and cost-effectiveness and also on priority relative to other state projects.

1.5 ACKNOWLEDGMENTS

The co-operation of officers of the Water Resources Commission, Rockhampton City Council, Livingstone Shire Council, Fitzroy Shire Council, other Commonwealth and State Government Officers, local interest and business groups and members of the public in providing input to and data for this Study is acknowledged with appreciation.

A list of the principal contributors was given in Appendix C of the Phase 1 Report.



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ROCKHAMPTON FLOOD MANAGEMENT STUDY
LOCATION PLAN

FIGURE 1 – 1

SECTION 2

2. HYDRAULIC MODEL STUDIES

2.1 GENERAL

This section describes the setting up and calibration of a mathematical model to simulate flood behaviour of the lower Fitzroy River and its associated floodplain.

This model was the single most important component of the Phase 2 studies, as the satisfactorily calibrated model was utilised for the following:

- prediction of flood levels for a range of design floods for current floodplain conditions for the preparation of flood maps;
- prediction of the effects on flood levels and flow distribution of the flood mitigation options short listed in Phase 1 of the Study.

As discussed at length in the Phase 1 Report (section 13), the previous hydraulic model studies had some shortcomings. The physical models have been broken up, hence their further use was not an option. Due to perceived shortcomings in the 1987 mathematical model, it was determined that a new model independent of previous studies was required. This section describes the objectives of the hydraulic model studies; the data available for model calibration and validation; model calibration and validation; the use of the model for flood map preparation; and consideration of flood mitigation options.

2.2 OBJECTIVES

As stated above hydraulic modelling was the single most important component of Phase 2. It was required in order to provide information in regard to the following:

- determination of the distribution of flood flows between the river and the floodplain;
- estimation of flood levels throughout the floodplain resulting from a range of flood magnitudes with existing conditions, to enable flood maps to be prepared;
- modelling of the effect on flood levels in the river and in the floodplain of those levee schemes which have merit on an economic basis;
- modelling of the effect on the distribution of flood flows and flood levels resulting from the proposed runway extension at Rockhampton Airport;
- modelling of the effect on flood levels, and duration of submergence of the Yeppen Crossing, for various combinations of increased bridge waterway area and raised embankment heights;

- modelling of the effect on flood levels of the Rockhampton City commonage landfill;

- simulation of a range of control levels at Pink Lily in order to establish the appropriate level for construction of bank stabilisation works.

2.3 DATA REQUIREMENTS AND AVAILABILITY

In order to enable the model to be calibrated, validated and used for predictive purposes the following data were required:

- topographic survey data to enable river and floodplain cross-sections to be generated;

- hydrologic data for floods used in calibration, validation and predictive modes;

- flood level data for historic floods for use in calibration and validation modes;

- details of structures in the floodplain.

These requirements are discussed in more detail in the following paragraphs.

a) Topographic Survey

One limitation of the previous models has been their limited downstream extent. Ideally, the model should extend to the ocean, with tidal levels used for model tailwater. No topographic survey information suitable for modelling purposes is available more than about 10 km downstream of the City. Field survey to obtain cross-section information from the current limits to the ocean would have been prohibitively expensive, and it was felt that this expenditure was not warranted as the modelling of the flood levels in the main area of interest was not expected to be sensitive to even broad assumptions regarding conditions in the lower section of the river. Conditions in the lower reaches were approximated by estimation of cross-sections on the basis of the available spot height information and published nautical charts. This approach enabled the model to be extended to the ocean.

In the Rockhampton City area, improved base mapping was required for flood mapping purposes. This mapping together with available river cross-sections formed the basis of cross-sectional information in the city area.

In the Yeppen floodplain, data available from previous surveys was utilised.

Topographic survey, in the form of river cross-sections, for the Pink Lily to Yaamba reach was provided by the Yaamba Oil Shale Joint Venture, whose co-operation in this regard is acknowledged.

River cross-sections in the city reach were provided by Rockhampton City Council, and in the Barrage to Pink Lily reach by the Water Resources Commission.

b) Hydrologic Data

Hydrologic data required for the model study comprised main river and tributary discharges in hydrograph form.

Historic discharges for major floods were available at Yaamba and/or The Gap. Hydrographs for The Gap for the 1991, 1988, 1983 and 1978 floods were converted to equivalent hydrographs at Yaamba by the WRC. Flood frequency curves for use in design were presented in section 4 of the Phase 1 Report. These, however, give peak discharge only. For predictive purposes, design hydrographs were prepared, based on scaling of historic hydrographs, to match the magnitude of design flows.

There are currently no data available on inflows from Alligator Creek except that measured for Hedlow Creek in 1983. A discharge hydrograph for Neerkol Creek was available for the 1991 flood, but no design hydrographs are currently available. Due to the relatively small magnitude of these inputs, these were ignored for modelling purposes except for the input of the Neerkol Creek hydrograph for modelling of the 1991 flood.

The model was calibrated using discharge hydrographs and flood levels for the 1991 and 1988 floods, as these are the only floods on record consistent with current floodplain conditions. Validation was based on the 1978, 1983, 1954 and 1918 floods. Predictive runs were based on a range of design floods (eg. 2% AEP, 1% AEP, 0.5% AEP, 0.2% AEP). Floodplain conditions under assumed extreme flood conditions were also simulated.

c) Historic Flood Levels

Peak water levels for the 1991 flood were available at a number of points along the river and in the floodplain and many of these have been utilised in the calibration of the hydraulic model. The sources of information available were:

- Rockhampton City Council;
- Department of Transport;
- Water Resources Commission.

Subsequent field survey of flood marks in the floodplain area was carried out as part of the study to supplement the above.

A number of flood levels were also available for the 1988 flood from information supplied by the Rockhampton City Council.

Historic tidal information at Port Alma was obtained from the Department of Transport for the 1978, 1988 and 1991 flood periods, for use as downstream boundary condition hydrographs.

d) **Floodplain Structures**

It was important that the model accurately reflected current floodplain conditions particularly at the major hydraulic controls. These include:

- Fitzroy River Barrage;
- Yeppen highway/railway crossing;
- Capricorn Highway;
- Fairy Bower Road;
- Nine Mile Road/Airport;
- Pink Lily meander.

Detailed information regarding the above was provided by the relevant authorities.

2.4 MODEL DESCRIPTION

The hydraulic model utilised in this study was MIKE-II. This is a state-of-the-art mathematical model for the simulation of river and floodplain dynamics, developed by the Danish Hydraulics Institute. Since its introduction into Australia in the 1980's, it has become one of the widely used such models.

The model is essentially a one-dimensional unsteady flow model which may be used in a pseudo two-dimensional form to simulate flows in complex floodplains. The model was set up to represent the Fitzroy River and its floodplain from Yaamba to the mouth of the river in Keppel Bay, a total length of 103 km.

The model must have specified boundary conditions at the upstream and downstream ends. The upstream boundary condition comprised the river discharge hydrograph (flow variation with time) at Yaamba for a specified event, whilst the downstream boundary hydrograph comprised the variation in tidal level with time as recorded at Port Alma for the duration of the flood event being modelled. A tributary hydrograph for Neerkol Creek was also utilised in regard to the 1991 flood only.

Figure 2-1 shows the overall extent of the model, whilst Figure 2-2 shows the main floodplain area around Rockhampton in greater detail. Figure 2-2 demonstrates how the model has been set up to represent a number of flow paths in the floodplain. For each of the locations shown, cross-section information was input to the model to define the model geometry. The model comprised a total of 258 cross-sections. A summary of the flow paths and the number of sections in each flow path is given in Table 2-1. Inlet and outlet geometry within each floodplain is controlled by sections represented by broad-crested weirs. Other hydraulic control sections such as Nine Mile Road were also represented in this way, as was the Fitzroy River Barrage. In regard to the Barrage, it was assumed that the gates are all fully open for the flood periods simulated. Other weirs represented high level spills between various floodplain flow paths, in order to approximate a two-dimensional flow regime. The model incorporated a total of 51 weir structures and 8 bridge structures.

The most upstream section of the model, between Yaamba and Pink Lily, was modelled in a single flow path.

The main breakout flow location in the Pink Lily area was represented by 3 flow paths, namely FP-MAIN, FP-1 and FP-2. FP-MAIN is the major flow path representing the initial point of breakout on the south-east bank of the Pink Lily Meander. FP-1 and FP-2 are the secondary breakouts to the west of FP-MAIN. These flow paths join in the Lotus Lagoon area, along with FP-3 which breaks from FP-MAIN upstream of the Rockhampton-Ridgelands Road.

The flow path FP-LION runs from FP-MAIN to the Fitzroy River along the course of Lion Creek and can flow in either direction. Two further flow paths flow out of FP-LION to represent flow on the north and south sides of the airport respectively (AP-NTH and AP-STH). These join together to the south of the airport and rejoin FP-MAIN in the Fairybower area.

FP-SCRUBBY flows off FP-MAIN to represent the proportion of flow crossing the Capricorn Highway, and a spill path CAPRICORN allows further exchange between these flow paths upstream of the Yeppen crossing.

The Bruce Highway and North Coast Railway crossing of the Yeppen floodplain was represented by a number of flow paths, called FP-MAIN, FP-SCRUBBY, FP-CURTIS, BRUCEY2 and BRUCEY3. FP-MAIN is the major floodplain flow path and includes the Yeppen 1 road and rail bridges. FP-SCRUBBY represents that part of the floodplain flow which crosses the Capricorn Highway and flows through Scrubby Creek bridges. Other flow paths (BRUCEY2, BRUCEY3) were included to represent flows through the Yeppen 2 and Yeppen 3 bridges. These rejoin FP-SCRUBBY downstream of the crossing.

The flood flows prior to overtopping are presented in each of these two flow paths by two bridge sections in series for the road and rail crossing respectively. Flow over the road/rail embankment was represented by a separate weir flow path for each of the above, plus a third weir in FP-CURTIS includes that proportion of flow over the Yeppen crossing between Yeppen 1 bridge and the city. The section between Yeppen 1 bridge and the roundabout was represented by YEPPEN1, and that south of the roundabout by YEPPEN2.

Downstream of the Yeppen crossing FP-SCRUBBY rejoins FP-MAIN and the combined flow joins FP-GAVIAL which represents flow in the Gavial Creek area of the floodplain, which itself breaks off from the Fitzroy River where Gavial Creek normally flows into the river. Spill paths between FP-SCRUBBY, FP-MAIN and FP-CURTIS downstream of Yeppen allow equalisation of levels in this region. The FP-GAVIAL flow path is terminated downstream of Edinda Lane as this is the lowest point at which topographic information is available.

From this point, all flow return to the FITZROY flow path. From this junction to the ocean, the river was modelled with extensive overbank flow areas.

Other minor flow paths simulated flow across the Pink Lily and Lakes Creek meanders.

The spill from the river into the Splitters Creek area was modelled by a flow path SPLITTERS which rejoins the river on the upstream side of the barrage.

TABLE 2-1

Summary of Model Flow Paths

Description	Flow Path Specifications			No.of Cross-sections
	Name in Model	Upstream Chainage km	Downstream Chainage km	
Fitzroy River Yaamba - Ocean	FITZROY	100.000	203.124	61
Main floodplain Pink Lily - Gavial Creek	FP MAIN	0.000	19.042	35
Gavial Creek	FP GAVIA	0.000	6.066	8
Minor flow path Lakes Creek Road	LAKESCK	0.000	2.930	9
Minor flow path across Pink Lily meander	PLILY2	0.000	1.069	7
Minor flow path across Pink Lily meander	PLILY3	0.000	1.670	8
Breakout flow path Pink Lily	FP 1	2.600	6.900	8
Breakout flow path Pink Lily	FP 2	0.000	3.200	6
Subsidiary flow path Lotus Lagoon	FP 3	0.000	4.800	7
Scrubby Creek flood flow path	FP SCRUB	0.000	5.150	14
Flood flow Port Curtis/Depot Hill	FP CURTI	0.690	4.200	8
Lion Creek	FP LION	0.000	5.300	10
Flood flow north side of airport	AP NTH	0.000	2.600	10
Flood flow south side of airport	AP STH	0.000	3.000	12
Road/rail overflow Yeppen 1	YEPPEN1	0.000	0.170	4
Road/rail overflow Yeppen 2/3	YEPPEN2	0.000	0.120	4
Spill over Capricorn Highway	CAPRICORN	0.000	0.020	2
Spills from AP Sth to AP Nth	SP AP1	0.000	0.020	2
Spills from AP STH to AP NTH	SP AP2	0.000	0.020	2
High level spill from River to FP3	SPFITZROY	0.000	0.020	2
Spill from FPMAIN to FP3	SPFPMAIN1	0.000	0.020	2
Spill from FPMAIN to FP3	SPFPMAIN2	0.000	0.020	2
Spill from FP3 to Lion Creek	SPFP3 1	0.000	0.020	2
Spill from FP3 to Lion Creek	SPFP3 2	0.000	0.020	2
Spill from AP STH to FP MAIN	SP AP3	0.000	0.020	2
Spill from FP MAIN to FP CURTIS	SPCURTIS	0.000	0.020	2
Spill from FP SCRUB to FP MAIN	SPSCRUB	0.000	0.020	2
Yeppen 2 bridge flow	BRUCEY2	0.000	0.300	7
Yeppen 3 bridge flow	BRUCEY3	0.000	0.300	7
Spill into Splitters Creek area	SPLITTERS	0.000	2.360	11
	TOTAL			258

2.5 MODEL CALIBRATION AND VALIDATION

2.5.1 Approach

In order to develop a robust model suitable for design purposes it was necessary to produce a model which performed adequately over a range of floods for which flood flow and level data were available. Performance was judged principally on the reproduction of observed water levels, within acceptable limits, and also on the timing and sequence of events such as the initiation and cessation of breakout flows.

The approach comprised two stages, namely calibration and validation. In the calibration stage, model parameters such as channel roughness were varied systematically to give reasonable agreement for the calibration events. Calibration was based on the 1991 and 1988 events. These events were chosen for a number of reasons:

- they are the two most recent events and are the only events fully representative of current conditions;

- they have the most data available in terms of flood levels together with some information regarding floodplain flows or velocities;

- they represent a reasonable range of flood magnitudes, with AEP of 8.5% and 2% respectively. The 1988 flood caused a relatively small flow in the floodplain whereas the 1991 flood caused major floodplain flow.

Once calibration was completed to a satisfactory level, the process of validation was carried out whereby the model was run with further historic floods with no further manipulation of model structure or parameters. The degree of agreement or disagreement between observed and estimated flows for these events gave an indication of the acceptability of the model, within the limitations of the available data.

Validation was carried out using the floods of 1983, 1978, 1954 and 1918. The first two of the above are post-barrage but predate reconstruction of the Yeppen crossing. The 1918 flood was included as it is the highest on record, but of course conditions in the river and floodplain have changed significantly since that time. The same comment applies to the 1954 flood, which was very similar in magnitude to the 1991 flood.

The process of calibration and validation, and the results obtained therefrom are outlined in the following paragraphs.

2.5.2 Calibration

Model calibration consisted of varying model parameters such as channel roughness, weir coefficients and, where justified, section geometry in order to achieve a satisfactory level of agreement in terms of water levels and discharge (where known) throughout the modelled area. Attention was also paid to trying to ensure coincidence of timing of peaks and overflows.

As discussed above, calibration was based on the 1991 and 1988 flood events. The approach taken to calibration was to vary the model parameters firstly in regard to the 1988 flood, whereupon the performance with 1991 conditions was assessed. Parameters were then varied as necessary and a number of iterations were required to obtain parameters giving reasonable performance for both calibration events. Further model refinement was necessary for the more severe 1991 flood, in relation to spills between the various floodplain flow paths.

Channel roughness values used in the model (Manning's n) to give the above results are given in Table 2-2.

TABLE 2-2

Model Values of Channel Resistance (Manning's 'n')

Flow Path	Location	Chainages km	Mannings 'n'
Fitzroy	Yaamba - u/s Pink Lily	100-137.2	0.047
Fitzroy	Pink Lily - Barrage	137.2-149.27	0.041
Fitzroy	Barrage - The Rocks	149.27-151.5	0.050
Fitzroy	The Rocks - Gavial Creek	151.5-154.87	0.022
Fitzroy	Gavial Creek - Edinda Lane	154.87-165.02	0.035
Fitzroy	Edinda Lane - Keppel Bay	165.02-203.12	0.042
FP Main	Yeppen - Gavial Creek	13.95-19.04	0.100
FP Scrub	Yeppen - FP MAIN	4.14-5.15	0.100
FP Curtis	Yeppen - Gavial Creek	0.91-4.2	0.100
AP NTH	Airport - North Side	0 - 2.6	0.070
AP STH	Airport - South Side	0 - 3.0	0.070
FP1	River - Lotus Lagoon	2.6 - 6.9	0.060
FP2	River - Lotus Lagoon	0 - 3.2	0.060
FP3	FP Main - Lotus Lagoon	0 - 4.8	0.060
All other Sections			0.080

The performance of the model for these two events is summarised in Tables 2-3 and 2-4 and in Figures 2-3 to 2-12. Figures 2-3 shows observed and estimated levels throughout the model in plan form. Figures 2-4 to 2-7 give modelled and observed hydrographs at key locations for the 1991 flood to enable comparison of relative hydrograph shape and timing as well as peak levels. Figures 2-8 and 2-9 show longitudinal profiles of observed and estimated flood levels along the Fitzroy River and along the main floodplain flow path (FP-MAIN) for the 1991 flood.

The corresponding diagrams relating to the 1988 flood are given in Figures 2-10 to 2-14.

Geometry of some control sections in the floodplain, where the only ground level information was the 1960 Department of Local Government (DLG) survey, were varied by a maximum of ±0.3 m to maximise agreement of observed and modelled flood levels. Discrepancies of this order exist between the DLG survey and the Australian Survey Office (ASO) surveys where these overlap in the vicinity of the Rockhampton Airport were noticed during the preparation of contour mapping in Phase 1 of the study. Hence this was believed to be justified in physical terms.

Weirs to represent overtopping of the Bruce Highway at Yeppen were based on road levels plus 200 mm to allow for kerbing/median strips. The corresponding railway crossing weirs were based on top of rail level.

The results obtained for the 2 events are discussed below.

a) Model Calibration 1991 Flood

Reference to Table 2-3 and Figure 2-3 shows a maximum discrepancy between the observed and estimated flood levels of 0.23 m. At key points, agreement is better than this, with estimated values being +0.01 m at the City flood gauge, -0.09 m at the Barrage and -0.02 m at Yaamba. Flood levels at these locations are given the greatest weight as recorders and/or staff gauges are erected at these points. Other flood levels have generally been obtained by subsequent levelling of flood markers, and are subject to a greater level of uncertainty.

Key locations in regard to floodplain flows are where breakout flows cross the Rockhampton-Ridgelands Road, where levels were estimated within 0.15 m of observed levels and at the Yeppen crossing. Estimated levels were in close agreement with observed levels on the upstream side of the crossing and within 0.04 m on the downstream side of the crossing.

This level of agreement is regarded as being satisfactory especially considering the known limitations of some of the topographic information, and the difficulty of actually recording flood levels under very bad conditions.

TABLE 2-3

Summary of Model Calibration 1991 Flood

Flow Path	Location	Chainage km	Flood Levels			Peak Discharge		Comments
			Observed Level m AHD	Modelled Level m AHD	Difference m	Observed m³/s	Modelled m³/s	
Fitzroy River	Yaamba	100.0	17.95	17.93	–0.02	14,200	14,200	Yaamba discharge input is upstream boundary condition
	u/s Pink Lily	134.0	12.36	12.26	–0.10		14,140	includes flow in Pink Lily overbanks
	start of FP2	139.2	~11.6	11.48	–0.12		13,000	includes flow in Pink Lily overbanks
	start of FP Main	140.1	~11.4	11.42	+0.02		10,200	includes flow in Pink Lily overbanks
	near Water Treatment Works	144.78	~11.2	10.98	–0.22		10,000	
	Barrage	149.27	9.59	9.50	–0.09		10,250	335 m³/s from Lion Creek
	d/s Barrage	149.47		9.27			10,250	
		150.17	8.83	8.56	–0.23		10,250	
	Railway Bridge	150.67	8.53	8.36	–0.17		10,250	
	Fitzroy Street Bridge	151.57	8.03	8.03	0.00		10,250	
	City Flood Gauge	152.57	7.85	7.84	–0.01		10,180	approx 75 m³/s in Lakes Creek flow path
	Gavial Creek	154.27	~7.5	7.59	+0.09		7,300	approx. 3,150 in Gavial Creek overflow path
	Edinda Lane	165.02		6.1			14,140	single flow path adopted from this point
		173.00	~5.6	5.44	–0.16		14,120	
FP MAIN	J/w Fitzroy River	0.0	~11.4	11.42	+0.02		1,500	total outflow FP MAIN, FP1, FP2; 4,125 m³/s
	Rockhampton–Ridgelands Road	1.75	10.9	11.04	+0.14		750	
	Lotus Lagoon	4.56	~10.8	10.99	+0.19		3,665	includes 1,970 m³/s FPI, 675 m³/s FP2 less spills to Airport area
	Nine Mile Road	6.63	~10.4	10.55	+0.15		3,412	includes input from Lion Creek
	Start FP SCRUBBY	11.0	9.32	9.44	+0.12		3,445	u/s of junction, 1,600 m³/s downstream
	Junction with AP–STH	13.0	8.98	9.16	+0.18		2,060	d/s of junction
	u/s Yeppen Crossing	13.6	8.64	8.64	0.00		1,420	spill of 640 over Capricorn Hwy to FP SCRUBBY
	d/s Bruce Highway	13.84	8.32	8.39	+0.07		1,420	bridge flow 950 overflow north of bridge 290, south 180
	d/s Yeppen Crossing	14.0	8.06	8.10	+0.04		950	
	Old Burnett Highway Old Bruce Highway	15.15 16.98	~7.8 ~7.3	7.97 7.25	+0.12 –0.05		3,150 3,150	net of spill to FP CURTIS, FP SCRUBBY includes FP SCRUBBY, but not to FP CURTIS
	J/w FP GAVIAL	19.04	~7.1	7.09	–0.01		3,150	
FP SCRUBBY	Start	0	9.32	9.44	+0.12		1,845	
	d/s Capricorn Highway	1.3	8.88	9.01	+0.13		1,845	
	u/s Bruce Highway	4.0	8.55	8.64	+0.09		2,510	bridge flow 1,550 m³/s, overflow of 960 m³/s
	d/s Railway	4.3	~8.1	8.11	+0.01		2,190	spill to FP MAIN 320 m³/s
	J/w FP MAIN	5.15	~7.9	7.97	+0.07		2,190	
FP CURTIS	u/s Bruce Highway	0.69	8.64	8.47	–0.17		290	
	Port Curtis Junction	1.4	~7.78	7.73	–0.05		290	
	Depot Hill	2.1	7.55	7.69	+0.14		290	
	Gavial Creek	4.2	7.55	7.57	+0.02		750	includes spill from FP MAIN
FP LION	J/w FP MAIN	0	~10.4	10.55	+0.15		250	
	J/w AP STH	2.2	10.44	10.44	0.00		255	
	J/w AP NTH	3.35	10.48	10.25	–0.23		185	
	Fitzroy River	5.3	9.59	9.50	–0.09		245	
Airport North	J/w FP LION	0	10.48	10.25	–0.23		75	
	New Terminal	2.12	9.33	9.45	+0.12		200	
	J/w AP STH	2.6	~9.0	9.16	+0.16		200	
Airport South	J/w FP LION	0	10.45	10.44	–0.01		385	
	Opposite Terminal	1.3		9.62			255	
	J/w AP NTH	2.3	9.0	9.17	+0.17		255	
	J/w FP MAIN	3.0	8.98	9.16	+0.18		460	d/s AP–NTH
Lakes Creek Road	near Fitzroy River Bridge	0	8.03	8.04	+0.01		75	
	Lakes Creek Road (STW)	1.5	7.39	7.34	–0.05		75	
	Lakes Creek Road (Landfill)	2.0	7.39	7.27	–0.12		75	
	J/w Fitzroy River	2.93	7.29	7.22	–0.07		75	
Gavial Creek	Fitzroy River	0	~7.5	7.59	+0.09		3,050	
	J/w FP CURTIS	1.03	~7.55	7.57	+0.02		372	d/s FP CURTIS
	J/w FP MAIN	4.21	~7.1	7.09	–0.01		6,770	d/s FP MAIN
	Edinda Lane	6.06	~6.6	6.43	–0.17		6,770	
Splitters Creek	Fitzroy River	1.34	9.67	9.51	–0.16		66	
	Fitzroy River	2.36	9.59	9.50	+0.09		66	

In terms of river discharge, of the total peak flow at Yaamba of 14,200 m³/s, the model indicates 10,250 m³/s remaining in the city reach of the river. The total estimated breakout flow at Pink Lily was 4,125 m³/s of which 335 m³/s returns to the river upstream of the barrage via Lion Creek. About 480 m³/s flows through the airport area with 3,400 m³/s passing down the main floodway across Nine Mile Road. Of this total flow, 1,300 m³/s was estimated to pass through the Yeppen 1 flow path and 2,500 m³/s through the Scrubby Creek flow path. The Yeppen 1 flow is broken down into 950 m³/s through the bridge and 470 m³/s over the road/rail embankments between the roundabout and Jellicoe Street. The combined flow through the 3 southern bridges was estimated as 1,550 m³/s with 950 m³/s overflowing the embankment south of the roundabout. The bridge flows are in good agreement with those estimated from velocity measurements taken during the flood which gave estimates of 1,020 m³/s for Yeppen 1 bridge and 1,500 m³/s for the 3 southern bridges (as given in Appendix E of the Phase 1 Report) and theoretical flows based on measured water levels of 925 m³/s and 1,580 m³/s respectively. The weir flows are, however, substantially less than the preliminary estimates in Phase 1, as those estimates did not allow for the effect of the high tailwater levels.

It was concluded that the model performed very well at the Yeppen Crossing as it reproduced both flood levels and flows to a high level of agreement with observations.

In regard to hydrograph shape and timing (refer Figures 2-4 to 2-6) there is good agreement in respect of the main river hydrographs, except that the early rise due to inputs from Alligator Creek and other local catchments has not been taken into account. As these flows were receding by the time of arrival of the main flood wave, and as they could not be quantified, their exclusion was not of concern.

The hydrograph at Yeppen crossing has a good shape in relation to the observed hydrograph, but is about 18 hours late. This is due to the model floodplain storage needing to be filled prior to flow occurring in the lower section of the floodplain, whereas in reality the heavy local rain and local runoff would have filled these storages. The overflow at Pink Lily occurred in the model early on 4th January, as it did in reality, but whereas overtopping at Yeppen occurred early on 5th January, this did not occur in the model until early on 6th January. However, the overflow duration is well modelled at 10.5 days. This delay is not believed to be a serious impediment to the operation of the model. As will be noted later, this delay did not occur in the model for 1988 in which the floodplain was dry.

The hydrographs at the airport are in good agreement (within ±0.15 m) with the records from temporary flood markers erected there, especially given that these markers do not coincide exactly with the location of cross-sections in the model.

Hydrographs from the lower reaches of the Fitzroy River show that tidal influence is negligible at the height of the flood in the area of interest, see Figure 2-7.

Longitudinal profiles along the river and the main floodplain flow path are given in Figures 2-8 and 2-9.

b) Model Calibration 1988 Flood

Reference to Table 2-4 and Figure 2-10 shows that the peak flood level at Yaamba was underestimated by 0.18 m for this event, with underestimation of 0.13 m at the Barrage, but with good agreement at the flood gauge. At the junction of Lakes Creek Road flow path with the river, the flood level was overestimated by 0.25 m. However, this was a low accuracy observation. Other discrepancies of ±0.2 m occurred at minor points.

However, a significant discrepancy of 0.62 m occurred at the Yeppen crossing, where the estimated level was 7.45 m compared to observed values of 6.83 m. The latter were taken in connection with gauging of the floodplain flow. However, these measurements show virtually the same level, on both upstream and downstream side of the highway of 6.83 m and 6.82 m respectively. This is inconsistent with the measured flows, which must have created greater afflux than 0.01 m. These levels may have been taken within the drawdown zone. As the flows were well modelled, as discussed in a later paragraph, this discrepancy is not believed to invalidate the model. On the southern side of the floodplain the estimated levels were 0.09 m high.

The total discharge for this event was 9,420 m^3/s at Yaamba. The WRC measured the discharge at Yeppen during this flood (see Appendix E2, Phase 1 Report), and recorded a peak discharge of 640 m^3/s for Yeppen 1 and a combined total of 76 m^3/s for the 3 southern bridges. Thus in this event, a total floodplain peak flow of 716 m^3/s was measured, the bulk of which stayed in the main flow path. No overtopping of the road/rail embankment occurred. The WRC report states that these flows are expected to be accurate within about ±20% due to the difficulties associated with gauging of flood flows.

In the model, the peak floodplain flows for this event were 705 m^3/s at Yeppen 1 and 110 m^3/s for the 3 southern bridges, giving a total of 815 m^3/s. This total is within 14% of the measured total, and the flow at Yeppen 1 within 10%. These values are within the noted tolerance of ±20% reported by WRC. In the model, as in reality, no overtopping of the embankments occurred. Hydrograph shape and timing was acceptable throughout, as can be seen in Figures 2-11 and 2-12. The delay in the latter which occurred in the 1991 event was not experienced in the 1988 event.

Longitudinal sections along the Fitzroy River and the main floodplain flow path are given in Figures 2-13 and 2-14.

c) Conclusion

The fitted model was able to represent floods of 9,400 m^3/s and 14,200 m^3/s (8.5% AEP and 2% AEP respectively) with predicted levels mostly within ±0.15 m at key points and generally within ±0.2 m. Floodplain flows in the 1988 flood were within 14% of measured flows, and bridge flows in 1991 were in very close agreement with those estimated from measured levels and velocities.

TABLE 2–4

Summary of Model Calibration 1988 Flood

Flow Path	Location	Chainage km	Flood Levels: Observed Level m AHD	Flood Levels: Modelled Level m AHD	Flood Levels: Difference m	Peak Discharge: Observed m³/s	Peak Discharge: Modelled m³/s	Comments
Fitzroy River	Yaamba	100.0	16.52	16.34	–0.18	9,420	9,420	Yaamba hydrograph input as upstream bordering condition
	u/s Pink Lily	134.0		11.11			9,170	include flow in Pink Lily 3 overbanks
	start of FP2	139.2		10.15			8,280	include flow in Pink Lily 2 & 3 overbanks
	start of FP Main	140.1	10.41	10.36	–0.05		8,280	include flow in Pink Lily 2 & 3 overbanks
	near Water Treatment Works	144.78	9.60	9.69	+0.09		8,280	
	Barrage	149.27	8.59	8.46	–0.13		8,365	approx 90 m³/s from Lion Creek
	d/s Barrage	149.47		8.10			8,365	
		150.17	7.76	7.57	–0.19		8,365	
	Railway Bridge	150.67		7.44			8,365	
	Fitzroy Street Bridge	151.57	7.09	7.11	+0.02		8,365	
	City Flood Gauge	152.57	6.95	6.95	+0.00		8,330	approx. 30 m³/s in Lake Creek Road flow path
	Gavial Creek	154.27	6.74	6.73	–0.01		6,150	approx. 2,200 Gavial Creek overflow
	Edinda Lane	165.02		5.32			9,030	single flow path from this point
		173.00		4.67			9,000	
FP MAIN	J/w Fitzroy River	0.0	10.41	10.36	–0.05		560	
	Rockhampton–Ridgelands Road	1.76	9.69	9.67	–0.02		340	
	Lotus Lagoon	4.56		9.45			890	includes flows from FP1, FP2
	Nine Mile Road	6.64	9.19	9.19	0.00		810	reduced by overflow into Lion Creek
	Start FP SCRUBBY	11.0		8.02			790	upstream of junction 710 downstream
	Junction with AP–STH	13.0		7.54			705	
	u/s Yeppen Crossing	13.6	6.83	7.45	+0.62		705	
	d/s Bruce Highway	13.84		7.10			705	
	d/s Yeppen Crossing	14	6.82	6.71	–0.11		705	
	Old Burnett Highway	15.52		6.67			810	
	J/w FP GAVIAL	19.04		6.16			810	
FP SCRUBBY	Start	0		8.02			110	
	d/s Capricorn Highway	1.3		6.73			110	
	u/s Bruce Highway	4.0	6.63	6.72	+0.09		110	combined flow 3 bridges
	d/s Railway	4.3	6.62	6.71	+0.09		110	
	J/w FP MAIN	5.15		6.67			110	
FP CURTIS	u/s Bruce Highway	0.69		7.45			0	no flow
	d/s Railway	0.91	¯6.83	6.71	–0.12		0	
	Depot Hill	2.1	¯6.55	6.71	+0.16		24	tidal backwater
	Gavial Creek	4.18		6.71			36	tidal backwater
FP LION	J/w FP MAIN	0		9.19			–24	flow reversal occurs
	J/w AP STH	2.2	9.25	9.08	–0.17		90	flow reversal occurs
	J/w AP NTH	3.35		8.74				flow reversal occurs
	Fitzroy River	5.3	8.59	8.46	–0.13			flow reversal occurs
Airport North	J/w FP LION	0					0	no flow
	New Terminal	1.54					0	
	J/w AP STH	2.6	7.52	7.72	+0.20		0	
Airport South	J/w FP LION	0					0	no flow
	Opposite Terminal	1.3					0	
	J/w AP NTH	2.3	7.52	7.72	+0.20		0	
	J/w FP MAIN	3.0		7.54			0	
Lakes Creek Road	near Fitzroy River Bridge	0	7.09	7.11	+0.02		30	
	Lakes Creek Road (STW)	1.0		7.10			30	
	Lakes Creek Road (Landfill)	2.0		6.42			30	
	J/w Fitzroy River	2.93	6.15	6.40	+0.25		30	
Gavial Creek	Fitzroy River	0	6.74	6.73	–0.01		2,200	
	J/w FP CURTIS	1.03		6.71			2,200	
	J/w FP MAIN	4.21		6.15			2,940	
	Edinda Lane	6.05		5.72			2,940	

Given the limitations on available topographic and cross-section data, and the expected accuracy of flood levels and flows, it was concluded that the model accurately reflects current conditions, over at least this range of flows.

Further evidence of this was sought from model validation runs for a wider range of flood flows, as outlined in the next paragraph.

2.5.3 Validation

Validation runs were carried out with the 1983, 1978, 1954 and 1918 floods. The results obtained are outlined in the following paragraphs and summarised in Tables 2-5 to 2-8.

Typical hydrographs for these events are given in Figures 2-15 to 2-18. These model runs have been carried out with no change to model structure or parameters from that giving the calibration results discussed above.

It should be noted that none of these floods are representative of current conditions.

The model calibration runs had shown that the floodplain flows were controlled primarily by the configuration at the breakouts in the Pink Lily area together with the level in the river only and not by backwater from the lower reaches of the floodplain except possibly under extreme flood conditions. Hence recent changes to the lower section of the floodplain would not impact on these aspects of model performance. Of course, modelled levels in the lower floodplain in particular will not be directly comparable to levels observed in these historic floods.

Model performance with each of these events is discussed in the following paragraphs.

a) 1983 and 1978 Floods

These floods will be considered together as they are very similar in magnitude. The 1983 flood reached 16.27 m AHD at Yaamba and 6.80 m AHD at Rockhampton. Corresponding values for the 1978 flood were 16.05 AHD and 6.70 m AHD. Discharge hydrographs at Yaamba for these events were estimated from those for The Gap by WRC. These had almost identical peak flows of 7,860 m^3/s and 7,850 m^3/s respectively. The higher level in 1983 probably results from local runoff, which is not modelled. This was expected to result in some discrepancies in levels downstream. Flows at The Gap were measured during these floods and have high reliability. The estimated flows at Yaamba are based on maintaining the measured flood volume, with some adjustments to the rating curve being required.

In both of these floods, which are smaller than either of the calibration floods, the peak level at Yaamba was significantly underestimated, by 0.59 m and 0.35 m respectively for the 1983 and 1978 events. However, it is understood that there may be errors in the recorded flood levels for these events as apparently some of the gauge boards had been replaced off the station datum.

At the Barrage, the underestimation had reduced to 0.25 m and 0.18 m respectively, and to 0.14 m, and 0.03 m at the flood gauge.

The only other location for which levels were available was for the 1978 flood in the floodplain at the Rockhampton-Ridgelands Road, where the estimated value was 0.08 m high.

It appears that flood levels were underestimated for these events, to an acceptable degree in the Rockhampton area, but an unacceptable degree at Yaamba.

A report on the 1978 flood (Dept of Transport, 1988) quoted rough estimates of floodplain flow of 225 m^3/s to 400 m^3/s. The model estimated 220 m^3/s at Pink Lily reducing to 185 m^3/s at Yeppen.

These estimates are thus in broad agreement with the approximate values available, although the significant underestimation at Yaamba was of concern.

This underestimation of flood levels may be due to any or all of the following:

- error in the fitted model;
- error in discharge hydrographs;
- observation error in observed flood levels;
- variation in channel characteristics over time ie. main river channel cross-section area less than used in the model.

b) 1918 and 1954 Floods

In order to simulate the 1918 and 1954 floods the barrage was removed from the model structure. No tidal records were available for these events, so a constant tailwater was assumed. Initially, the tailwater was set at the mean sea level (0 AHD). A sensitivity check with a constant tailwater of 3 m AHD produced only a very small difference in estimated level at Rockhampton (0.02 m at the city flood gauge for the 1918 flood).

For the 1954, the model gave reasonable agreement, overestimating the level at Yaamba by 0.28 m, overestimating by 0.15 m at Pink Lily, 0.15 m at the barrage site, with agreement within 0.05 m at the flood gauge. In regard to flows, with a peak discharge at Yaamba of 15,080 m^3/s, the model had a peak of 10,760 m^3/s in the river at the site of the barrage and a total breakout flow at Pink Lily of 4,620 m^3/s.

Considerable variation between observed and modelled levels occurred in the floodplain section, particularly upstream of the Yeppen Crossing but this was expected because of the significant changes which have occurred since 1954. In the Depot Hill area, the estimated level was within 0.2 m of that observed.

Within the limits of available data, and with recognition of the substantial changes in river and floodplain conditions, the model gave a good representation of the 1954 flood.

In regard to the 1918 flood, the estimated level at Yaamba was 0.23 m high but was 0.21 m low at the city flood gauge. In this event, with a modelled flow of 17,800 m^3/s at Yaamba, the total breakout flow at Pink Lily was estimated to be 6,370 m^3/s, with 11,600 m^3/s in the city reach of the river.

The discrepancy between observed and estimated levels at Rockhampton may be due to any or all of the following:

- model error;
- error in discharge hydrograph input to the model;
- observation error in recorded flood levels;
- variation in river cross-section with time;
- variation in control level at Pink Lily due to progressive erosion.

c) **Discussion**

In regard to levels at Yaamba, these were overestimated by 0.28 m and 0.23 m for the higher flood magnitudes of 15,000 m^3/s and 18,000 m^3/s but underestimated for the smaller floods of about 8,000 m^3/s. This suggests that the current model has relatively too great a cross-section area in the within-bank section which is counteracted at higher flows. As higher flows are the main interest, this was not a severe problem. There may also have been significant change in cross-section over the years. As discussed above, there is some doubt as to the accuracy of the recorded flood levels at Yaamba in the smaller events.

In regard to levels at Rockhampton, the validation runs have differences in the range +0.06 m to -0.21 m. Figures 13-11 and 13-12 of the Phase 1 Report show that between surveys taken in 1950 and 1989/90, bed levels in the reach from upstream of the barrage to Pink Lily (AMTD 61.16 km to 70.50 km), have lowered by as much as 3 m. Whilst no detailed data are available in regard to conditions at the time of each flood, it would be expected that each major flood would result in further degradation (erosion), possibly with some aggradation (deposition) taking place between major floods. This ongoing erosion is consistent with the change of river course in recent geological time to the current channel through the city, possible as recently as 8,000 years BP (Cameron McNamara 1981). This could account for some of this discrepancy.

Similarly, the ongoing erosion at Pink Lily has been reported to be lowering the level of the natural levee controlling the threshold of overbank flow in this area. If this were higher during the 1918 flood than it is now, a greater proportion of the flow would have remained in the river channel than predicted by the current model, hence the current model would predict lower levels for a given flood magnitude than occurred previously.

In regard to the peak flow for the 1918 flood, this has recently been revised by WRC to about 18,000 m^3/s, whereas previous estimates were about 25,000 m^3/s. Whilst the revised value is regarded as being accurate, the size of this revision, which results from reassessment of the Yaamba stage discharge rating curve, may mean that this figure is of low accuracy. A 10% underestimation in flow, for example, would probably result in a difference in water level equal to the modelled error. For example, using design flows of 19,000 m^3/s and 22,500 m^3/s see section 2.6, predicted levels at Rockhampton flood gauge were 8.59 m AHD and 9.04 AHD respectively compared to the 1918 recorded peak level of 8.65 m AHD. Thus a flow of about 20,000 m^3/s with the model would give a flood level in good agreement with the recorded 1918 level.

The main area of change appears to be related to the river cross-section information. The cross-sections for the Yaamba to Pink Lily reach used in the model are those obtained in relation to the Yaamba Oil Slate Project and are dated 1982. There has apparently been significant accretion of sand bars in the upper Barrage storage since it was commissioned in 1970. This accumulation of material and its subsequent movement during floods indicates that the stage - discharge relationship in this reach is not stable over time. As the model has a fixed geometry, it cannot reflect these transitory effects and will subsequently not adequately reflect historic flood levels in the Yaamba area.

The above discussion suggests that there have been changes in river and floodplain characteristics, since the earlier major historic floods, together with possible errors in their magnitude, which explain, to a large extent, the discrepancies between observed flood levels and those estimated using the model which has been set up to represent current conditions as closely as possible.

It was concluded from the above, that the model performs within the acceptable limits for the range of validation floods.

As such, it was concluded that the model may be utilised with acceptable confidence in the estimation of flood levels for a range of design floods for current conditions, and for consideration of the effectiveness and impact of a range of flood mitigation options. It is reiterated, that the model applies to current conditions and not to specific historic flood events other than the 1988 and 1991 events.

TABLE 2-5

Summary of Model Validation 1978 Flood

Flow Path	Location	Chainage km	Flood Levels			Peak Discharge		Comments
			Observed Level m AHD	Modelled Level m AHD	Difference m	Observed m^3/s	Modelled m^3/s	
Fitzroy River	Yaamba	100.0	16.05	15.70	−0.35	7,860	7,860	'The Gap' hydrograph used for flow at Yaamba
	u/s Pink Lily	134		10.57			7,800	
	start of FP2	139.2		9.96			7,760	Includes overbank flow at Pink Lily
	start of FP Main	140.1	9.61	9.86	+0.25		7,520	Includes overbank flow at Pink Lily
	near Water Treatment Works	144.78		9.19			7,520	
	Barrage	149.27	8.23	8.05	−0.18		7,590	
	d/s Barrage	149.47		7.71			7,590	
		150.17		7.23			7,590	
	Railway Bridge	150.67		7.09			7,590	
	Fitzroy Street Bridge	151.57		6.82			7,590	
	City Flood Gauge	152.57	6.70	6.67	−0.03		7,570	20 m^3/s in Lakes Creek Road flow path
	Gavial Creek	154.27		6.47			5,760	2,015 m^3/s in Gavial Creek overflow path
	Edinda Lane	165.02		5.12			7,730	single flow path
		173.00		4.48			7,730	tidal influence
FP MAIN	J/w Fitzroy River	0.0	9.61	9.86	+0.25		220	
	Rockhampton-Ridgelands Road	1.76	9.04	9.12	+0.08		105	
	Lotus Lagoon	4.56		8.92			260	Includes FP1, FP2
	Nine Mile Road	6.64		8.83			185	75 m^3/s into Lion Creek
	Start FP SCRUBBY	11.0		6.76			185	
	Junction with AP-STH	13.0		6.42			185	Note changed since 1978
	u/s Yeppen Crossing	13.6		6.22			185	
	d/s Bruce Highway	13.84		6.07			185	
	d/s Yeppen Crossing	14.0		5.99			185	
	Old Burnett Highway	15.15		5.96			185	
	J/w FP GAVIAL	19.04		5.79			185	
FP SCRUBBY	Start	0		6.76			0	no flow u/s Yeppen
	d/s Capricorn Highway	1.3		5.97			0	Note changed since 1978
	u/s Bruce Highway	4.0		5.97			0	
	d/s Railway	4.3		5.97				tidal flows – downstream of Yeppen
	J/w FP MAIN	5.15		5.97				
FP CURTIS	u/s Bruce Highway	0.69		6.22				tidal backwater flows only
	d/s Railway	0.91		6.45				
	Depot Hill	2.1		6.45				
	Gavial Creek	4.18		6.45				
FP LION	J/w FP MAIN	0		8.83				
	J/w AP STH	2.2		8.69			−15	flow reversal occurs
	J/w AP NTH	3.35		8.33			to	
	Fitzroy River	5.3	8.23	8.09	−0.14		75	
Airport North	J/w FP LION	0					0	no flow
	New Terminal	1.54					0	
	J/w AP STH	2.6					0	
Airport South	J/w FP LION	0					0	no flow
	Opposite Terminal	1.3					0	
	J/w AP NTH	2.3					0	
	J/w FP MAIN	3.0					0	
Lakes Creek Road	near Fitzroy River Bridge	0		6.82			20	
	Lakes Creek Road (STW)	1.0		6.80			20	
	Lakes Creek Road (Landfill)	2.0		6.20			20	
	J/w Fitzroy River	2.93		6.16			20	
Gavial Creek	Fitzroy River	0		6.47			1,825	
	J/w FP CURTIS	1.03		6.45			1,825	
	J/w FP MAIN	4.21		5.79			1,985	
	Edinda Lane	6.06		5.43			1,985	

TABLE 2-6

Summary of Model Validation 1983 Flood

Flow Path	Location	Chainage km	Flood Levels			Peak Discharge		Comments
			Observed Level m AHD	Modelled Level m AHD	Difference m	Observed m^3/s	Modelled m^3/s	
Fitzroy River	Yaamba	100.0	16.27	15.68	−0.59	7,850	7,850	Yaamba discharge input is upstream boundary condition
	u/s Pink Lily	134		10.56			7,765	includes flow in Pink Lily overbanks
	start of FP2	139.2		9.95			7,765	includes flow in Pink Lily overbanks
	start of FP Main	140.1		9.85			7,560	includes flow in Pink Lily overbanks
	near Water Treatment Works	144.78		9.18			7,560	
	Barrage	149.27	8.30	8.04	−0.26		7,570	70 m^3/s from Lion Creek
	d/s Barrage	149.47		7.70			7,570	
		150.17		7.22			7,570	
	Railway Bridge	150.67		7.08			7,570	
	Fitzroy Street Bridge	151.57		6.81			7,570	
	City Flood Gauge	152.57	6.80	6.66	−0.14		7,550	approx 20 m^3/s in Lakes Creek flow path
	Gavial Creek	154.27		6.47			5,750	approx. 1,800 in Gavial Creek overflow path
	Edinda Lane	165.02		5.11			7,720	single flow path adopted from this point
		173.00		4.49			7,720	
FP MAIN	J/w Fitzroy River	0.0		9.85			210	
	Rockhampton-Ridgelands Road	1.75		9.10			105	
	Lotus Lagoon	4.56		8.91			250	includes 30 m^3/s FPl, 15 m^3/s FP2
	Nine Mile Road	6.63		8.81			180	includes input to Lion Creek
	Start FP SCRUBBY	11.0		6.74			180	u/s of junction, 1,860 m^3/s downstream
	Junction with AP-STH	13.0		6.40			180	d/s of junction
	u/s Yeppen Crossing	13.6		6.20			180	spill of 890 over Capricorn Hwy to FP SCRUBBY
	d/s Bruce Highway	13.84		6.06			180	
	d/s Yeppen Crossing	14.0		5.98			180	
	Old Burnett Highway	15.15		5.95			180	
	J/w FP GAVIAL	19.04		5.79			180	
FP SCRUBBY	Start	0		6.74			0	no flow u/s Yeppen
	d/s Capricorn Highway	1.3		5.96			0	
	u/s Bruce Highway	4.0		5.96			0	
	d/s Railway	4.16		5.96			0	
	J/w FP MAIN	5.15		5.95			0	
FP CURTIS	u/s Bruce Highway	0.69		6.20				no flow
	d/s Railway	0.91		6.45				tidal backwater
	Depot Hill	2.1		6.45				
	Gavial Creek	4.18		6.45				
FP LION	J/w FP MAIN	0		8.81			70	
	J/w AP STH	2.2		8.67			70	flow reversal occurs
	J/w AP NTH	3.35		8.32			70	
	Fitzroy River	5.3		8.04			70	
Airport North	J/w FP LION	0					0	no flow
	New Terminal	1.54					0	
	J/w AP STH	2.6					0	
Airport South	J/w FP LION	0					0	no flow
	Opposite Terminal	1.3					0	
	J/w AP NTH	2.3					0	
	J/w FP MAIN	3.0					0	d/s AP-NTH
Lakes Creek Road	near Fitzroy River Bridge	0		6.81			20	
	Lakes Creek Road (STW)	1.0		6.80			20	
	Lakes Creek Road (Landfill)	2.0		6.19			20	
	J/w Fitzroy River	2.93		6.15			20	
Gavial Creek	Fitzroy River	0		6.47			1,820	
	J/w FP CURTIS	1.03		6.45			1,820	d/s FP CURTIS
	J/w FP MAIN	4.21		5.79			1,980	d/s FP MAIN
	Edinda Lane	6.06		5.43			1,980	

TABLE 2–7

Summary of Model Validation 1954 Flood

Flow Path	Location	Chainage km	Flood Levels			Peak Discharge		Comments
			Observed Level m AHD	Modelled Level m AHD	Difference m	Observed m^3/s	Modelled m^3/s	
Fitzroy River	Yaamba	100.0	17.89	18.17	+0.28	15,080	15,080	Yaamba discharge input is upstream boundary condition
	u/s Pink Lily	134		12.40			15,080	includes flow in Pink Lily overbanks
	start of FP2	139.2		11.59			15,030	includes flow in Pink Lily overbanks
	start of FP Main	140.1	11.34	11.53	+0.19		13,700	includes flow in Pink Lily overbanks
	near Water Treatment Works	144.78	10.82	10.91	+0.09		10,400	
	Barrage site	149.27	9.27	9.42	+0.15		10,800	360 m^3/s from Lion Creek
	d/s Barrage	149.47		9.39			10,800	
		150.17	8.90	8.77	−0.13		10,800	
	Railway Bridge	150.67		8.55			10,800	
	Fitzroy Street Bridge	151.57		8.21			10,800	
	City Flood Gauge	152.57	7.96	8.02	+0.06		10,730	approx 80 m^3/s in Lakes Creek flow path
	Gavial Creek	154.27		7.74			7,660	approx. 3,800 in Gavial Creek overflow path
	Edinda Lane	165.02		6.21			15,000	single flow path adopted from this point
		173.00		5.54			15,000	
FP MAIN	J/w Fitzroy River	0.0	11.34	11.53	+0.19		1,620	total breakout flow of 4,500 m^3/s
	Rockhampton–Ridgelands Road	1.75		11.14			770	
	Lotus Lagoon	4.56		11.10			3,910	includes 2,125 m^3/s FPI, 755 m^3/s FP2
	Nine Mile Road	6.63		10.65			3,640	includes input from Lion Creek
	Start FP SCRUBBY	11.0	8.65	9.55	+0.90		3,670	u/s of junction, 1,620 m^3/s downstream
	Junction with AP–STH	13.0	8.56	9.27	+0.71		2,200	d/s of junction
	u/s Yeppen Crossing	13.6	8.28	8.74	+0.46		1,505	spill of 720 over Capricorn Hwy to FP SCRUBBY
	d/s Bruce Highway	13.84		8.48			1,505	bridge flow 980, overflow north of bridge 220, bridge–roundabout
	d/s Yeppen Crossing	14	8.14	8.18	+0.04		1,505	
	Old Burnett Highway	15.15	8.06	8.04	−0.02		3,280	includes FP SCRUBBY, spill of 655 to FP CURTIS
	Old Bruce Highway	16.98		7.34			3,280	
	J/w FP GAVIAL	19.04	7.26	7.20	−0.06		3,280	
FP SCRUBBY	Start	0	8.65	9.55	+0.90		2,010	
	d/s Capricorn Highway	1.3	8.32	9.13	+0.81		2,010	
	u/s Bruce Highway	4.0	8.32	8.73	+0.41		2,730	bridge flow 1,570 m^3/s, overflow of 1,180 m^3/s
	d/s Railway	4.30	8.06	8.18	+0.12		2,270	spill to FP MAIN 460 m^3/s
	J/w FP MAIN	5.15		8.04			2,270	
FP CURTIS	u/s Bruce Highway	0.69	8.28	8.74	+0.46		335	
	d/s Railway	0.91	7.95	8.49	+0.54		335	
	Depot Hill	2.1	7.68	7.87	+0.19		335	
	Gavial Creek	4.18	7.51	7.72	+0.21		985	includes spill from FP MAIN
FP LION	J/w FP MAIN	0		10.65			265	
	J/w AP STH	2.2		10.54			190	
	J/w AP NTH	3.35		10.38			285	
	Fitzroy River	5.3		9.44			285	
Airport North	J/w FP LION	0		10.38			105	
	New Terminal	1.54	9.43	9.78	+0.35		210	
	J/w AP STH	2.6	8.94	9.28	+0.34		235	
Airport South	J/w FP LION	0		10.54			460	
	Opposite Terminal	1.3	9.43	9.78	+0.34		320	
	J/w AP NTH	2.3	8.94	9.28	+0.34		530	
	J/w FP MAIN	3.0	8.56	9.27	+0.71		530	d/s AP–NTH
Lakes Creek Road	near Fitzroy River Bridge	0		8.21			85	
	Lakes Creek Road (STW)	1.0		8.20			85	
	Lakes Creek Road (Landfill)	2.0		7.38			85	
	J/w Fitzroy River	2.93		7.36			85	
Gavial Creek	Fitzroy River	0		7.74			3,175	
	J/w FP CURTIS	1.03	7.51	7.72	−0.10		4,130	d/s FP CURTIS
	J/w FP MAIN	4.21	7.26	7.20	−0.15		7,370	d/s FP MAIN
	Edinda Lane	6.06		6.52			7,370	

Note: Barrage deleted, otherwise represents 1991 conditions, hence levels in flood plain section do not represent 1954 conditions.

TABLE 2–8

Summary of Model Validation 1918 Flood

Flow Path	Location	Chainage km	Flood Levels			Peak Discharge		Comments
			Observed Level m AHD	Modelled Level m AHD	Difference m	Observed m^3/s	Modelled m^3/s	
Fitzroy River	Yaamba	100.0	18.62	18.85	+0.23	17,750	17,750	Yaamba hydrograph used as upstream boundary condition
	u/s Pink Lily	134	13.54	13.35	–0.18		17,720	
	start of FP2	139.2	11.84	12.08	+0.24		15,300	2,500 in FP1
	start of FP Main	140.1		12.03			11,960	
	near Water Treatment Works	144.78		11.49			11,240	barrage deleted to model 1918 conditions
	Barrage site	149.27		9.99			11,750	includes flow from Lion Creek
	d/s Barrage site	149.47		10.03			11,750	
		150.17	9.59	9.19	–0.30		11,750	
	Railway Bridge	150.67	9.31	9.00	–0.31		11,750	
	Fitzroy Street Bridge	151.57	8.80	8.64	–0.16		11,750	
	City Flood Gauge	152.57	8.65	8.44	–0.21		11,600	160 m^3/s in Lakes Creek Road flow path
	Gavial Creek	154.27	8.53	8.15	–0.38		8,260	4,020 m^3/s in Gavial Creek flood flow path
	Edinda Lane	165.02		6.54			17,700	single flow path
		173.00		5.84			17,700	
FP MAIN	J/w Fitzroy River	0.0		12.03			2,400	total breakout flow 6,370 m^3/s
	Rockhampton–Ridgelands Road	1.76	11.40	11.56	+0.16		1,710	
	Lotus Lagoon	4.56	11.28	11.53	+0.25		4,660	includes FP1, FP2 less spills to Lion Creek
	Nine Mile Road	6.64		11.14			4,600	includes flow input via Lion Creek
	Start FP SCRUBBY	11.0		10.10			5,130	upstream of junction, 2,150 downstream
	Junction with AP–STH	13.0		9.81			2,940	940 m^3/s spills across Capricorn Highway
	u/s Yeppen Crossing	13.6		9.16			2,010	overflow across road/rail of 970 m^3/s
	d/s Bruce Highway	13.4		8.88			2,010	
	d/s Yeppen Crossing	14		8.53			1,310	
	Old Burnett Highway	15.51		8.39			4,030	spill 720 m^3/s to FP CURTIS
	J/w FP GAVIAL	19.04		7.56			4,030	
FP SCRUBBY	Start	0		10.10			3,000	
	d/s Capricorn Highway	1.3		9.68			3,000	
	u/s Bruce Highway	4.0		9.16			3,940	bridge total 1,610 m^3/s
	d/s Railway	4.3		8.54			3,940	overflow across road/rail 2,330 m^3/s
	J/w FP MAIN	5.15		8.39			2,720	
FP CURTIS	u/s Bruce Highway	0.69		9.16			540	
	d/s Railway	0.91		8.93			540	
	Depot Hill	2.1		8.44			540	
	Gavial Creek	4.18		8.13			1,900	includes spill from FP MAIN
FP LION	J/w FP MAIN	0		11.14			225	
	J/w AP STH	2.2		11.14			190	
	J/w AP NTH	3.35	11.28	11.12	–0.16		440	
	Fitzroy River	5.3		9.94			440	
Airport North	J/w FP LION	0		11.12			375	
	New Terminal	1.54		10.37			355	spills to AP STH
	J/w AP STH	2.6	9.88	9.82	–0.06		355	
Airport South	J/w FP LION	0		11.14			960	
	Opposite Terminal	1.3		10.38			475	spills to FP MAIN
	J/w AP NTH	2.3	9.8	9.88	–0.05		795	
	J/w FP MAIN	3.0		9.81			795	
Lakes Creek Road	near Fitzroy River Bridge	0		8.64			165	
	Lakes Creek Road (STW)	1.0		8.62			165	
	Lakes Creek Road (Landfill)	2.0		7.82			165	
	J/w Fitzroy River	2.93		7.74			165	
Gavial Creek	Fitzroy River	0	8.53	8.15	–0.38		3,360	
	J/w FP CURTIS	1.03		8.13			5,250	
	J/w FP MAIN	4.21		7.56			9,280	
	Edinda Lane	6.06		6.80			9,280	

Note: Barrage deleted, otherwise represents 1991 conditions, hence levels in flood plain section do not represent 1918 conditions.

2.6 DESIGN FLOODS

2.6.1 General

Following completion of the calibration/validation stage, the hydraulic model was used to simulate water levels resulting from a range of design floods.

This served the following purposes:

- estimation of water levels for a range of flood magnitudes for the development of flood maps;
- estimation of velocities for the delineation of floodways;
- to enable a comparison of water levels under existing conditions with those pertaining to a range of flood mitigation options.

2.6.2 Design Inputs

The required design inputs are:

- flood discharge hydrographs at Yaamba;
- water level hydrographs at the ocean;
- model structure for existing conditions as per model calibration runs.

The design hydrographs were based on the peak discharges given in Table 4-8 of the Phase 1 Report. For floods of 5%, 2% and 1% AEP the shape of the 1991 hydrograph was adopted and scaled to give the appropriate peak discharge. For the more extreme floods (0.5%, 0.2% and 0.1% AEP), the same principle was adopted but the longer duration 1918 flood hydrograph was used. The design hydrographs are plotted in Figure 2-19.

As the calibration runs had shown that, in the area of interest, the model was not sensitive to or influenced by the tidal levels, the design runs were based on a constant tailwater level of 0 m AHD (mean sea level). Sensitivity testing with levels as high as 3 m AHD produced insignificant effect in the region of interest.

2.6.3 Modelled Flood Levels

The results of these runs are shown in Tables 2-9 and 2-10. Table 2-9 shows the distribution of flows between the river and the floodplain over a range of flood discharges of 11,500 m^3/s to 24,000 m^3/s (5% AEP to 0.1% AEP). The 2% AEP flood is almost identical to the 1991 event.

The lower floodplain flow at Yeppen compared to that breaking out of the river at Pink Lily reflects the small proportion returning to the river via Lion Creek, relative timing and storage effects.

TABLE 2-9

Summary of Peak Discharges in Design Runs

Flow Path	Location	Peak Discharge (m^3/s) for AEP of					
		5%	2%	1%	0.5%	0.2%	0.1%
Fitzroy River	Yaamba	11,500	14,200	16,400	19,000	22,500	24,000
	Barrage	9,150	10,250	11,100	12,100	13,400	14,000
Floodplain	Breakout at Pink Lily	2,435	4,130	5,600	7,400	9,810	10,850
	Yeppen Crossing						
	– bridge flow	2,100	2,500	2,650	2,670	2,675	2,680
	– overflow	200	1,410	2,600	4,420	6,920	7,920
	– total	2,300	3,910	5,250	7,090	9,595	10,600

Comparison of this distribution of flows between the river and the floodplain with those from the previous model studies (Table 13–1 of the Phase 1 Report) shows these to be consistent with the two physical models but with substantially greater floodplain flow than the 1987 mathematical model.

Figure 2–20 presents the flow frequency curve from Table 2–9 in graphical form. From this figure it can be seen that the proportion of breakout flow to the total flow increases with the severity of the flood. Data from the previous model tests have been added to this figure. These have been plotted to match the frequency curve for total flow and also show the flow distribution in the earlier models. This illustrates that the current model is broadly consistent with the previous physical models but not with the earlier mathematical model. As the latter was found to give inconsistent results in the review carried out in Phase 1 the latter does not detract from the current model.

Table 2–10 summarises the peak flood levels at a number of locations in the river and the floodplain for the range of flows considered. Levels for floods more extreme than 1% AEP should be regarded as tentative as they may exceed the levels of topographic information.

Comparing the range of levels at Yaamba and at the City Flood Gauge with those given in Table 4–8 of the Phase 1 Report shows that modelled levels at Yaamba are generally above those estimated directly from the sequence of flood level records, converging to a similar value at 0.1% AEP. The figures for Rockhampton are higher than those in Table 4–8 for the less extreme floods but lower for the more extreme floods. As stated in the Phase 1 report, the validity of the values for Rockhampton especially given in Table 4–8 was questionable due to changes in the floodplain characteristics over the years which were reflected in levels reached by certain floods.

A frequency curve based on the modelled values is included in Figure 2–20.

TABLE 2-10

Summary of Peak Flood Levels for Design Runs

Flow Path	Location	Chainage km	Peak Flood Levels (m AHD) for AEP of					
			5%	2%	1%	0.5%	0.2%	0.1%
Fitzroy River	**Yaamba**	100.0	17.11	17.93	18.52	19.14	19.88	20.18
	u/s Pink Lily	134.0	11.74	12.26	12.69	13.14	13.72	13.96
	start of FP2	139.2	10.97	11.48	11.90	12.32	12.87	13.10
	start of FP Main	140.1	10.88	11.42	11.84	12.27	12.82	13.05
	near Water Treatment Works	144.78	10.23	10.86	11.32	11.77	12.34	12.57
	Barrage	149.27	8.93	9.49	9.91	10.35	10.90	11.13
	d/s Barrage	149.47	8.58	9.17	9.60	10.07	10.65	10.89
		150.17	8.02	8.55	8.96	9.39	9.91	10.13
	Railway Bridge	150.67	7.85	8.35	8.74	9.15	9.64	9.84
	Fitzroy Street Bridge	151.57	7.54	8.03	8.40	8.79	9.26	9.45
	City Flood Gauge	152.57	7.37	7.84	8.21	8.59	9.04	9.23
	Gavial Creek	154.27	7.13	7.58	7.94	8.30	8.73	8.91
	Edinda Lane	165.02	5.71	6.10	6.38	6.68	7.03	7.18
		173.00	5.05	5.43	5.69	5.97	6.30	6.43
FP MAIN	J/w Fitzroy River	0.0	10.88	11.42	11.84	12.27	12.82	13.05
	Rockhampton-Ridgelands Road	1.75	10.47	11.04	11.41	11.75	12.24	12.47
	Lotus Lagoon	4.56	10.37	10.99	11.38	11.73	12.23	12.46
	Nine Mile Road	6.63	9.93	10.55	10.95	11.39	11.92	12.15
	Start FP SCRUBBY	11.0	8.83	9.43	9.89	10.35	10.87	11.09
	Junction with AP-STH	13.0	8.48	9.15	9.60	10.04	10.54	10.74
	u/s Yeppen Crossing	13.6	8.06	8.64	9.00	9.32	9.67	9.83
	d/s Bruce Highway	13.84	7.83	8.39	8.71	9.06	9.48	9.64
	d/s Yeppen Crossing	14.0	7.60	8.10	8.36	8.75	9.24	9.42
	Old Burnett Highway	15.15	7.49	7.96	8.22	8.60	9.07	9.24
	Old Bruce Highway	16.98	6.73	7.24	7.47	7.83	8.27	8.47
	J/w FP GAVIAL	19.04	6.67	7.08	7.38	7.72	8.13	8.29
FP SCRUBBY	Start	0	8.83	9.43	9.89	10.35	10.87	11.09
	d/s Capricorn Highway	1.3	8.17	9.00	9.46	9.91	10.40	10.61
	u/s Bruce Highway	4.0	7.91	8.63	8.99	9.31	9.66	9.82
	d/s Railway	4.3	7.59	8.10	8.37	8.77	9.27	9.45
	J/w FP MAIN	5.15	7.49	7.96	8.22	8.60	9.07	9.24
FP CURTIS	u/s Bruce Highway	0.69	7.13	8.64	8.96	9.32	9.67	9.83
	Port Curtis Junction	0.91	7.26	8.37	8.76	9.10	9.49	9.66
	Depot Hill	2.1	7.12	7.67	8.19	8.67	9.19	9.37
	Gavial Creek	4.2	7.11	7.56	7.91	8.30	8.74	8.89
FP LION	J/w FP MAIN	0	9.93	10.55	10.95	11.39	11.92	12.15
	J/w AP STH	2.2	9.79	10.43	10.90	11.45	12.09	12.33
	J/w AP NTH	3.35	9.39	10.25	10.86	11.45	12.09	12.34
	Fitzroy River	5.3	8.93	9.49	9.91	10.35	10.90	11.13
Airport North	J/w FP LION	0	9.39	10.25	10.86	11.45	12.09	12.34
	New Terminal	1.54	8.64	9.61	10.15	10.64	11.20	11.43
	J/w AP STH	2.6	8.64	9.16	9.61	10.05	10.55	10.76
Airport South	J/w FP LION	0	9.79	10.43	10.90	11.45	12.09	12.33
	Opposite Terminal	1.3	8.59	9.61	10.15	10.65	11.20	11.43
	J/w AP NTH	2.3	8.48	9.16	9.61	10.05	10.55	10.76
	J/w FP MAIN	3.0	8.48	9.15	9.60	10.04	10.54	10.74
Lakes Creek Road	near Fitzroy River Bridge	0	7.54	8.03	8.40	8.79	9.26	9.45
	Lakes Creek Road (STW)	1.0	7.53	8.01	8.39	8.77	9.22	9.41
	Lakes Creek Road (Landfill)	2.0	6.84	7.26	7.61	7.99	8.48	8.69
	J/w Fitzroy River	2.93	6.80	7.21	7.54	7.88	8.29	8.45
Gavial Creek	Fitzroy River	0	7.136	7.58	7.94	8.30	8.73	8.29
	J/w FP CURTIS	1.03	7.11	7.56	7.92	8.28	8.71	8.89
	J/w FP MAIN	4.21	6.66	7.08	7.38	7.72	8.13	8.29
	Edinda Lane	6.06	6.12	6.43	6.65	6.92	7.25	7.39
Splitters Creek		1.1	8.93	9.51	9.92	10.54	11.16	11.42
		2.1	8.93	9.50	9.91	10.38	10.98	11.23

Note: Levels for floods of AEP < 1% are tentative.

Longitudinal profiles for the Fitzroy River and the main floodplain flow for the modelled range of design floods are given in Figures 2-21 to 2-23.

The values given by design runs of the model, whilst they include model error, do present a consistent set of values being based on 1991 conditions, as modelled.

2.6.4 Flood Mapping

The flood levels obtained from these design runs were utilised to produce flood maps for existing conditions as described in Section 4.

2.7 FLOOD MITIGATION OPTIONS

2.7.1 General

Following completion of the calibration/validation process the model was modified to simulate the impacts of a number of flood mitigation options on flood levels and flow distribution.

This section describes the use of the model in this context and is limited to a consideration of the hydraulic impacts of such options. A discussion of the flood mitigation options themselves including a summary of the hydraulic aspects but also considering costs, social and environmental impacts is given in Section 3 hereof.

Those options listed in section 1.2 were considered firstly on an individual basis and then in various combinations, as described in the following paragraphs.

The range of options and impacts considered was:

- levee construction: Port Curtis - Depot Hill - Lower CBD and Depot Hill - Lower CBD only;
- levee construction: airport including the effect of the proposed runway extension;
- levee construction: Splitters Creek;
- improving flood immunity of the Yeppen Crossing, together with lessening the impact on upstream flood levels;
- reduction in floodplain flows by raising breakout control levels in the Pink Lily area;
- construction of a major floodway to the south of the city, either in whole or in part;
- impact of Commonage Landfill;
- lowering the elevated section of the Capricorn Highway.

These options were considered principally in relation to 2% and 1% AEP floods. The following paragraphs discuss the findings for these options.

2.7.2 Levees Port Curtis - Depot Hill - Lower CBD (Options A1, A2)

a) Option A1 - Depot Hill - CBD Only

The proposed levee around Depot Hill and lower CBD (Option A1) but excluding Port Curtis (scheme 2 on Figure 14-5 in Phase 1 Report) would not have significant impact on flood levels as the area protected is primarily flood storage and not a high velocity floodway. This was borne out by modifying the model to account for reduction in flow cross-section in the FP-CURTIS flow path, the results of this and other runs being given in Appendix J (Table J-1).

The impact of this scheme would be to raise the peak level in the river by a maximum of 0.03 m at the City flood gauge for 2% AEP flood and 0.04 m for 1% AEP. Levels in the Port Curtis flow path adjacent to the levee would be raised by 0.08 m, 0.09 m for 2%, 1% AEP. Elsewhere in the floodplain modelled level differences are negligible. It is considered that the above increases are acceptable.

b) Option A2 - Port Curtis - Lower CBD

The combined Port Curtis - Depot Hill - Lower CBD levee (Option A2) has a significantly greater impact on flood levels as it effectively blocks the FP-CURTIS flow path. The impact of this levee option was modelled by removing this flow path, and its associated spills, from the model. The results for this option are given in Table J-2.

This levee scheme would raise the level in the main floodway downstream of Yeppen Crossing by about 0.6 m for 2% AEP flood and 0.9 m for 1% AEP. This, in itself, is not a problem as there is little development in this part of the floodplain. Of greater impact, is the increase in the flood level on the upstream side of Yeppen crossing of 0.30 m for 2% AEP, and 0.42 m at 1% AEP. This impact reduced to near zero at Nine Mile Road for 2% AEP but was still 0.14 m at 1% AEP. In the airport region, if this were not itself protected, levels would be raised by about 0.07 m near the terminal and 0.14 m at the southern end and in the Fairybower Road area for 2%, 1% AEP floods.

It is unlikely that the above would be acceptable without some compensatory works. The following were considered in this regard:

- lower levels downstream of (and hence also upstream of) Yeppen Crossing by removal of old embankments and/or channel works;
- lower levels upstream of Yeppen Crossing by means of works at the crossing itself to increase bridge waterway area;
- reduce floodplain flow by raising breakout levels at Pink Lily.

These combinations are considered in Section 2.7.11.

2.7.3 Levee Construction at Rockhampton Airport (Options A3, A4)

Two scenarios were considered in regard to Rockhampton airport, namely:

- levees to provide protection to the existing airport up to 1% AEP;
- as above, but allowing for the proposed runway extension to the north extending across Lion Creek.

a) Protection to existing airport (Option A3)

The effect of protection of the existing airport was modelled by removing the flow paths AP-NTH and AP-STH and their associated spills.

The effect of this would be to reduce the capacity of the floodway and increase the proportion of the breakout flow from the Pink Lily area returning to the river via Lion Creek. This results (see Table J-3) in increased levels in Lion Creek of up to 0.37 m for 2% AEP and 0.58 m for 1% AEP, and a small increase of 0.03 m for 2% AEP and 0.07 m for 1% AEP at the Barrage, reducing to 0.01 m, 0.02 m respectively at the City Flood Gauge. Between the Pink Lily breakout and Nine Mile Road, levels in the upper part of the floodway would be increased by up to 0.1 m and 0.2 m at 2% AEP and 1% AEP respectively. At the Yeppen Crossing, the level would be reduced by 0.04 m at 2% AEP and 0.08 m at 1% AEP due to a small decrease in floodplain flow which results from the greater return flow via Lion Creek mentioned above.

b) Protection of Extended Airport (Option A4)

The proposed extension of the main runway to the north-west along Lion Creek would have a more profound effect on floodplain flows. Whilst it is anticipated that low flows from Lion Creek would either be carried under the runway in a culvert, or diverted around the northern boundary, the capacity of such drainage works is likely to be limited. As no details are available of the works which would be required, the effect of these works has been modelled approximately by severely restricting the capacity of the centre sections of FPLION and FP3.

Results given in Table J-4 show very little difference from the previous case of protection of the existing airport with maximum increase of 0.37 m for the 2% AEP flood and 0.58 m for 1% AEP flood in Lion Creek due to the redistribution of flows.

2.7.4 Levee Construction - Splitters Creek Area (Option A5)

A levee along the left bank (looking downstream) of the Fitzroy River near Splitters Creek (scheme 9 on Figure 14-5 of Phase 1 Report) would prevent the overflow occurring in that area and hence reduce flooding of this mainly residential area.

The impact of this on flood levels in the river was modelled by removing the SPLITTERS flow path. The results are given in Table J-5 which show the effect of this to be minimal.

2.7.5 Yeppen Crossing (Options B1-B9)

Works at Yeppen crossing would have 2 potential impacts, namely:

- reduction in closure times of this major crossing and hence reduction of indirect flood damage for the whole area;
- reduction in flood levels in the Fairybower Road area.

The first could be achieved primarily by raising the level of the approach embankments, say to the bridge levels, and the second by increasing the bridge waterway area.

These were studied initially separately and then in combination, as outlined below:

a) Increased Waterway Area (Options B1, B6)

Increased waterway area was considered both from bridge widening and lowering of bridge inverts. The current bridging length is 420 m. Increasing the bridge waterway area to twice the current amount was considered (Option B1). Results from this run are given in Table J-6. It is outside the scope of the present study to provide final design data, so if this proposal is adopted further analysis will be required to finalise bridge dimensions.

With doubling of the bridge waterway area (assuming each bridge would be doubled in length), the flood level on the upstream side of the Yeppen crossing would be reduced by 0.27 m and 0.29 m in 2% and 1% AEP floods respectively. Times of submergence would be reduced by 1.85 days (to 9.75 days) and 0.72 days (to 11.95 days) respectively. At the airport, levels would be reduced by 0.08 m and 0.14 m at the terminal area and southern end of the runway respectively for both 2% and 1% AEP floods. Flood levels would be reduced by 0.06 m and 0.10 m at Depot Hill for 2% and 1% AEP events with corresponding reduction at Port Curtis of 0.1 m and 0.25 m respectively due to reduced flow in the FP-CURTIS flow path. Levels in the main floodplain flow path FP-MAIN would be increased marginally by 0.05 m, 0.08 m downstream of the Yeppen crossing. There was an insignificant effect on levels in the city reach of the river.

Discharges through the bridges would increase by about 30% only but the afflux caused by the bridges would be significantly reduced. Velocity through the various bridges would range from 1.2 m/s to 1.7 m/s at 2% AEP, compared to 1.9 m/s to 2.8 m/s under existing conditions.

An alternative means of increasing waterway area would be to lower the invert level (ie. the bed level) below each bridge. The feasibility of this is considered in section 3, this section reports only the hydraulic effects. This possibility was investigated assuming a reduction in bed level of 2 m. This reduction was assumed to continue between the road and rail bridges with transition back to existing surface levels upstream of the highway bridges and downstream of the railway bridges. The result of this run (Option B6) is given in Table J-10.

This was found to have as nearly a beneficial impact on flood levels as bridge duplication, but with rather less improvement in regard to time of submergence. Bridge velocities were reduced compared to existing conditions and were in the range 1.4 to 2.0 m/s at 2% AEP.

b) Increased embankment height (Options B3, B4)

The existing road and rail crossings are higher at the bridges than in between. The simplest way of increasing embankment height to reduce closure time would be to increase the level of the road/rail sections between bridges to that at the bridges themselves. This represents a maximum increase of about 1.0 m. An intermediate increase of 0.5 m was also considered. Greater increases were not considered due to the need then to raise the bridges.

The results from these model runs (Tables J–7 and J–8) showed that with embankment heights raised to give constant road and rail height across the bridges and embankments, but with no change to waterway area, there would be an increase in flood level on the upstream side of Yeppen crossing of 0.38 m for 2% AEP and 0.31 m for 1% AEP. The corresponding increases at the Airport (terminal area) would be 0.16 m and 0.11 m increasing to 0.23 m, 0.19 m at the southern end of the airport and in the Fairybower Road area. Time of submergence would reduce by 4 days to 7.6 days for 2% AEP and by 3 days to 9.63 days for 1% AEP.

The entire crossing including the bridges would be overtopped in both of the events considered.

c) Combinations of the above (Options B5, B7)

Following the above separate consideration of various measures, their combined effect was investigated.

The combined effect of embankment raising (to bridge level) and doubling of the existing bridge waterway area was modelled as Option B5, the results being given in Table J–9.

This combination would not be overtopped in a 2% AEP flood providing a continuous kerb is provided between bridge sections, and would have a time of submergence in 1% AEP flood of 6.82 days, a reduction of 5.85 days. This combination would also result in a reduction in flood levels upstream of the Yeppen crossing of 0.17 m for 2% AEP reducing to 0.05 m for 1% AEP. At the airport (terminal) the reductions would be 0.05 m, and 0.02 m respectively, increasing to 0.09 m, 0.03 m at the southern end of the airport and at Fairybower Road. Bridge flow velocities at 2% AEP would be in the range 1.2 m/s to 2.0 m/s.

Flood levels would also be lowered in the Depot Hill area due to reduced overflow in the FP–CURTIS flow path. This effect is greatest just downstream of the railway where levels would be 0.78 m, 0.63 m lower for 2% and 1% AEP floods compared to existing conditions. At Depot Hill these differences would be 0.08 m, 0.15 m respectively.

Slightly higher levels would result in the main floodway downstream of Yeppen crossing because of the reduced flow in FP-CURTIS. However, this is a maximum of 0.06 m, 0.11 m for 2% and 1% AEP floods immediately downstream of the crossing and is of little consequence.

The combination of raising embankment height to give constant level across the crossing, and reducing invert levels through the bridges was considered as Option B7. The results of this run are summarised in Table J-11. This combination would also result in the crossing not being overtopped at 2% AEP, but would have an increased submergence time at 1% AEP of 8.0 days compared to 6.8 days with Option B5. Bridge flow velocities would be in the range of 1.7 to 2.3 m/s for 2% AEP, thus scour protection would need to be provided.

Flood levels would be reduced upstream of Yeppen Crossing by 0.03 m for 2% AEP, but would be increased by 0.14 m for 1% AEP compared to existing conditions. Airport flood levels would be reduced by only 0.01 m for 2% AEP, but increased by 0.04 m for 1% AEP. Levels at Depot Hill would be reduced by 0.08 m, 0.12 m respectively for 2%, 1% AEP floods.

As Option B7 still had an adverse impact on levels upstream of Yeppen for 1% AEP, two further means of reducing these levels were considered, namely:

- removing the Old Burnett Highway bridge and causeway and the section of disused railway embankment adjacent to the Old Bruce Highway between Port Curtis and Roopes Bridge (Option B8);

- as above together with construction of 200 m wide channel from downstream of the Yeppen 1 bridge to continue the lowered invert to below Edinda Lane (Option B9).

The results from these runs are given in Tables J-12 and J-13.

Option B8 produced a substantial improvement over Option B7. Flood levels upstream of Yeppen Crossing were reduced by 0.41 m for 2% AEP and by 0.09 m for 1% AEP, with reductions at the Airport of 0.15 m, 0.25 m respectively. Time of submergence was again zero for 2% AEP and reduced to 3.7 days for 1% AEP.

Option B9 produced only marginal benefit over Option B8 and was clearly not worthwhile. This conclusion would be reinforced when costs were also considered.

2.7.6 Control of Breakouts at Pink Lily and Gavial Creek (Options D1 to D4)

The threshold level at which flows commence in the main floodplain is controlled by the bank levels along the right bank of the Pink Lily meander. As discussed in section 13.4 of the Phase 1 Report, stabilisation is required to limit the continuing erosion of the meander, both because of its lateral progression but also because the control level will reduce as erosion progresses.

Stabilisation works in this area would also provide the opportunity to alter the control level. Raising the control level and/or reducing the length over which breakout flow takes place would have the effect of reducing flow and hence flood levels in the floodplain, but at the expense of raising flood levels in the City reach of the river. Conversely, lowering the control level, which would require excavation, would increase flows and the incidence of flows occurring in the floodplain and reduce flow and flood levels in the City reach.

It was considered that increasing flows in the floodplain by this means would be unacceptable, as would significantly raising flood levels in the city reach. However, as varying the control level could be a means, for example, of compensating the effect on floodplain levels of levee construction at Port Curtis, the impact of raising the control levels by 1.0 m was investigated (Option D1). This was done by raising the inlet weir levels in floodplain flow paths FP-MAIN, FP1 and FP2 by these amounts. The results of this run is given in Table J-14.

Raising the control level by 1.0 m was effective in lowering the 2% AEP flood level on the upstream side of Yeppen crossing by 0.22 m but this effect reduced to 0.09 m for 1% AEP flood. Levels at the airport were lowered by a maximum of 0.45 m in the terminal area for 2% AEP and 0.14 m for 1% AEP. Levels at Port Curtis were lowered by 0.20 m and 0.08 m respectively for these 2 flood magnitudes.

Conversely, flood levels at the Barrage were raised by 0.32 m for 2% AEP and 0.14 m for 1% AEP, with corresponding values at the City flood gauge being 0.10 m and 0.04 m respectively.

Extension of this principle was explored by further raising the Pink Lily breakout levels by an extreme amount, sufficient to prevent breakout flow occurring under a 2% AEP flood (Option D2). This was found to require breakout control levels to be raised to about 13.2 m, corresponding to an embankment with maximum height of about 4.5 m. If this were acceptable, it would eliminate the need for improvements at Yeppen Crossing. The results of this run (Table J-15) show that this would be effective in the latter regard, with Yeppen Crossing flood free at 1% AEP, and flood levels upstream of Yeppen being reduced by 1.57 m for 2% AEP and 1.29 m for 1% AEP. However, this would also cause significant rise in the flood levels in the city reach of the Fitzroy River. Increases in level at Pink Lily would be 1.3 m, 1.66 m for 2% and 1% AEP floods. Corresponding increases at the Barrage would be 1.14 m, 1.35 m and 0.49 m, 0.51 m at the City Flood gauge. It was considered that such an increase in levels in the city reach would not be acceptable.

Levels in the Depot Hill area would be worsened because of higher flows in the river and the Gavial Creek flow path and these levels were estimated to be increased by up to 0.25 m.

A compromise between Options D1 and D2 was subsequently tested, Option D3 with breakout level raised by 2.5 m. The results for this are given in Table J-16. In this case levels upstream of Yeppen were reduced by 0.71 m for 2% AEP and by 0.49 m for 1% AEP, sufficient to reduce time of submergence to 6.5 days and 9 days for 2% and 1% AEP respectively. Flood levels in the river would be raised by a maximum of about 1.2 m at Pink Lily for both events, 0.71 m for 2% AEP and 0.55 m for 1% AEP at the Barrage and 0.27 m, 0.19 for 2%, 1% AEP at the City flood gauge.

None of the above were considered to be suitable as stand alone measures but they were considered further in regard to combinations of measures.

As a further means of reducing floodplain flow the effect of reducing floodplain tailwater level by increasing the level at which breakout occurs for the Fitzroy River near the Gavial Creek junction was briefly investigated as Option D4. The results of this run are given in Table J–17. This was found to be ineffective in producing a marked reduction in floodplain levels but did increase river level by up to 0.7 m (at Gavial Creek) for 2% AEP. This was not considered further.

2.7.7 Improving Hydraulic Capacity Downstream of the Pink Lily - Yeppen - Gavial Creek Floodway (Options F3, F4)

Improving the hydraulic capacity of the floodplain downstream of Yeppen Crossing was investigated in two stages, namely:

a) removal of the bridge and causeway on the Old Burnett Highway together with the removal of disused railway embankment adjacent to the Old Bruce Highway (Option F3);

b) as above, together with excavation of a channel 900 m wide with invert level at 4.2 m AHD to the junction of the FP–MAIN and FP–GAVIAL flow paths (Option F4).

a) Option F3

The results of model runs to test these options are given in Tables J–18 a, b from which it can bee seen that the effect on flood levels and times of submergence at Yeppen are minimal.

The maximum reduction in water level with Option F3 for 1% AEP would be 0.07 m upstream of Yeppen and 0.17 m downstream with a reduction of times of submergence of only 0.04 days for 1% AEP and 0.6 days for 2% AEP.

This options would reduce flood levels in Port Curtis and Depot Hill by about 0.1 m for both 2% and 1% AEP floods.

Whilst this option is not sufficient alone, it was considered further in regard to combinations of measures, as discussed in 2.7.5 c.

b) Option F4

In regard to Option F4, the scope for channel improvement is limited because of very flat gradients in this area. In order to give an indication of the potential for lowering tailwater levels at Yeppen by this means, a model run was carried out with a channel at constant bed level of 4.2 m from downstream of Yeppen Crossing to Gavial Creek and with a bed width of 900 m, thus representing a major channel. The effect of this was to lower the flood level on the upstream side of Yeppen Crossing by 0.11 m for 2% AEP and 0.09 m for 1% AEP which is only a marginal improvement relative to removing the Old Burnett Highway bridge and the disused rail embankments adjacent to the Old Bruce Highway (Option 3). Similarly, downstream of Yeppen Crossing the water levels were lowered by 0.28 m for 2% AEP and 0.23 m for 1% AEP compared to 0.17 m for 1% AEP for Option F3.

Time of submergence of Yeppen Crossing was only marginally reduced by 0.8 days for 2% AEP and 0.67 days for 1% AEP for Option F4. This option would reduce water levels at Port Curtis and Depot Hill by about 0.16 m for both 2% AEP and 1% AEP floods.

This option was not considered to be worth pursuing.

2.7.8 Major Floodway (Option E1)

The option of a major floodway had been discounted in Phase 1 because of limited effectiveness, high cost and high environmental impact. It was reintroduced subsequent to being raised in written submissions received as part of the community consultation process. It has, therefore, been investigated in Phase 2 using the hydraulic model.

The floodway was modelled as a major channel with 1,000 m wide base width and 1,000 m wide right overbank channel, on a constant grade from the upstream part of the Pink Lily meander to Gavial Creek. Due to the very large nature of such a channel, it was assumed that new bridging would be incorporated as necessary. The remainder of the Pink Lily meander would need to have levees constructed to prevent outflow outside of the flood relief channel.

The results are given in Table J-19.

Levels would generally be higher in the channel than would occur under existing conditions. The capacity of the modelled channel was 3,570 m^3/s at 2% AEP and 4,580 m^3/s at 1% AEP, which is insufficient to have any beneficial impact on flood levels in the city reach. This option did not warrant further consideration.

2.7.9 Commonage Landfill (Option M1)

The impact the commonage landfill has on flood levels was investigated by removing the LAKESCK flow path from the model, thus simulating the effect of completely blocking the flow path. The results of model runs are given in Table J-20.

These runs show that this have a negligible impact, raising flood level at the City Flood gauge by 0.01 m for both 2% AEP and 1% AEP.

This has no impact on flows and levels in the main floodplain.

2.7.10 Lowering Capricorn Highway (Option M3)

Parts of the Capricorn Highway between the Bruce Highway roundabout and the edge of the floodplain near Gracemere are raised above existing ground level by up to about 1.5 m. The effect of lowering the highway, in the raised sections, by 1 m was investigated, and the results given in Table J-21.

This was found to be ineffectual, lowering the level on the upstream side of the Capricorn Highway by a maximum of 0.1 m for 2% AEP with a subsequent increase in level on the downstream side due to flow redistribution. For a 1% AEP flood, the impact was negligible. Due to the ineffectual nature of the option, this was not considered further.

2.7.11 Other Combinations (Options C1 to C10)

In addition to the options discussed above the following combinations were tested. Comparison of these options on economic and impact grounds is held over until Section 3 hereof. All the combined options include upgrading of Yeppen Crossing in order to enable substantial reductions to be made in indirect losses. Options C1 – C5 are based on lowering the inverts under the existing Yeppen bridges as well as raising embankment levels to bridge height. Options C6 – C10 are based on bridge duplication and embankment raising.

Option C1
- combination of the Levee from Port Curtis to the CBD (Option A2) with Option B8 to upgrade Yeppen Crossing (raise embankment, lower inverts, demolish old highway bridge and remove old railway embankment).
- this would provide protection to the flood liable areas of Port Curtis, Depot Hill and the CBD and would also raise the flood immunity of the Yeppen Crossing to 2% AEP.
- the results for this run are given in Table J–23. The flood level upstream of Yeppen would be reduced by 0.03 m for 2% AEP, but increased by 0.24 m at 1% AEP. In the Fairybower area the corresponding figures were –0.05 m, 0.11 m respectively.
- time of submergence at Yeppen Crossing would be zero at 2% AEP, 8 days at 1% AEP.

Option C2
- as for C1 plus levee to protect Rockhampton airport and a levee to prevent overflow in the Splitters Creek area.
- substantially as above – see Table J–24. The presence of the airport levee reduces levels in the Fairybower – Yeppen section (as for Option A3), with this combination reducing levels upstream of Yeppen by 0.09 m for 2% AEP, but increasing level by 0.17 m for 1% AEP.
- levels along Lion Creek adjacent to the levee would be raised by a maximum of 0.36 m at 2% AEP and by 0.59 m at 1% AEP.
- levels along the Rockhampton–Ridgelands Road would be raised by 0.05 at 2% AEP and by 0.12 m at 1% AEP.
- time of submergence at Yeppen Crossing would be zero for 2% AEP and 7.8 days for 1% AEP.

Option C3
- as Option C1 but with Port Curtis excluded from the protected area. Obviously this is to the detriment of Port Curtis but still provides levee protection to Depot Hill and the Lower CBD including the area subject to backwater flooding from the main drain.
- this has a positive impact on flood levels in the main floodway for 2% AEP with a reduction of 0.24 m upstream of Yeppen Crossing (see Table J–25). For 1% AEP the level upstream of Yeppen is unchanged from current conditions.

- time of submergence of Yeppen Crossing would be zero at 2% AEP and 7 days at 1% AEP.

Option C4
- as Option C3 but with levees around the airport and to prevent overflow into Splitters Creek. The results given in Table J–26 show that levels at Yeppen would be further reduced compared to Option C3. Levels upstream of Yeppen would be 0.3 m lower for 2% AEP than under existing conditions, and 0.06 m lower for 1% AEP.
- time of submergence of Yeppen crossing would be zero for 2% AEP and 6.5 days for 1% AEP.

Option C5
- as Option C2 but with breakout threshold levels at Pink Lily raised by 1.25 m. This was modelled to compensate for the worsening of peak flood levels upstream of Yeppen Crossing in Option C2. Results are given in Table J–27. This would result in reductions in levels at Yeppen of 0.53 m for 2% AEP and 0.02 m at 1% AEP compared to existing conditions. However, levels in the river would rise, by 0.50 m and 0.33 m for 2%, 1% AEP near the water treatment works, 0.30 and 0.20 m at the Barrage and 0.17 m, 0.08 at the City flood gauge. This is the minimum raising at Pink Lily which would cause no worsening of levels upstream of Yeppen.

Option C6 – C10 are similar to options C1 – C5 in the combinations given below except that they are based on duplication of the Yeppen bridges instead of invert lowering.

Option C6
- as per Option C3, see Table J–28 for results.
- the peak flood level upstream of Yeppen was 0.16 m lower for 2% AEP and 0.07 m lower for 1% AEP.
- corresponding levels in the Fairybower road area would be 0.09 m, 0.03 m lower.
- time of submergence at Yeppen would be zero for 2% AEP, 6.9 days for 1% AEP.
- marginal increases in river level of 0.02 m maximum would occur.

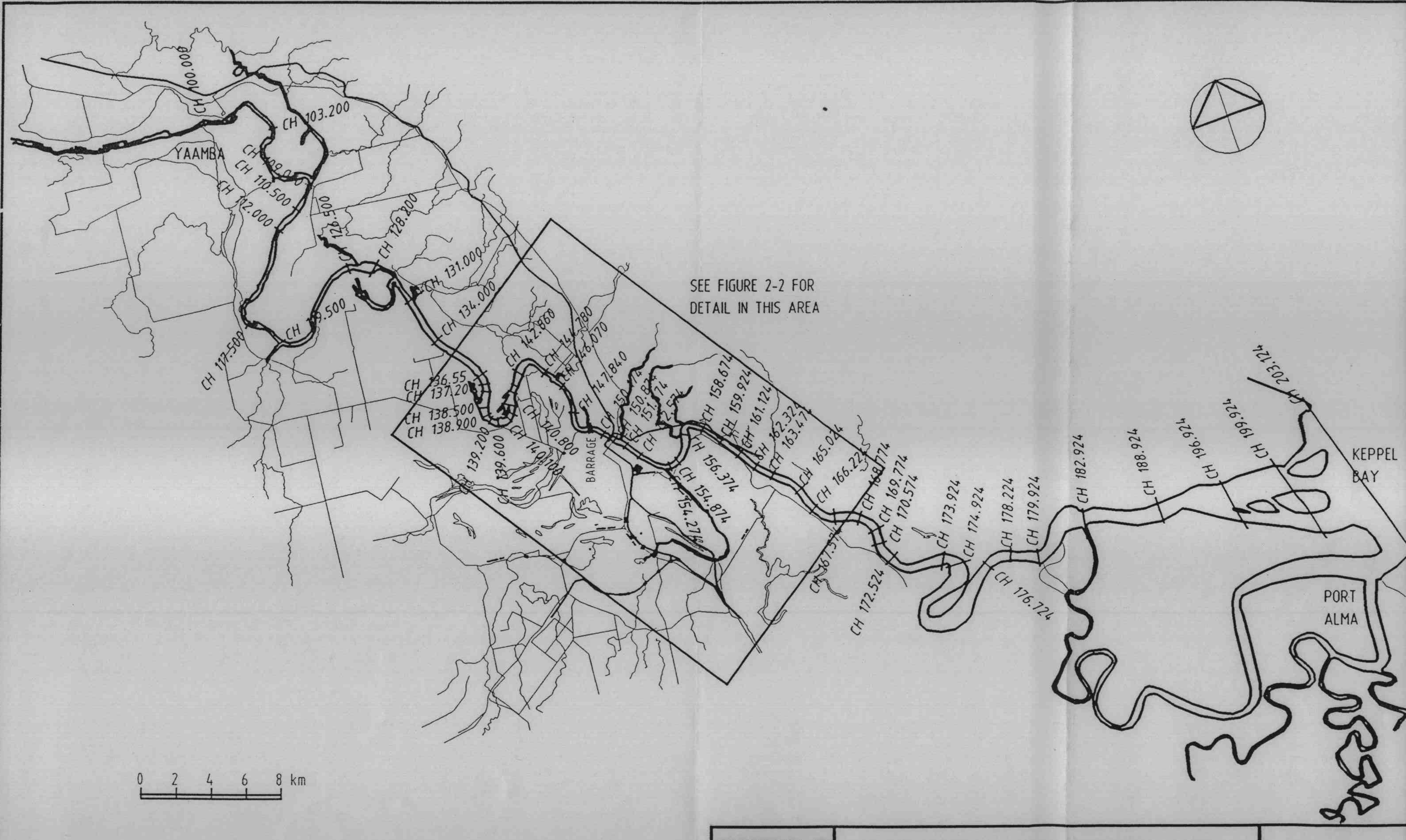
Option C7
- as per Option C4, see Table J–29 for results.
- the peak flood level upstream of Yeppen would be 0.20 m lower for 2% AEP and 0.13 m lower for 1% AEP.
- corresponding levels in the Fairybower area would be 0.18 m, 0.19 m lower.
- levels along Lion Creek would be raised by 0.37 m, 0.55 m maximum outside the airport levee.
- levels in the river would be increased by a maximum of 0.04 m, 0.08 m for 2%, 1% AEP between the water treatment works and the barrage.
- time of submergence at Yeppen would be zero for 2% AEP, 6.4 days for 1% AEP.

Option C8
- as per Option C1, see Table J–30 for results.
- the peak flood level upstream of Yeppen was 0.28 m lower than for existing conditions for 2% AEP and 0.02 m lower for 1% AEP.

- Levels in the Fairybower area would be decreased by 0.15 m, 0.02 m for 2%, 1% AEP respectively.
- Time of submergence would be zero for 2% AEP, 6.5 days for 1% AEP.

Option C9
- as per Option C2, see Table J-31 for results.
- the peak flood level upstream of Yeppen would be reduced by 0.33 m for 2% AEP and by 0.09 m for 1% AEP.
- levels in the Fairybower area would be reduced by 0.25 m in 2% AEP and 0.17 m in 1% AEP.
- levels in the river between Pink Lily and the City would be raised slightly by a maximum near the water treatment works of 0.03 m for 2% AEP, 0.06 m for 1% AEP, and reduced marginally in the City reach (0.02 m, 0.06 m at the City flood gauge for 2%, 1% AEP).
- time of submergence at Yeppen would be zero for 2% AEP and 3.5 days for 1% AEP.

Option C10
- as per Option C9 but with threshold level at Pink Lily raised by 1.25 m, instead of removal of the old highway bridge and disused rail embankment, see Table J-32 for results.
- this would reduce the level upstream of Yeppen crossing by 0.44 m for 2% AEP and by 0.01 m for 1% AEP.
- Maximum increase in level in the river would be 0.45 m higher for 2% AEP, 0.27 m for 1% AEP at Pink Lily, reducing to 0.26 m, 0.15 m for 2%, 1% AEP at the barrage, and 0.10 m for 2% AEP, zero increase for 1% AEP at the City flood gauge.
- there would be small increases of 0.02 m, 0.01 m for 2%, 1% AEP at Yaamba.



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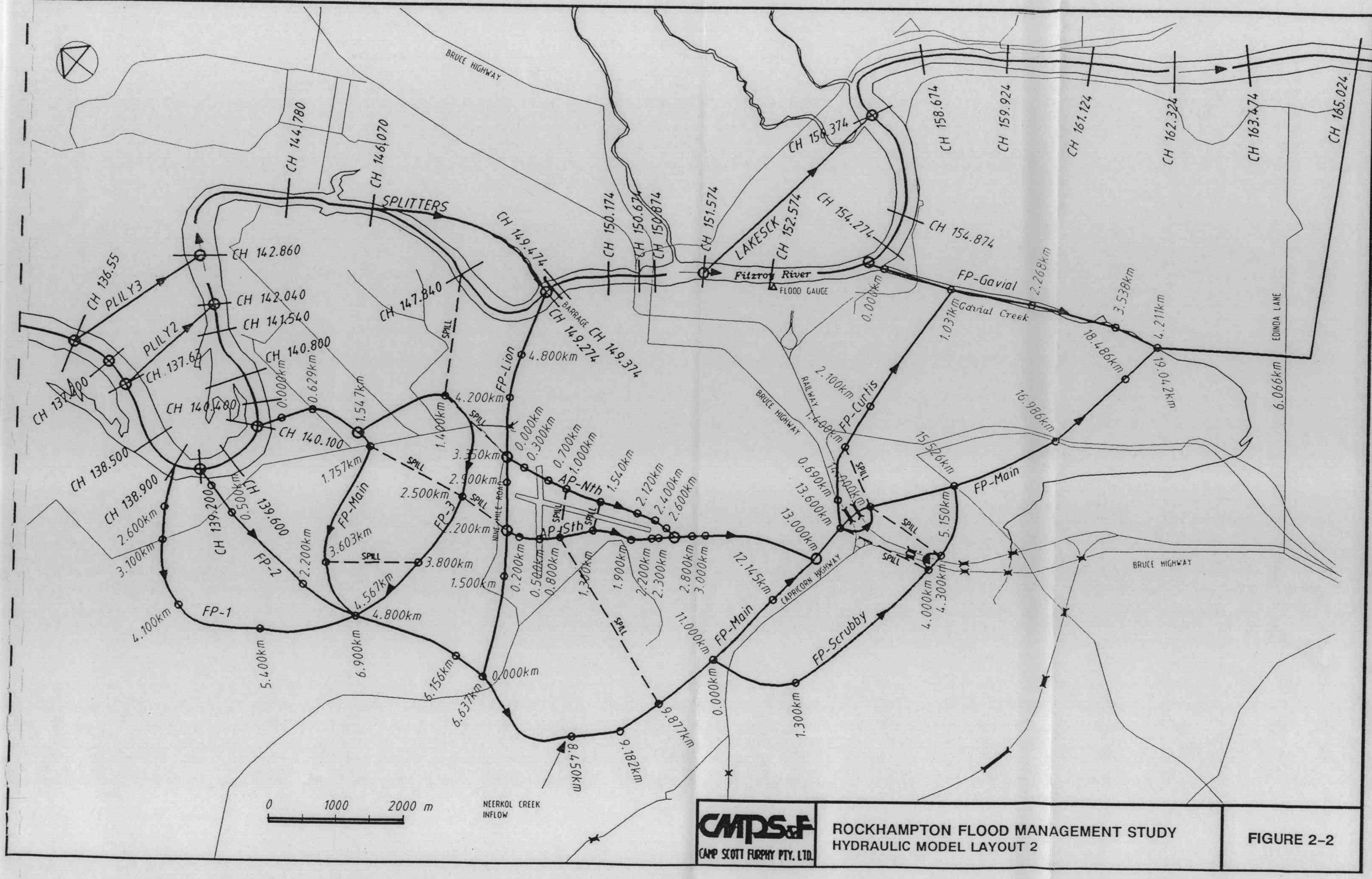
CAMP SCOTT FURPHY PTY. LTD.

ROCKHAMPTON FLOOD MANAGEMENT STUDY
HYDRAULIC MODEL LAYOUT 1

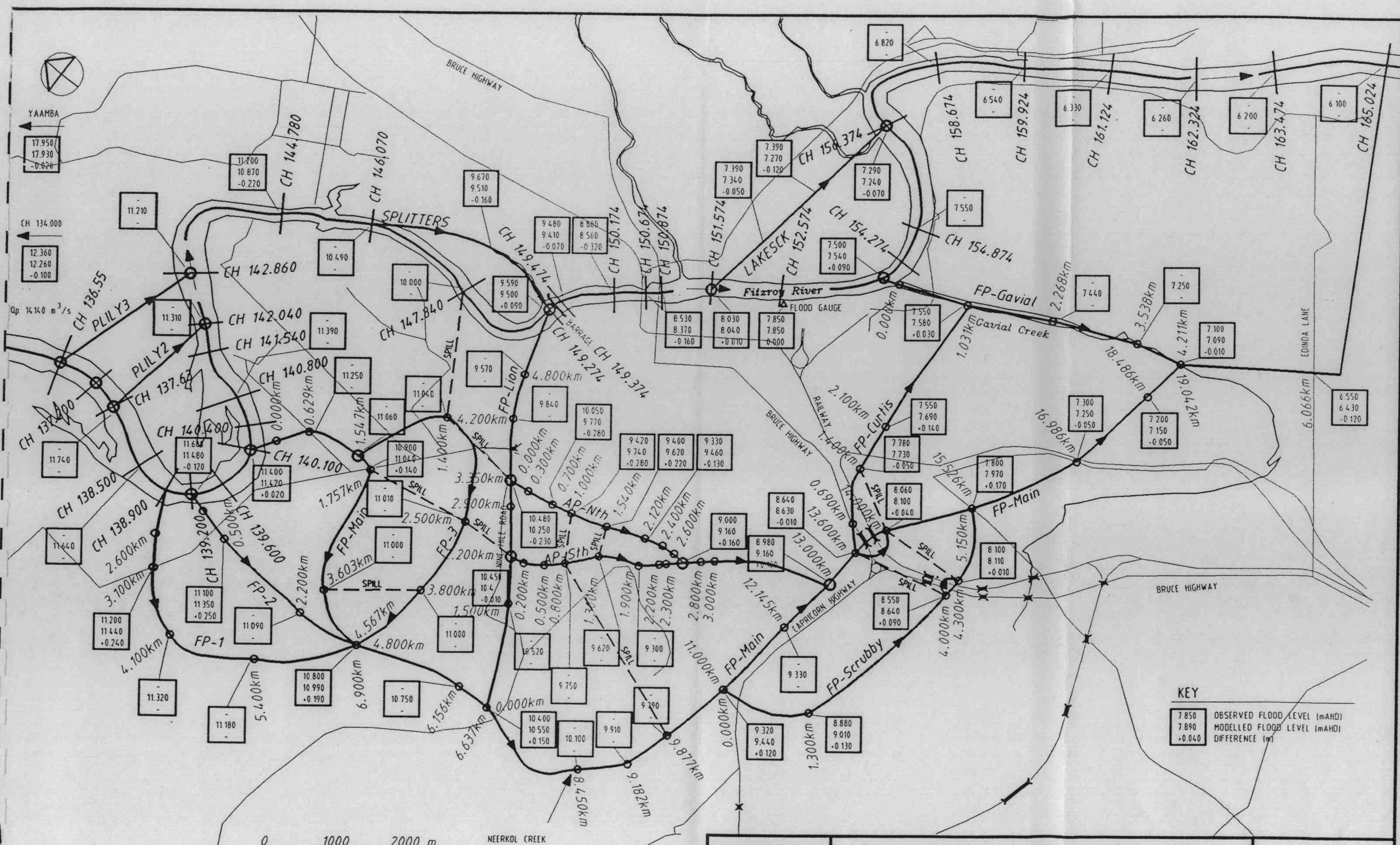
FIGURE 2–1



CMPS&F CAMP SCOTT FURPHY PTY. LTD.

ROCKHAMPTON FLOOD MANAGEMENT STUDY
HYDRAULIC MODEL LAYOUT 2

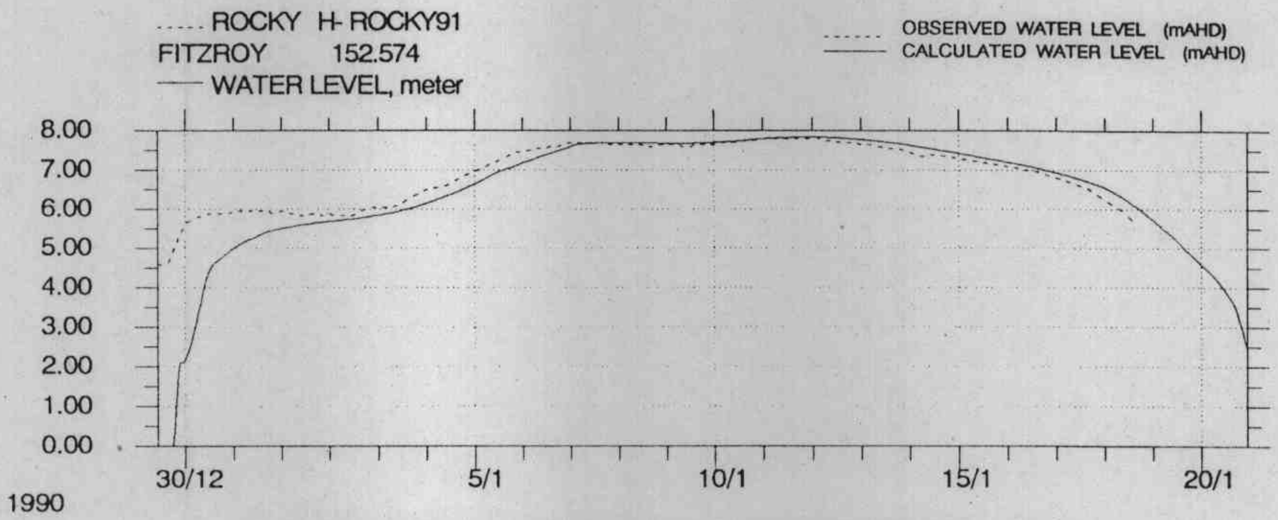
FIGURE 2-2



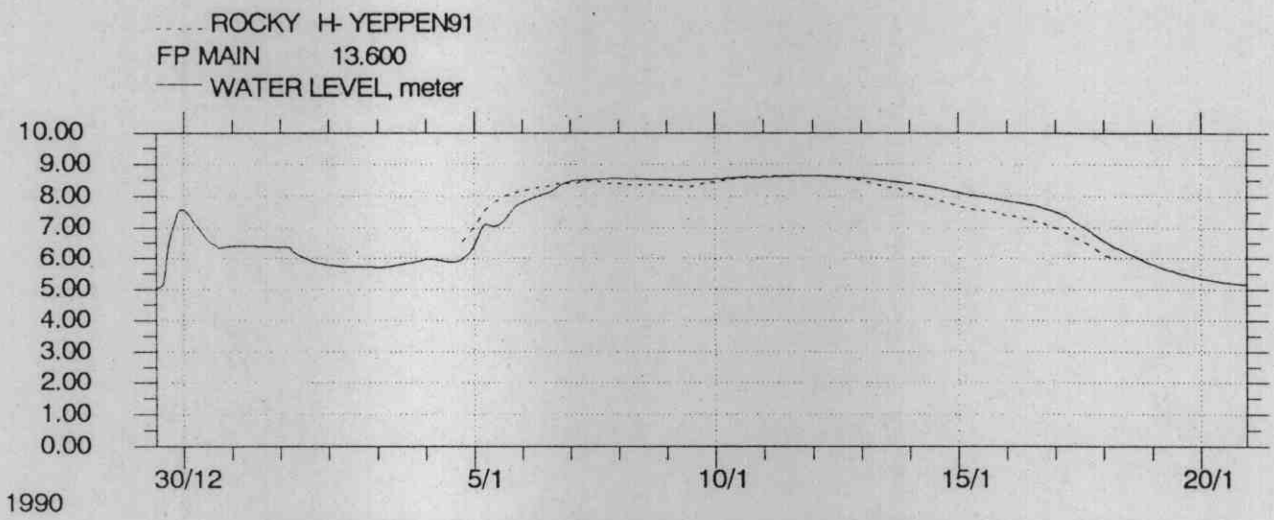
CMPS&F CAMP SCOTT FURPHY PTY. LTD.

ROCKHAMPTON FLOOD MANAGEMENT STUDY
HYDRAULIC MODEL LAYOUT 2
CALIBRATION 1991 FLOOD

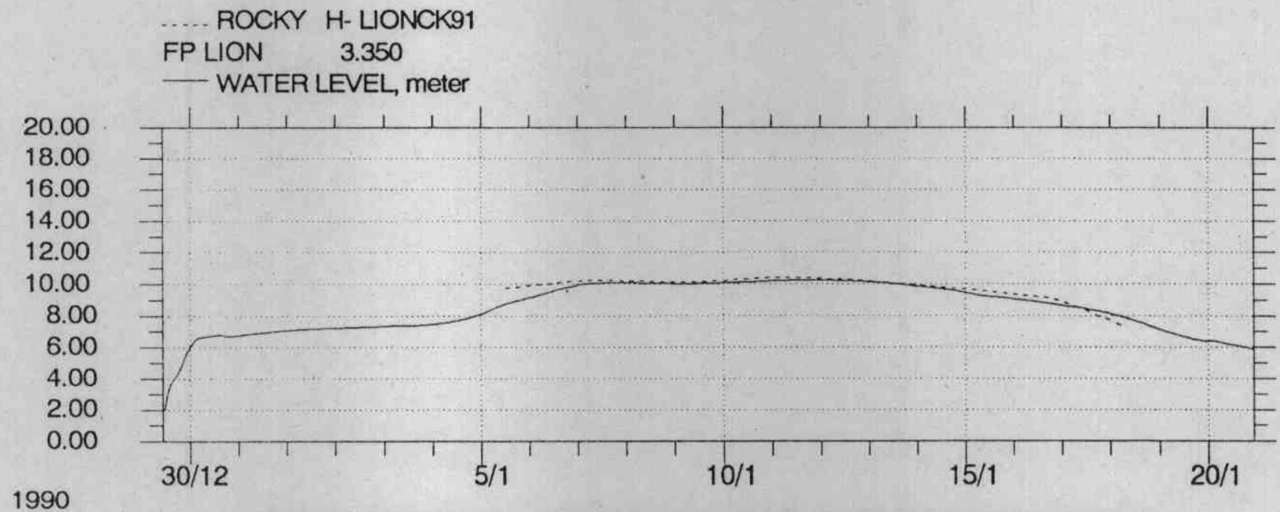
FIGURE 2-3



FITZROY RIVER - Rockhampton Flood Gauge

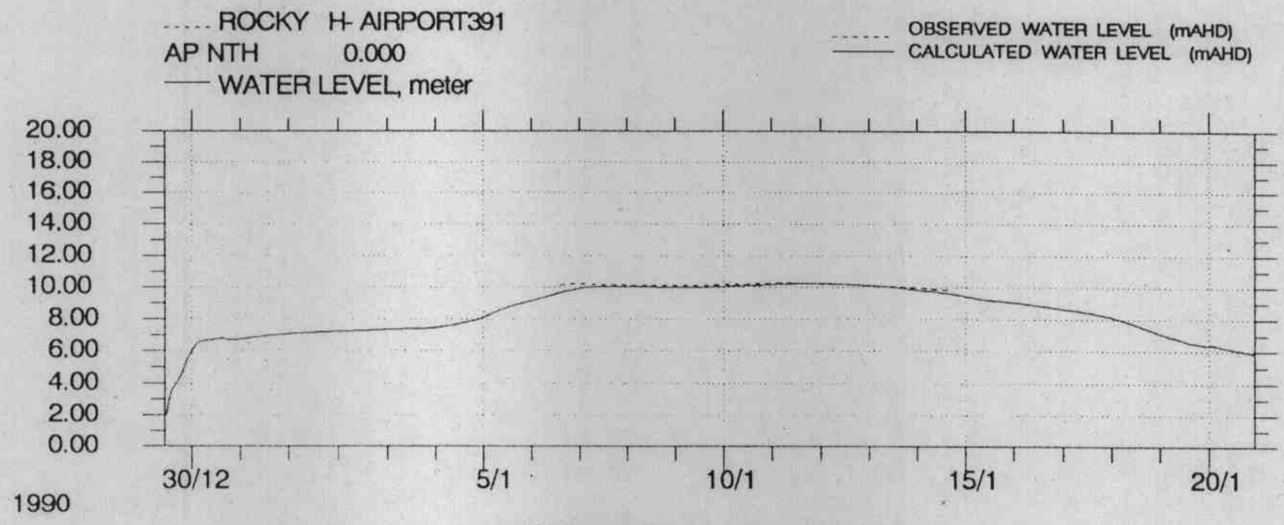


YEPPEN 1 BRIDGE (U / S) - Bruce Highway

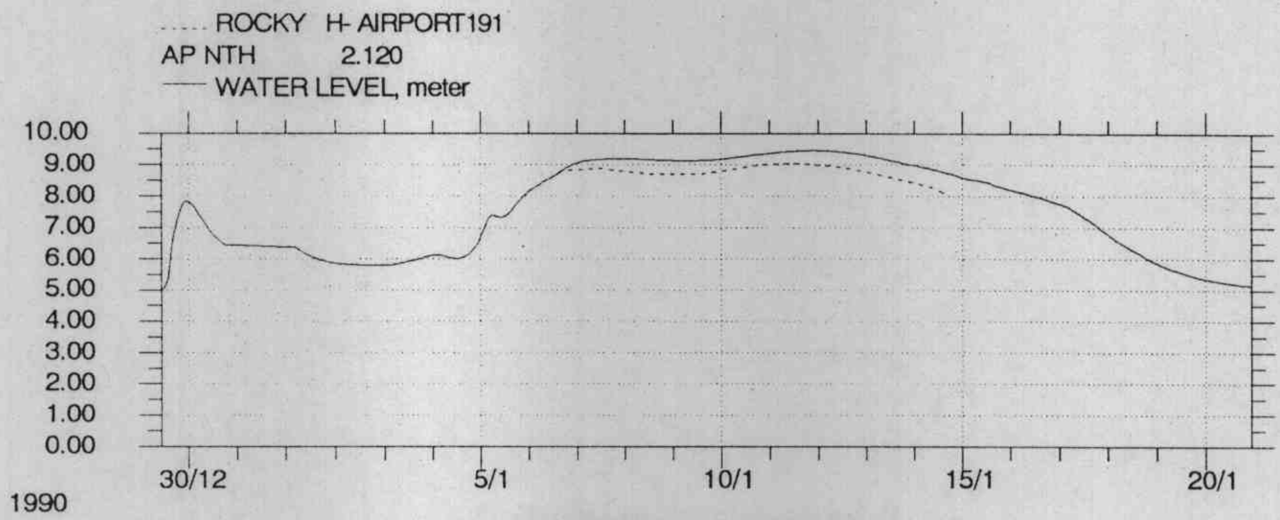


LION CREEK - Rockhampton - Ridgelands Road

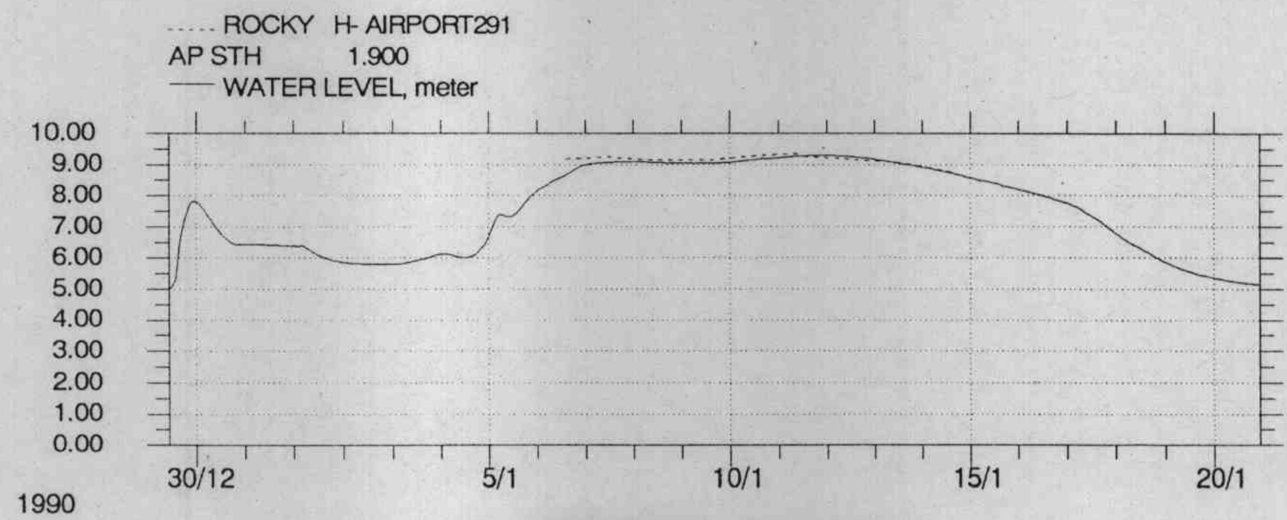
CMPS&F CAMP SCOTT FURPHY PTY. LTD.	**ROCKHAMPTON FLOOD MANAGEMENT STUDY** **1991 FLOOD CALIBRATION** **FLOOD LEVEL HYDROGRAPHS AT CITY FLOOD GAUGE, YEPPEN 1 BRIDGE & LION CREEK**	**FIGURE 2-5**



AIRPORT - Gauge 1



AIRPORT - Gauge 2

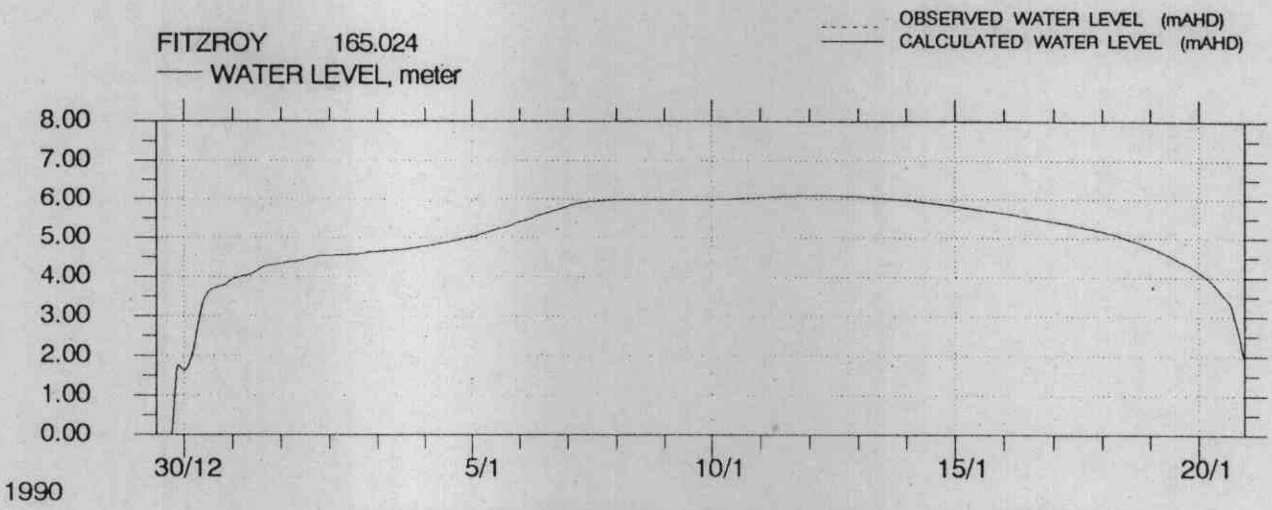


AIRPORT - Gauge 3

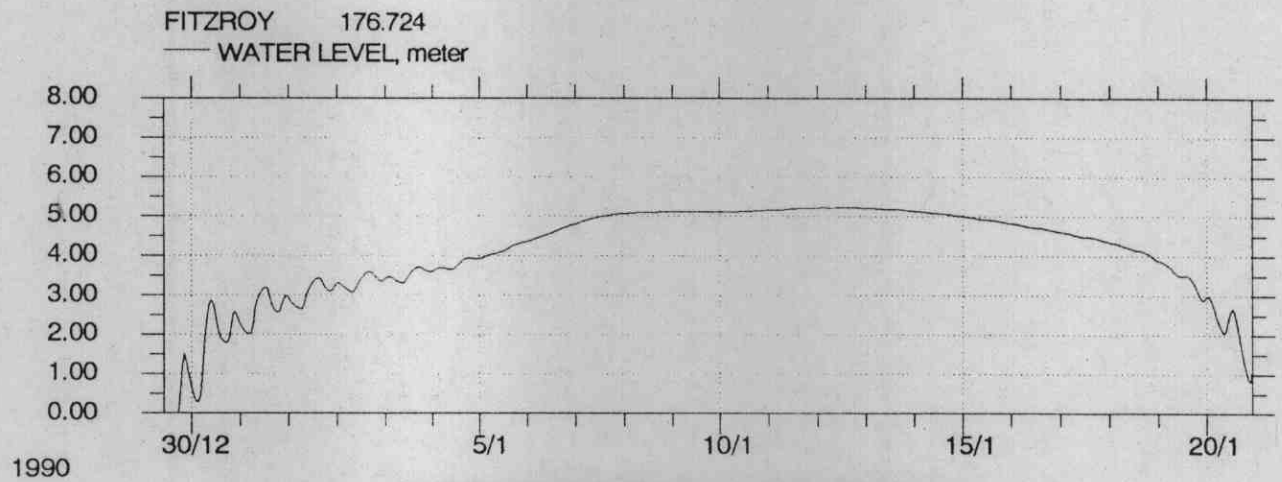
CMPS&F
CAMP SCOTT FURPHY PTY. LTD.

ROCKHAMPTON FLOOD MANAGEMENT STUDY
1991 FLOOD CALIBRATION
FLOOD LEVEL HYDROGRAPHS AT AIRPORT
GAUGES 1, 2 & 3

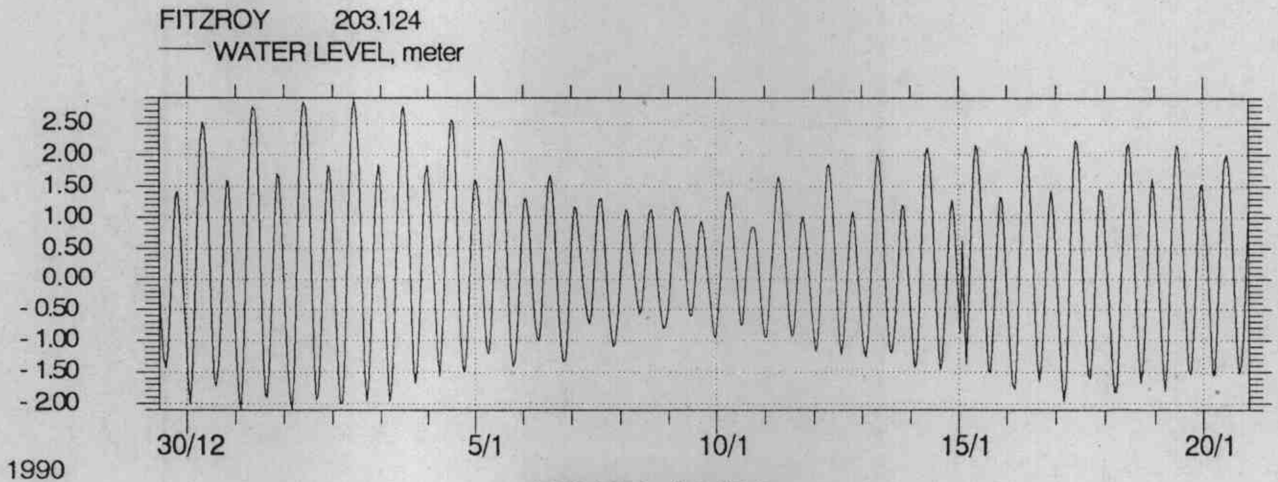
FIGURE 2-6



FITZROY RIVER - Gavial Creek Junction



FITZROY RIVER - Pirate Point

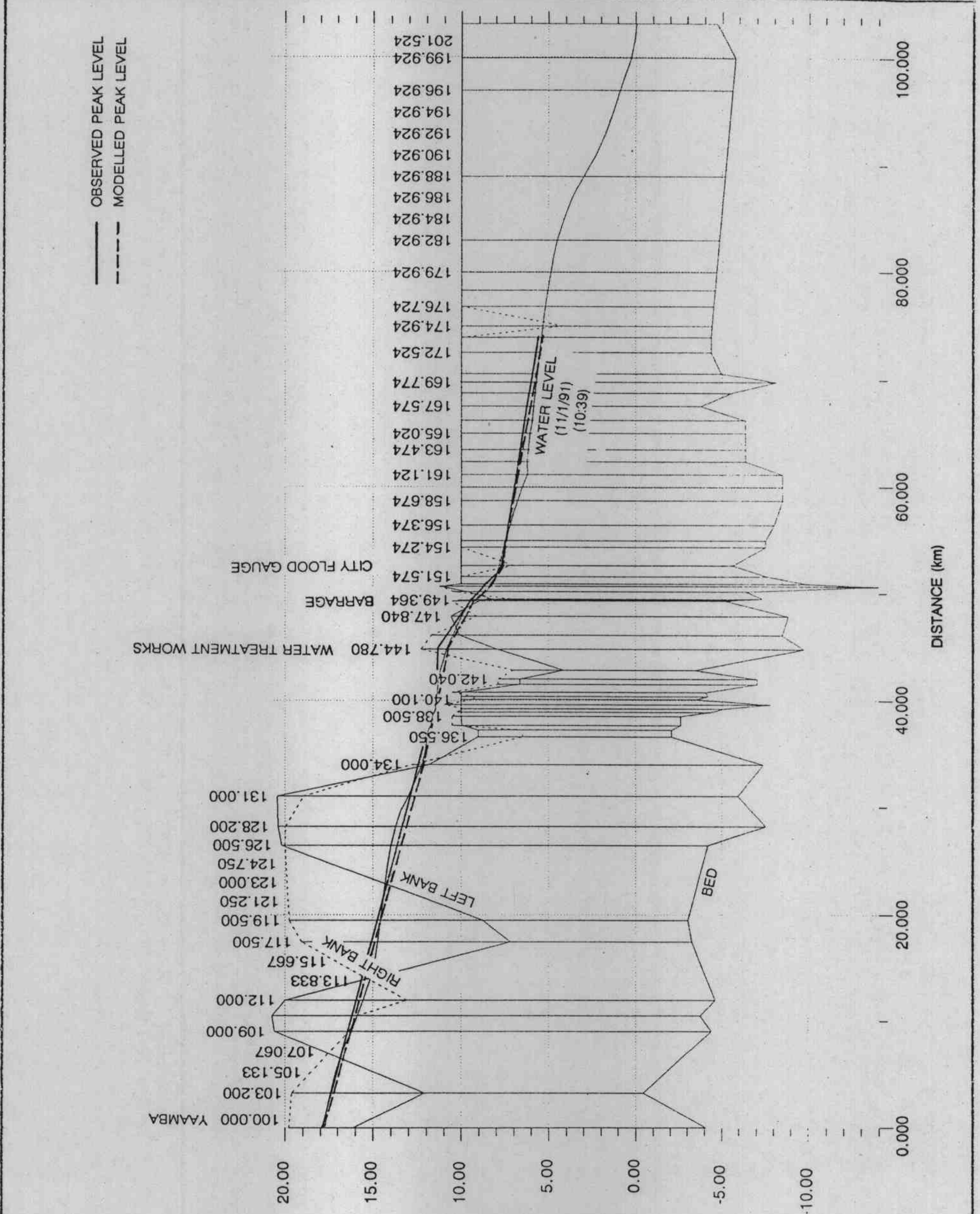


PORT ALMA

CMPS&F
CAMP SCOTT FURPHY PTY. LTD.

ROCKHAMPTON FLOOD MANAGEMENT STUDY
1991 FLOOD CALIBRATION
FLOOD LEVEL HYDROGRAPHS AT GAVIAL CK JUNCTION, PIRATE POINT & PORT ALMA

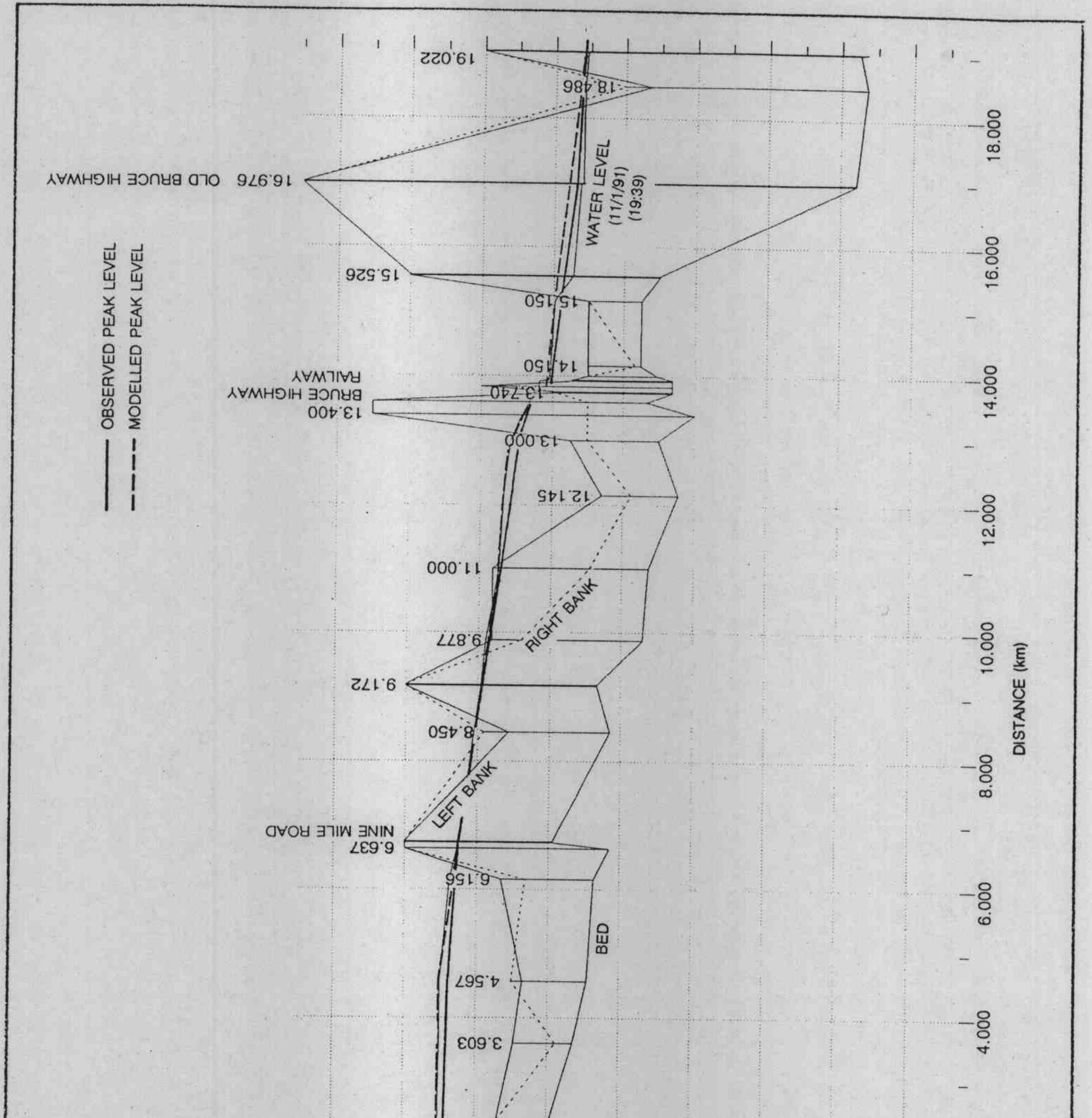
FIGURE 2-7



CAMP SCOTT FURPHY PTY. LTD.

ROCKHAMPTON FLOOD MANAGEMENT STUDY
1991 FLOOD CALIBRATION
LONGITUDINAL PROFILE OF FITZROY RIVER

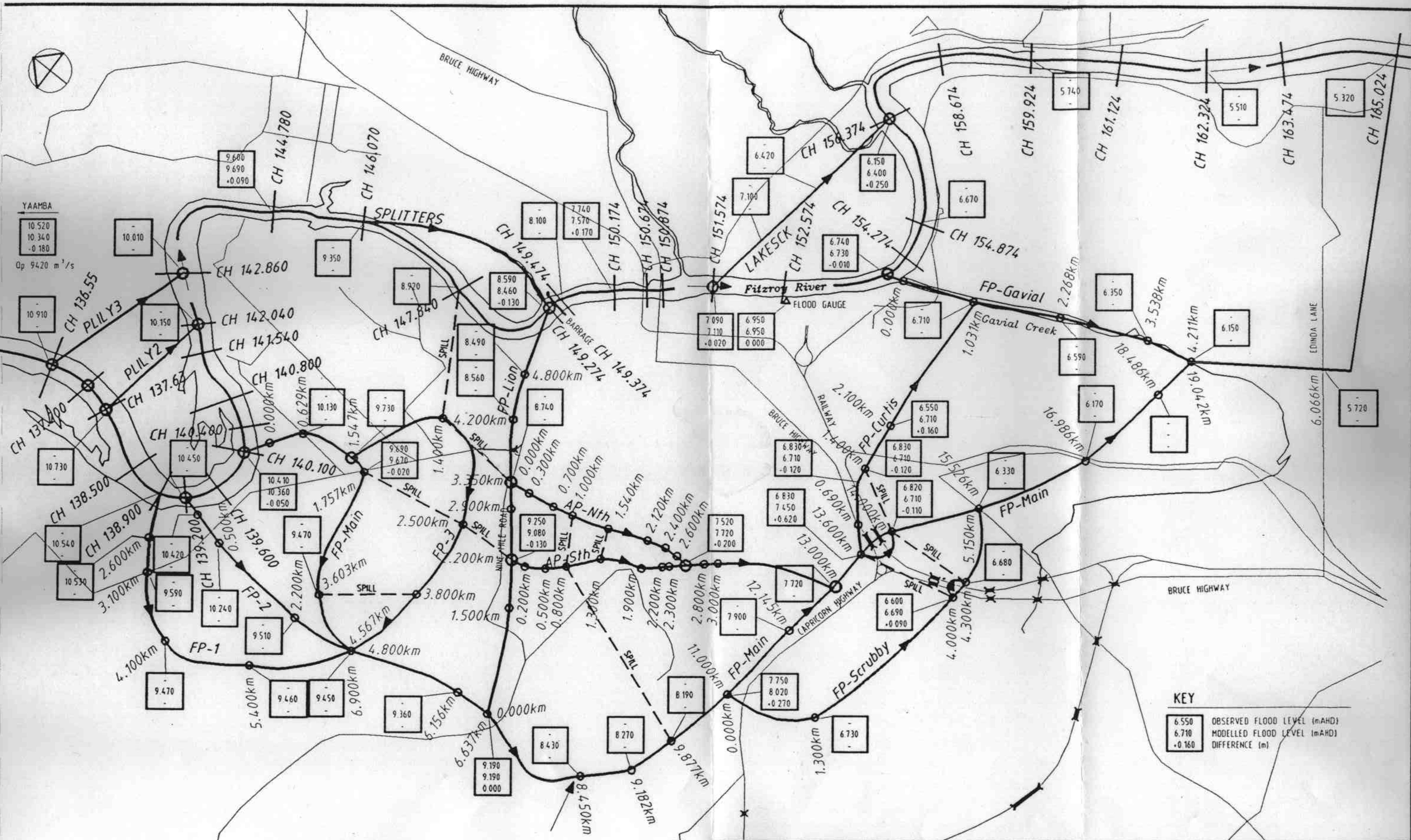
FIGURE 2-8



CAMP SCOTT FURPHY PTY. LTD.

ROCKHAMPTON FLOOD MANAGEMENT STUDY
1991 FLOOD CALIBRATION
LONGITUDINAL PROFILE OF MAIN FLOODWAY
(FP–MAIN)

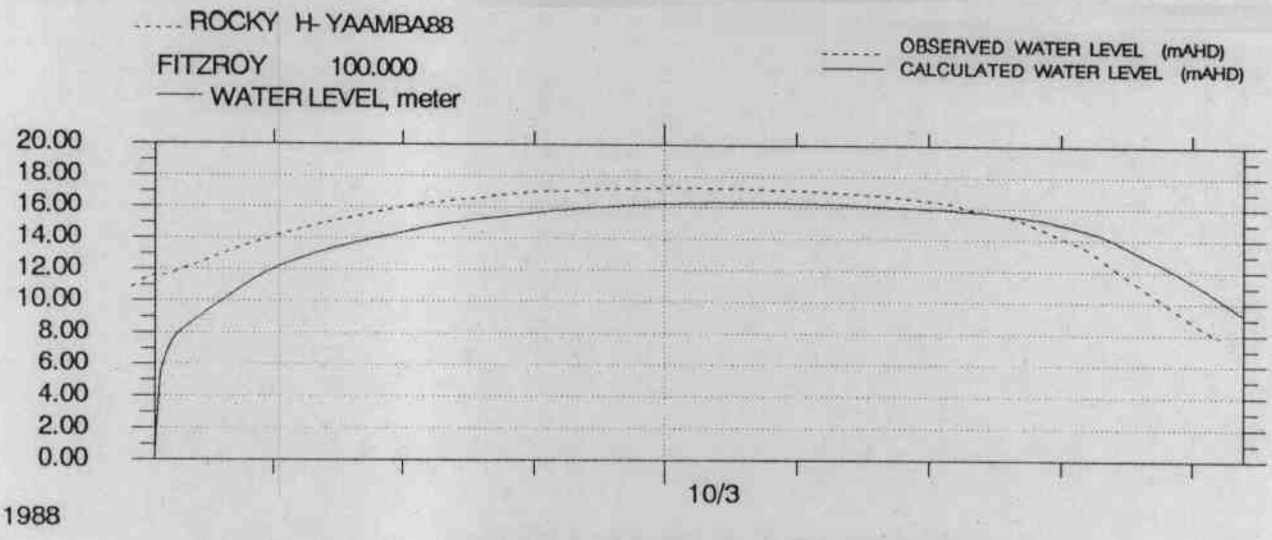
FIGURE 2–9



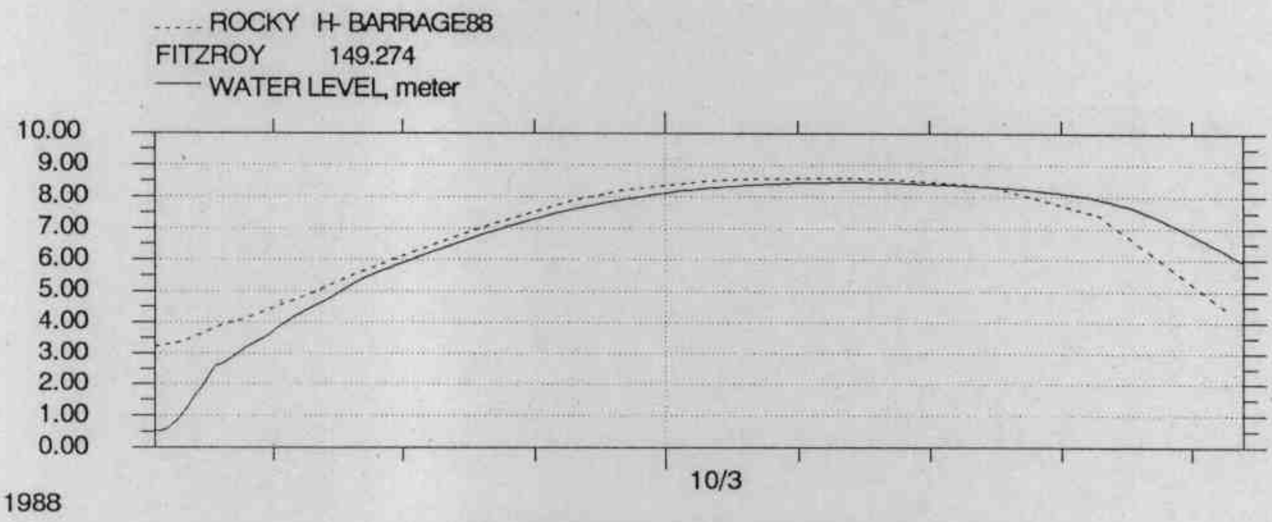
CMPS&F
CAMP SCOTT FURPHY PTY. LTD.

ROCKHAMPTON FLOOD MANAGEMENT STUDY
HYDRAULIC MODEL LAYOUT 2
CALIBRATION 1988 FLOOD

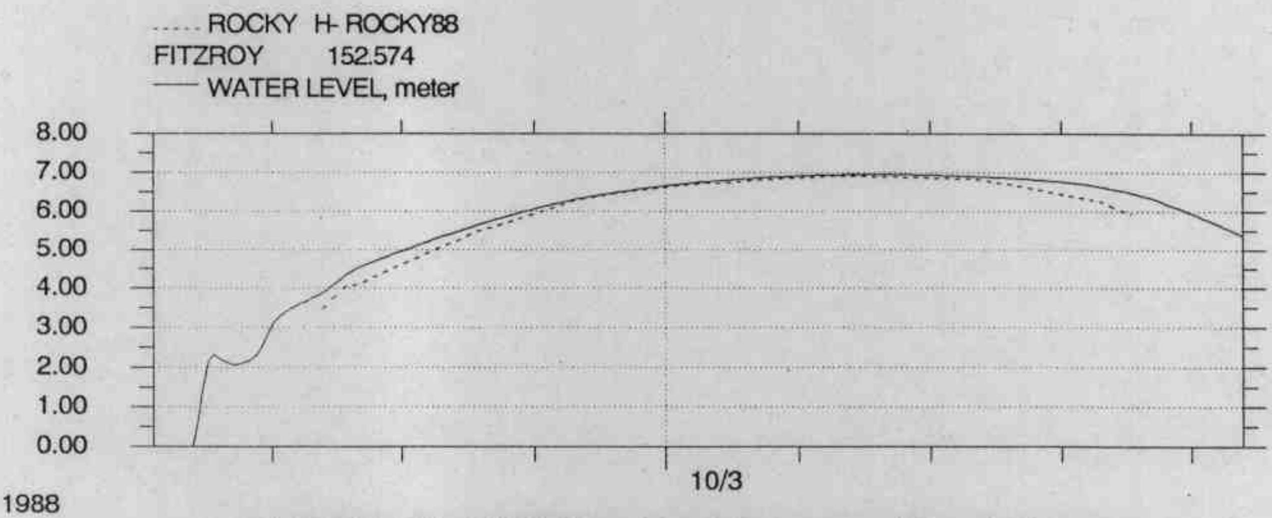
FIGURE 2–10



FITZROY RIVER - Yaamba

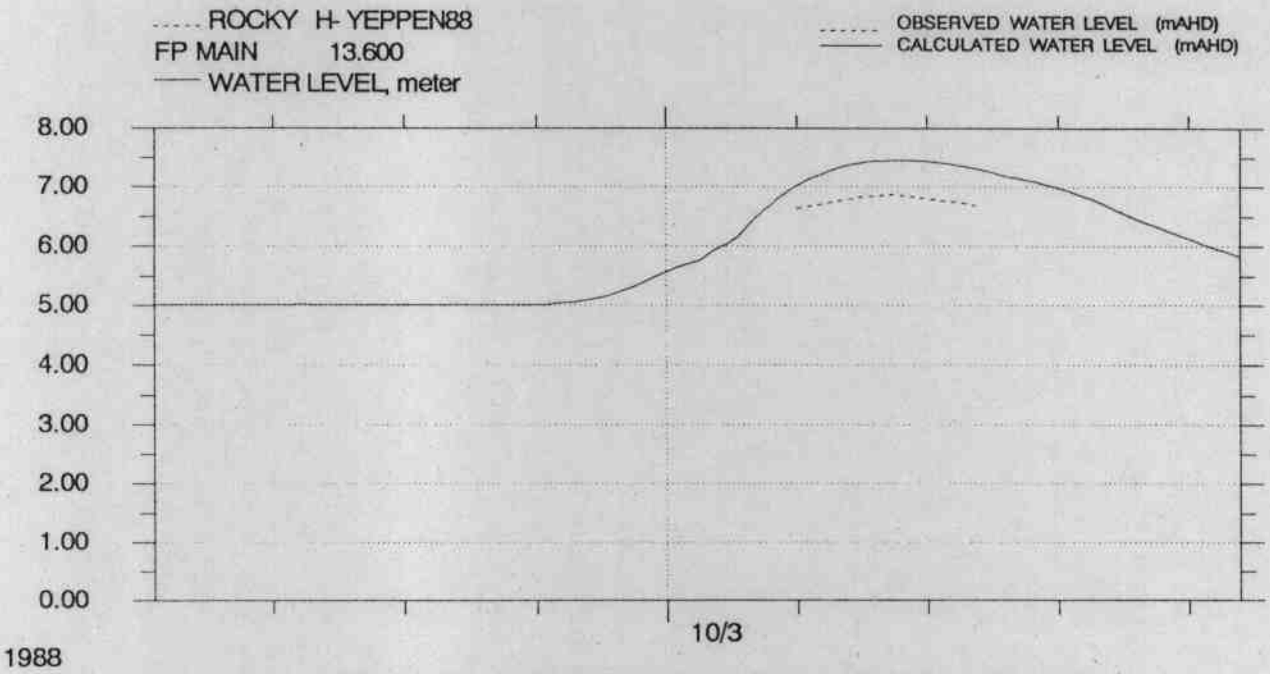


FITZROY RIVER - Barrage

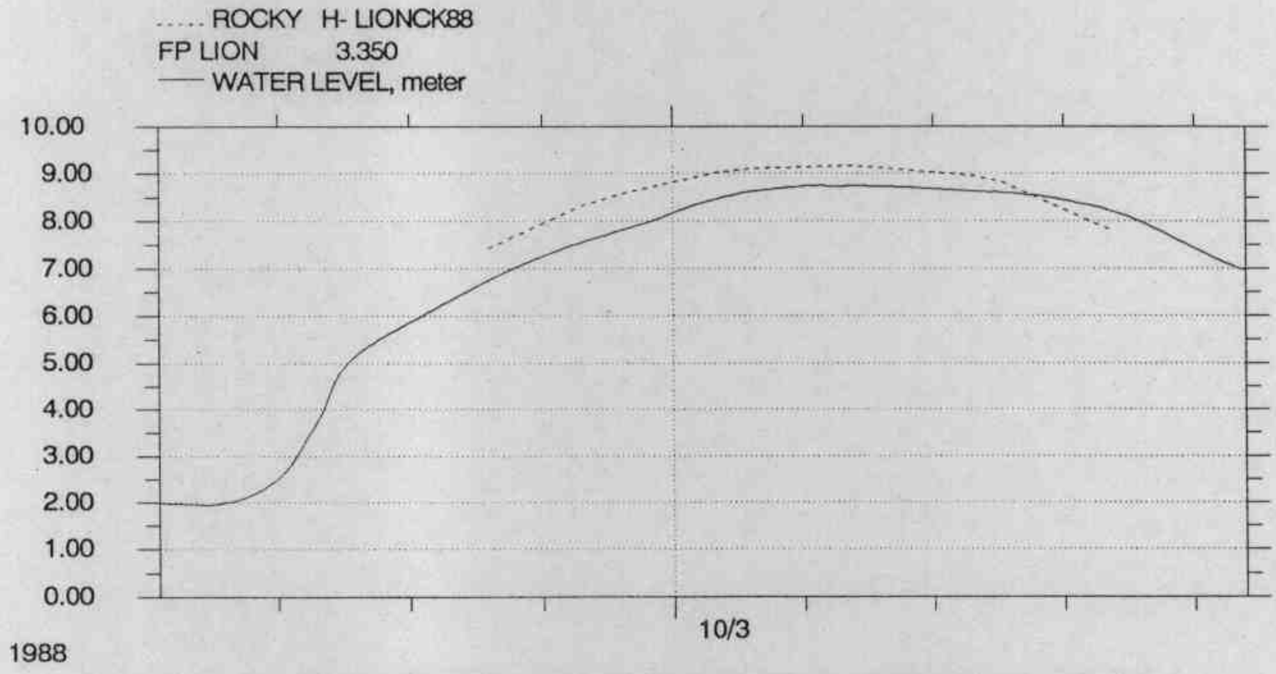


FITZROY RIVER - Rockhampton Flood Gauge

CMPS&F CAMP SCOTT FURPHY PTY. LTD.	**ROCKHAMPTON FLOOD MANAGEMENT STUDY** **1988 FLOOD CALIBRATION** **FLOOD LEVEL HYDROGRAPHS AT YAAMBA, BARRAGE & CITY FLOOD GAUGE**	**FIGURE 2-11**

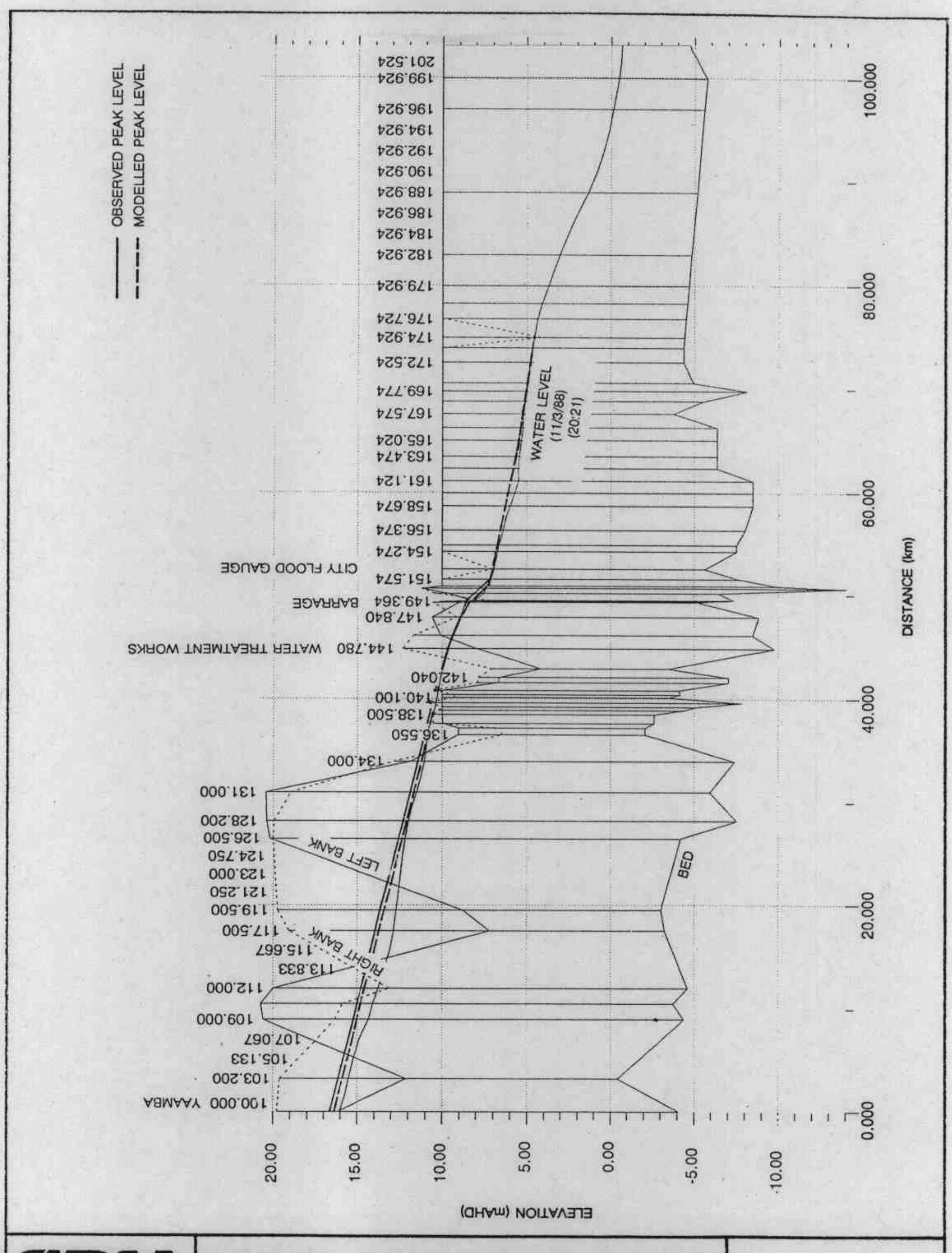


YEPPEN 1 BRIDGE (U / S) - Bruce Highway



LION CREEK - Rockhampton - Ridgelands Road

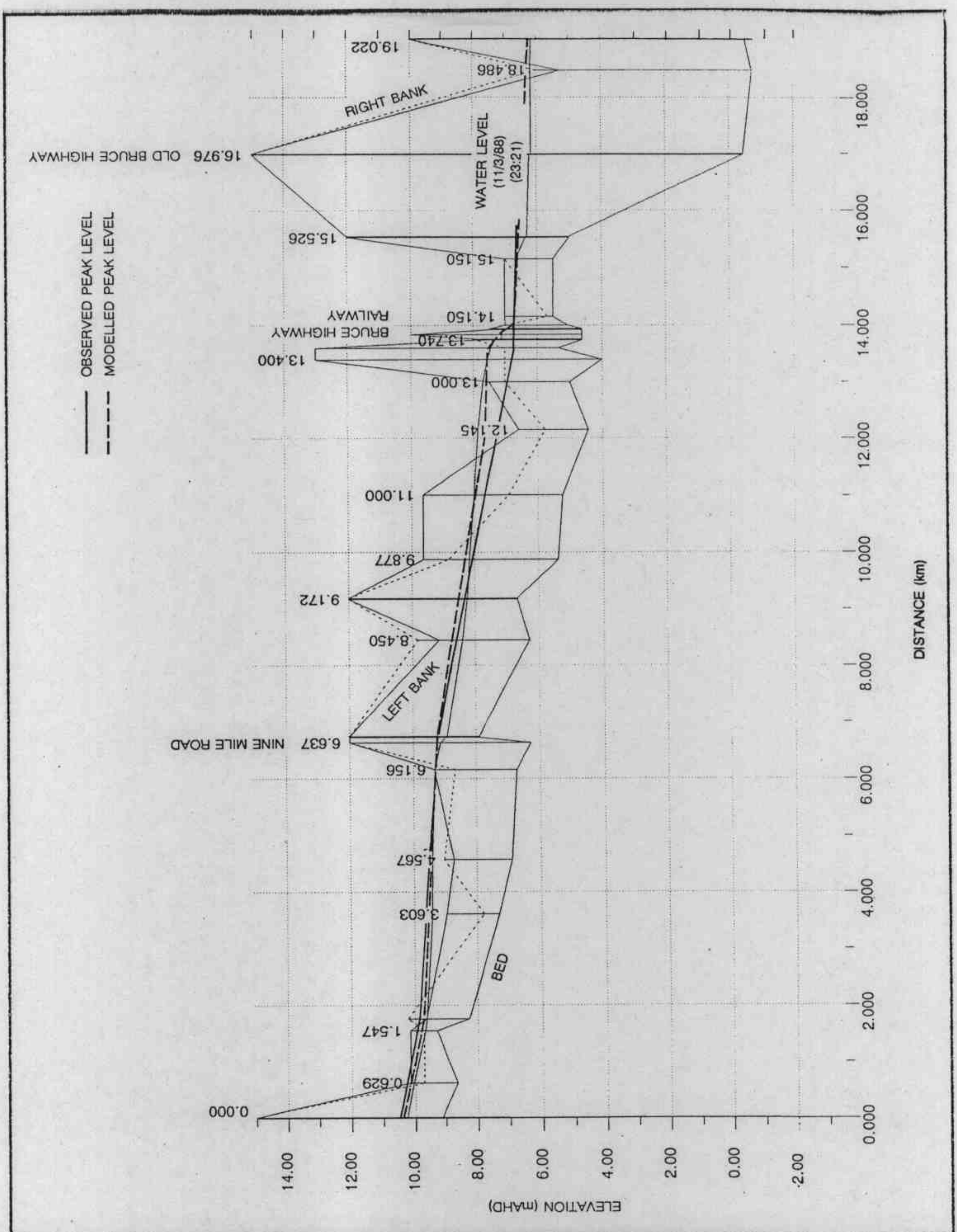
CMPS&F CAMP SCOTT FURPHY PTY. LTD.	**ROCKHAMPTON FLOOD MANAGEMENT STUDY** **1988 FLOOD CALIBRATION** **FLOOD LEVEL HYDROGRAPHS AT YEPPEN & LION CREEK**	**FIGURE 2–12**



CAMP SCOTT FURPHY PTY. LTD.

ROCKHAMPTON FLOOD MANAGEMENT STUDY
1988 FLOOD CALIBRATION
LONGITUDINAL PROFILE OF FITZROY RIVER

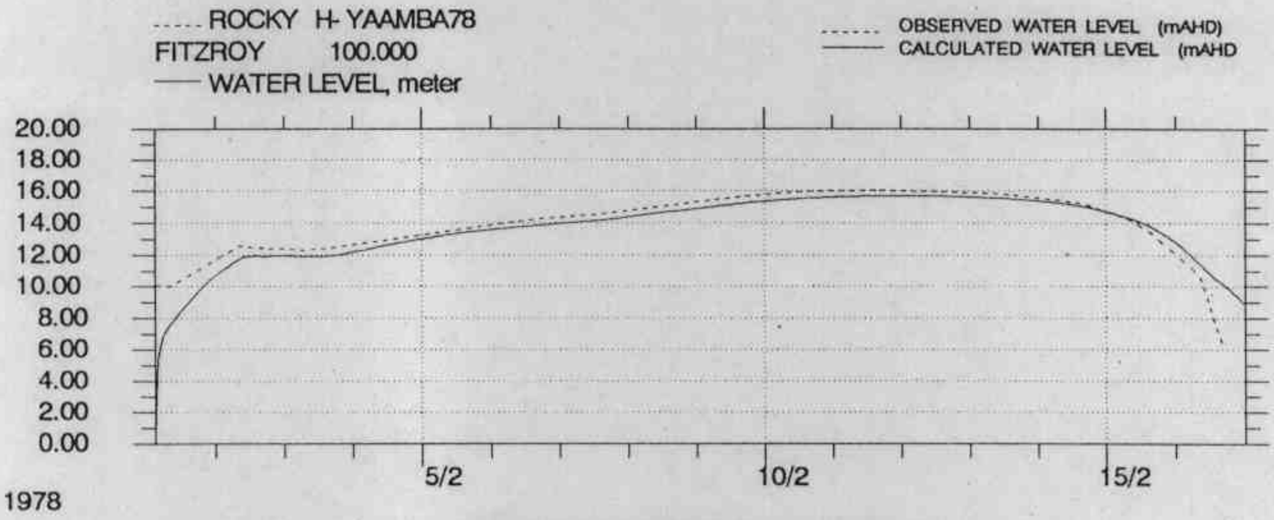
FIGURE 2-13



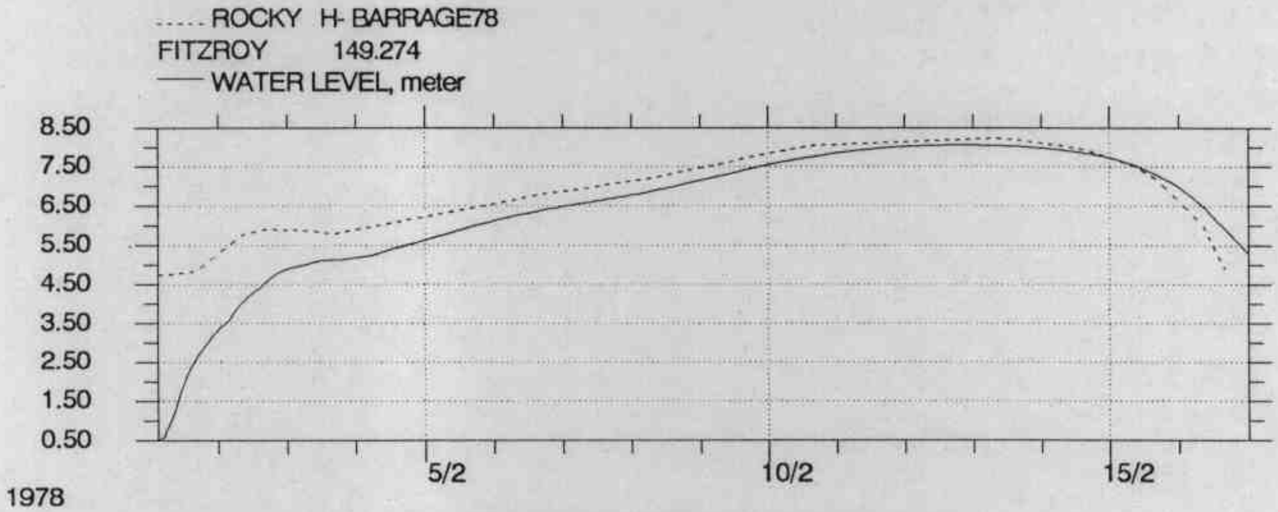
CAMP SCOTT FURPHY PTY. LTD.

ROCKHAMPTON FLOOD MANAGEMENT STUDY
1988 FLOOD CALIBRATION
LONGITUDINAL PROFILE OF MAIN FLOODWAY
(FP–MAIN)

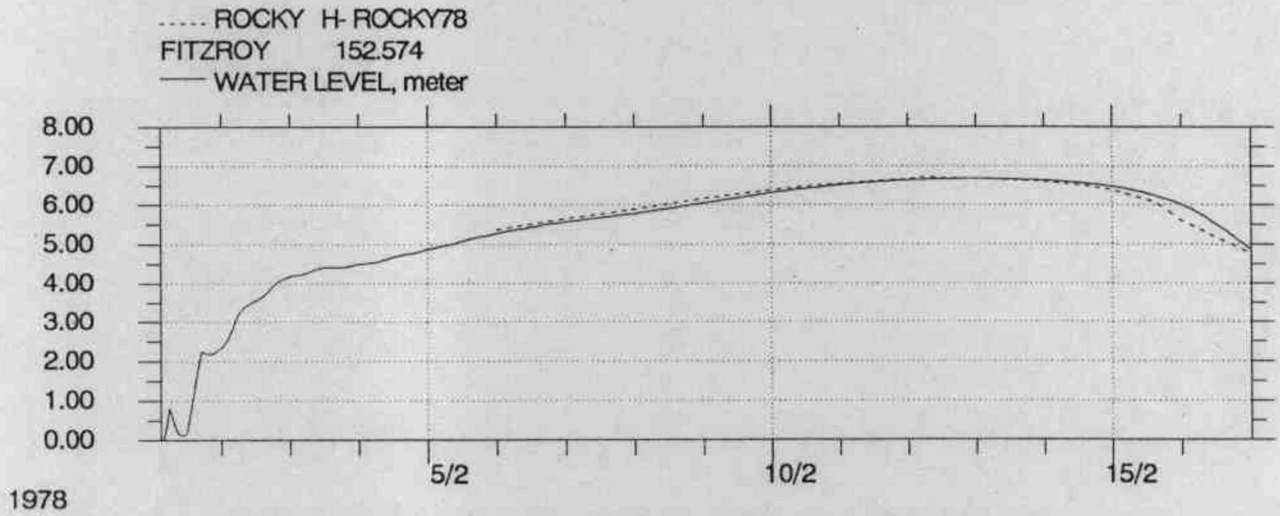
FIGURE 2–14



FITZROY RIVER - Yaamba

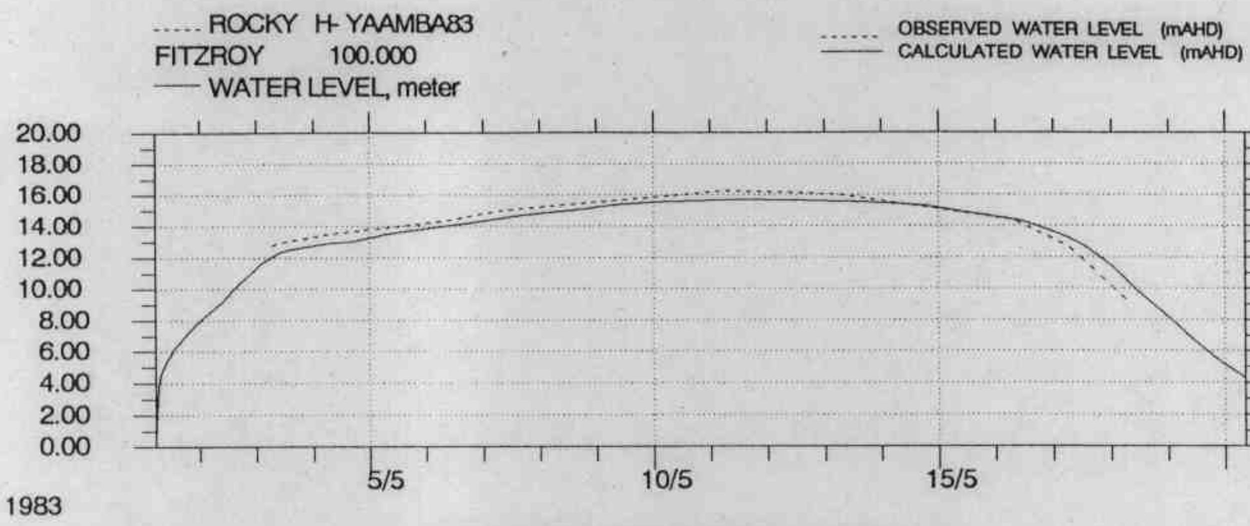


FITZROY RIVER - Barrage

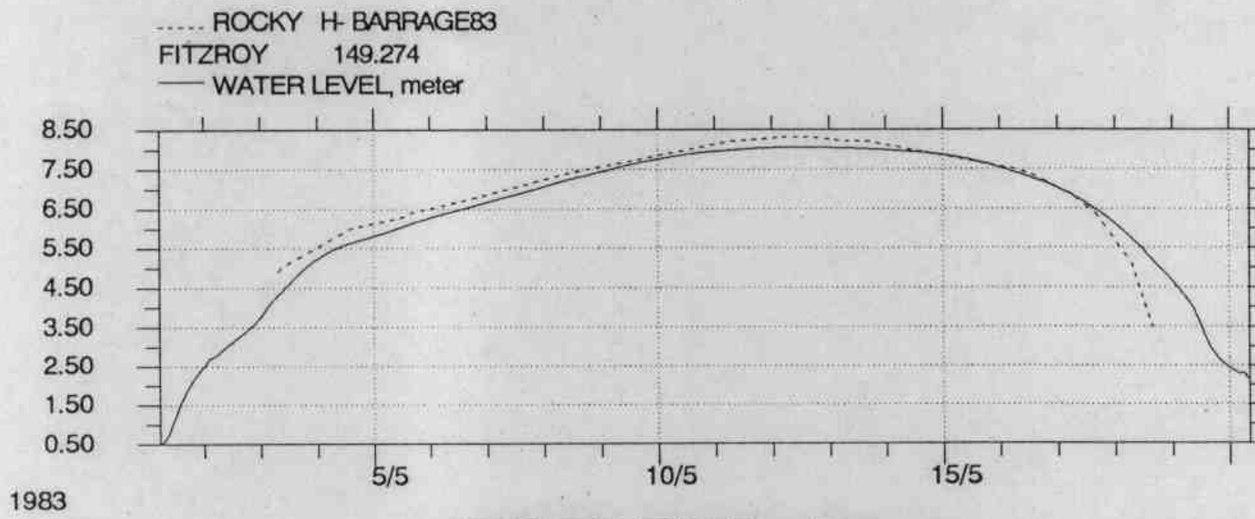


FITZROY RIVER - Rockhampton Flood Gauge

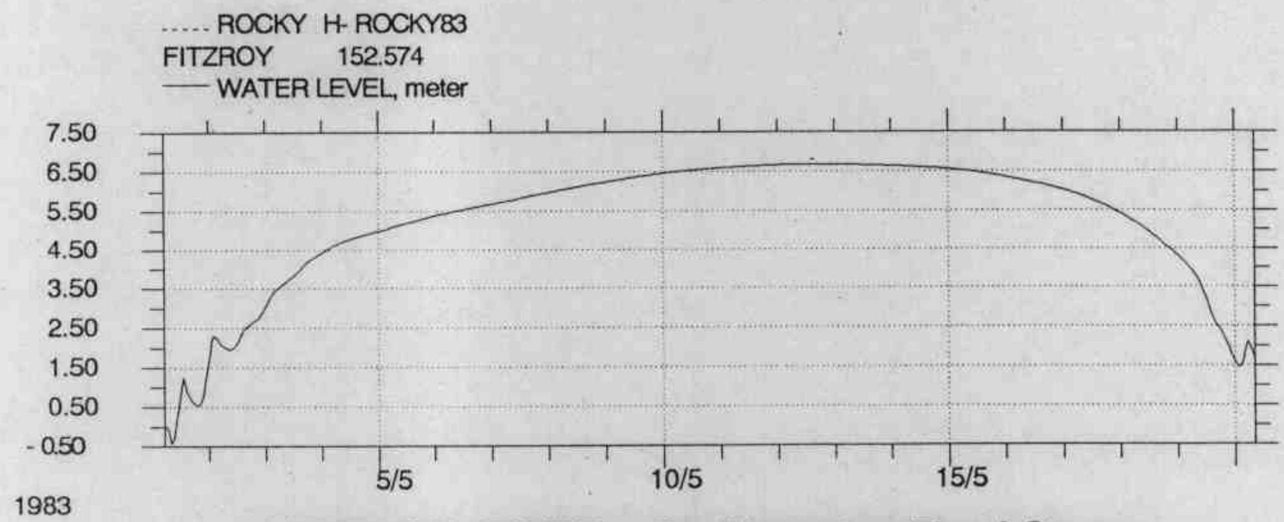
CMPS&F CAMP SCOTT FURPHY PTY. LTD.	**ROCKHAMPTON FLOOD MANAGEMENT STUDY** **1978 FLOOD VALIDATION** **FLOOD LEVEL HYDROGRAPHS AT YAAMBA, BARRAGE & CITY FLOOD GAUGE**	**FIGURE 2–15**



FITZROY RIVER - Yaamba

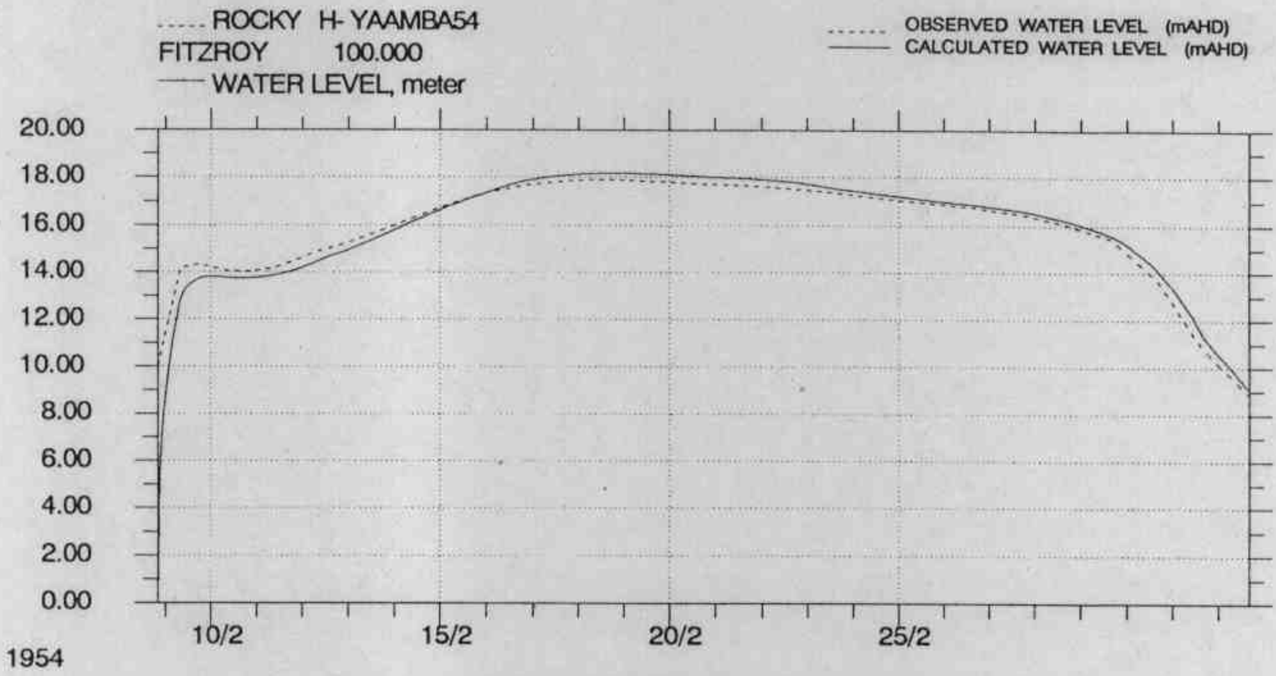


FITZROY RIVER - Barrage

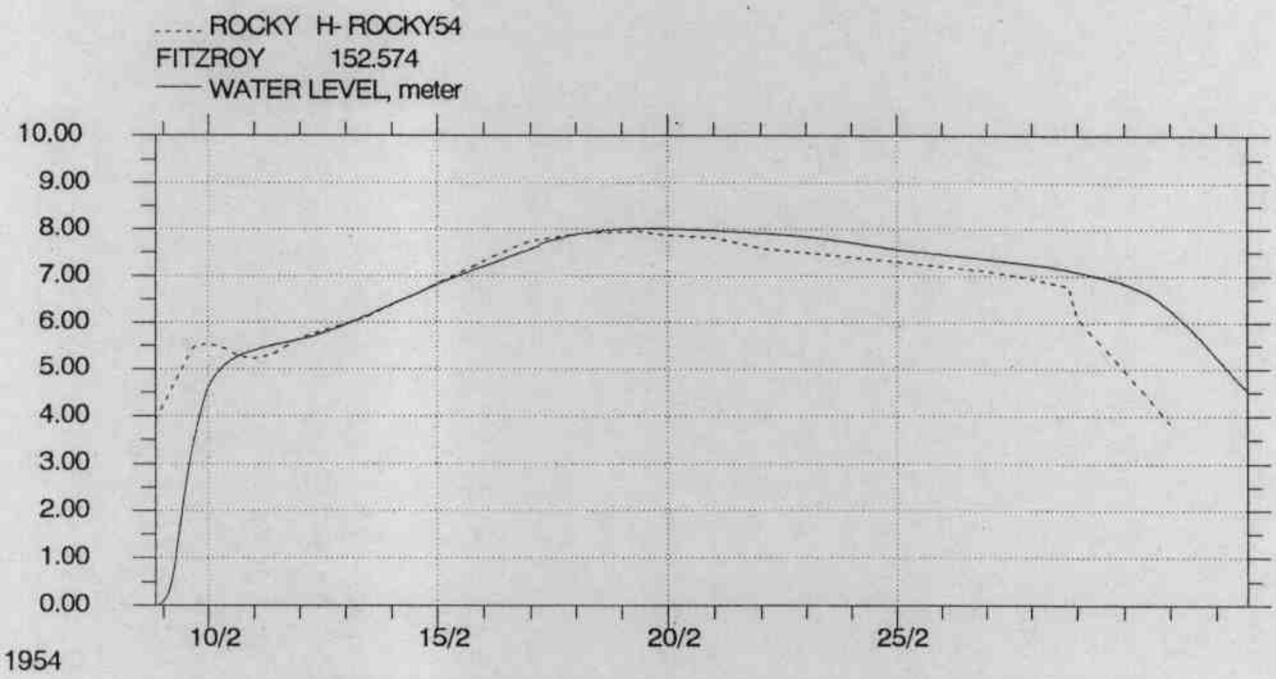


FITZROY RIVER - Rockhampton Flood Gauge

CMPS&F CAMP SCOTT FURPHY PTY. LTD.	ROCKHAMPTON FLOOD MANAGEMENT STUDY 1983 FLOOD VALIDATION FLOOD LEVEL HYDROGRAPHS AT YAAMBA, BARRAGE & CITY FLOOD GAUGE	FIGURE 2-16



FITZROY RIVER - Yaamba

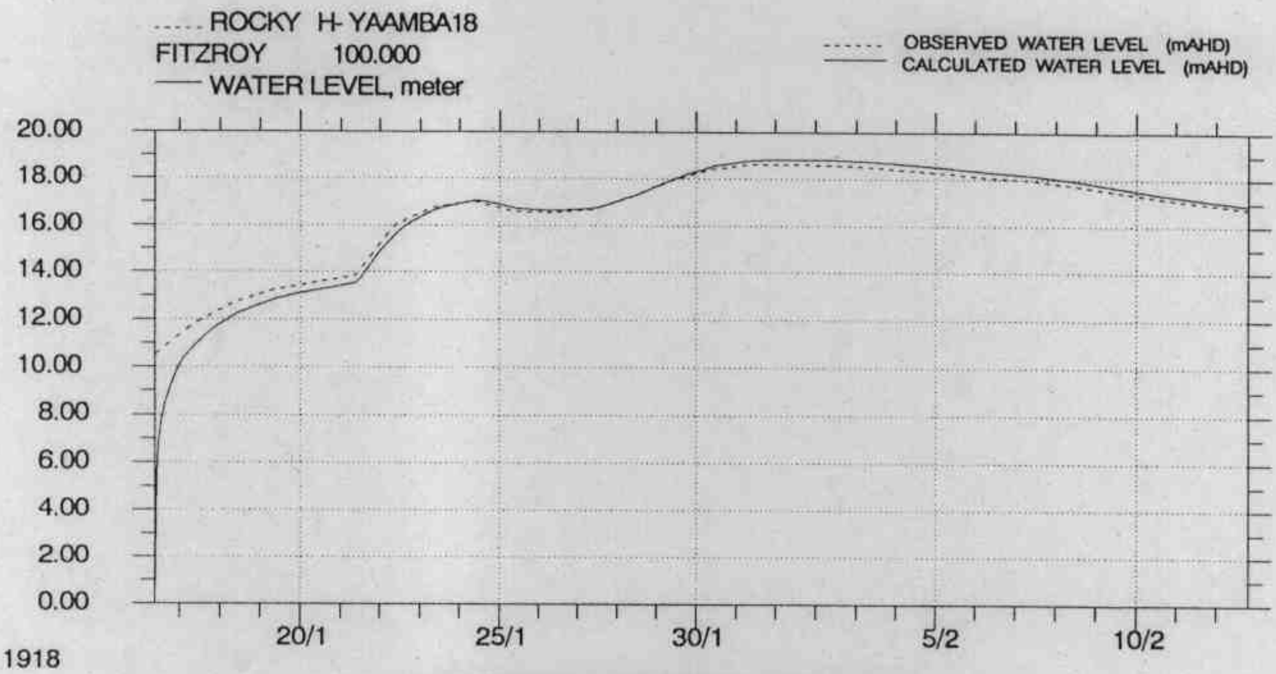


FITZROY RIVER - Rockhampton Flood Gauge

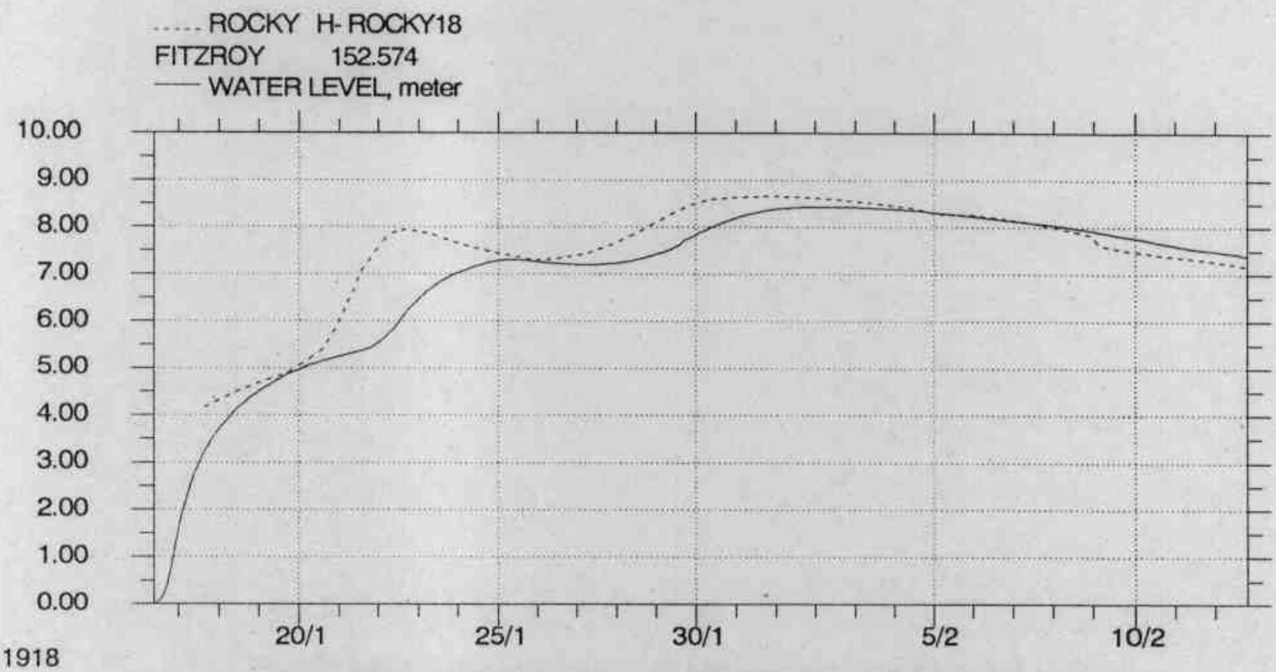
CMPS&F
CAMP SCOTT FURPHY PTY. LTD.

ROCKHAMPTON FLOOD MANAGEMENT STUDY
1954 FLOOD VALIDATION
FLOOD LEVEL HYDROGRAPHS AT YAAMBA
& CITY FLOOD GAUGE

FIGURE 2-17

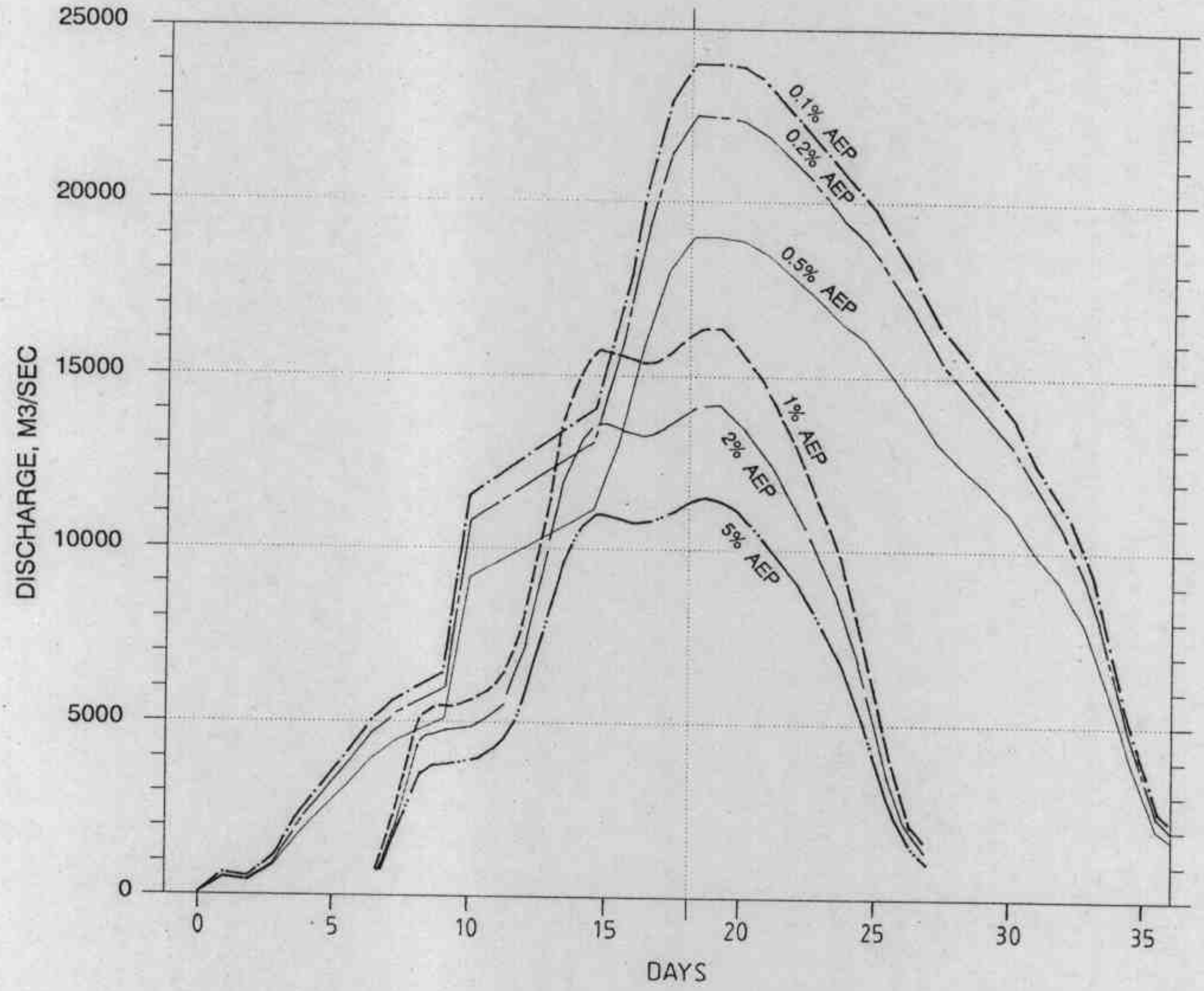


FITZROY RIVER - Yaamba



FITZROY RIVER - Rockhampton Flood Gauge

CMPS&F CAMP SCOTT FURPHY PTY. LTD.	**ROCKHAMPTON FLOOD MANAGEMENT STUDY** **1918 FLOOD VALIDATION** **FLOOD LEVEL HYDROGRAPHS AT YAAMBA & CITY FLOOD GAUGE**	**FIGURE 2-18**



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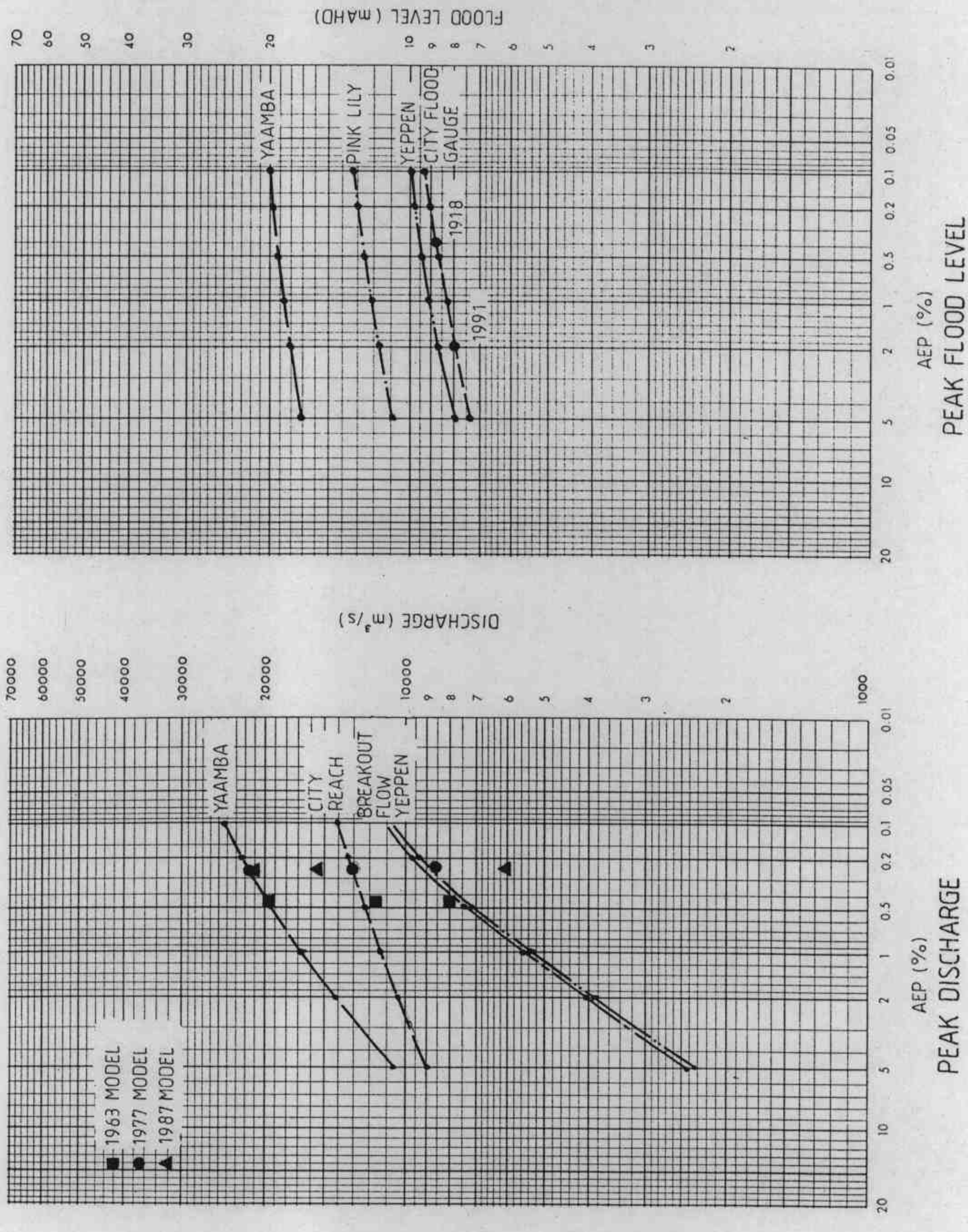
CAMP SCOTT FURPHY PTY. LTD.

ROCKHAMPTON FLOOD MANAGEMENT STUDY
DESIGN FLOOD DISCHARGE HYDROGRAPHS

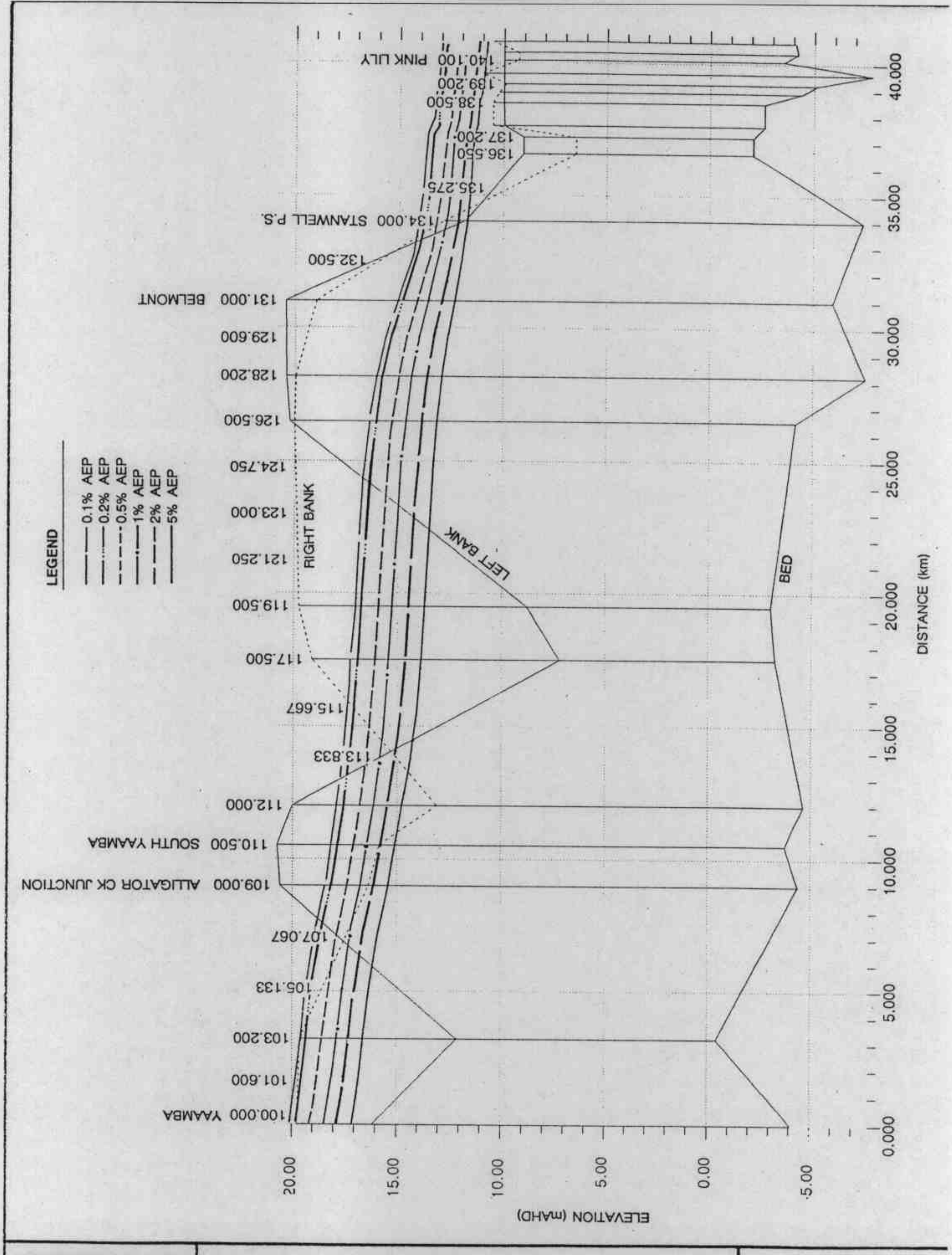
FIGURE 2–19



CAMP SCOTT FURPHY PTY. LTD.

ROCKHAMPTON FLOOD MANAGEMENT STUDY
DISCHARGE AND FLOOD LEVEL PROBABILITY CURVES

FIGURE 2-20



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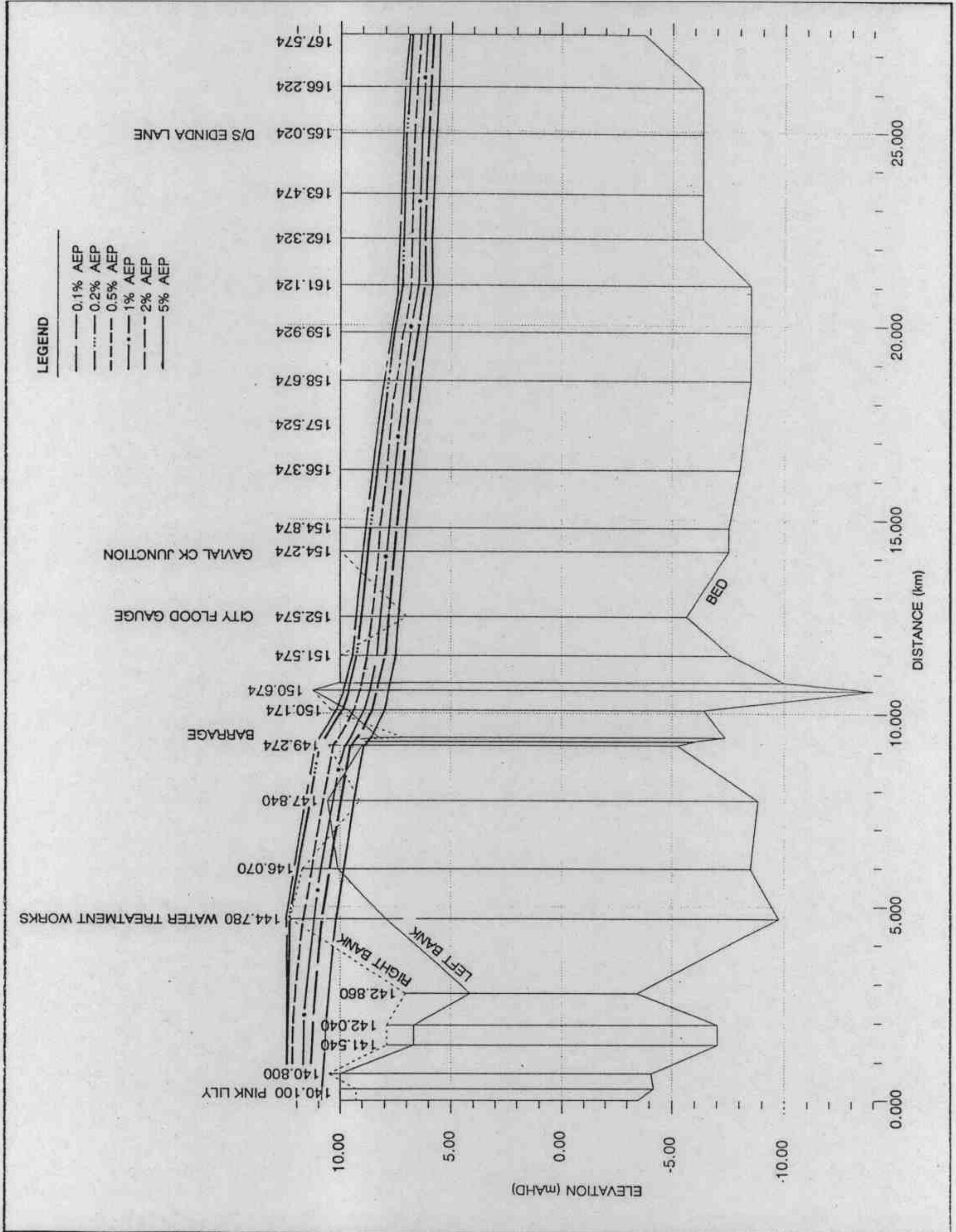
CAMP SCOTT FURPHY PTY. LTD.

ROCKHAMPTON FLOOD MANAGEMENT STUDY
LONGITUDINAL PROFILE FOR DESIGN FLOODS
FITZROY RIVER - YAAMBA TO PINK LILY

FIGURE 2-21



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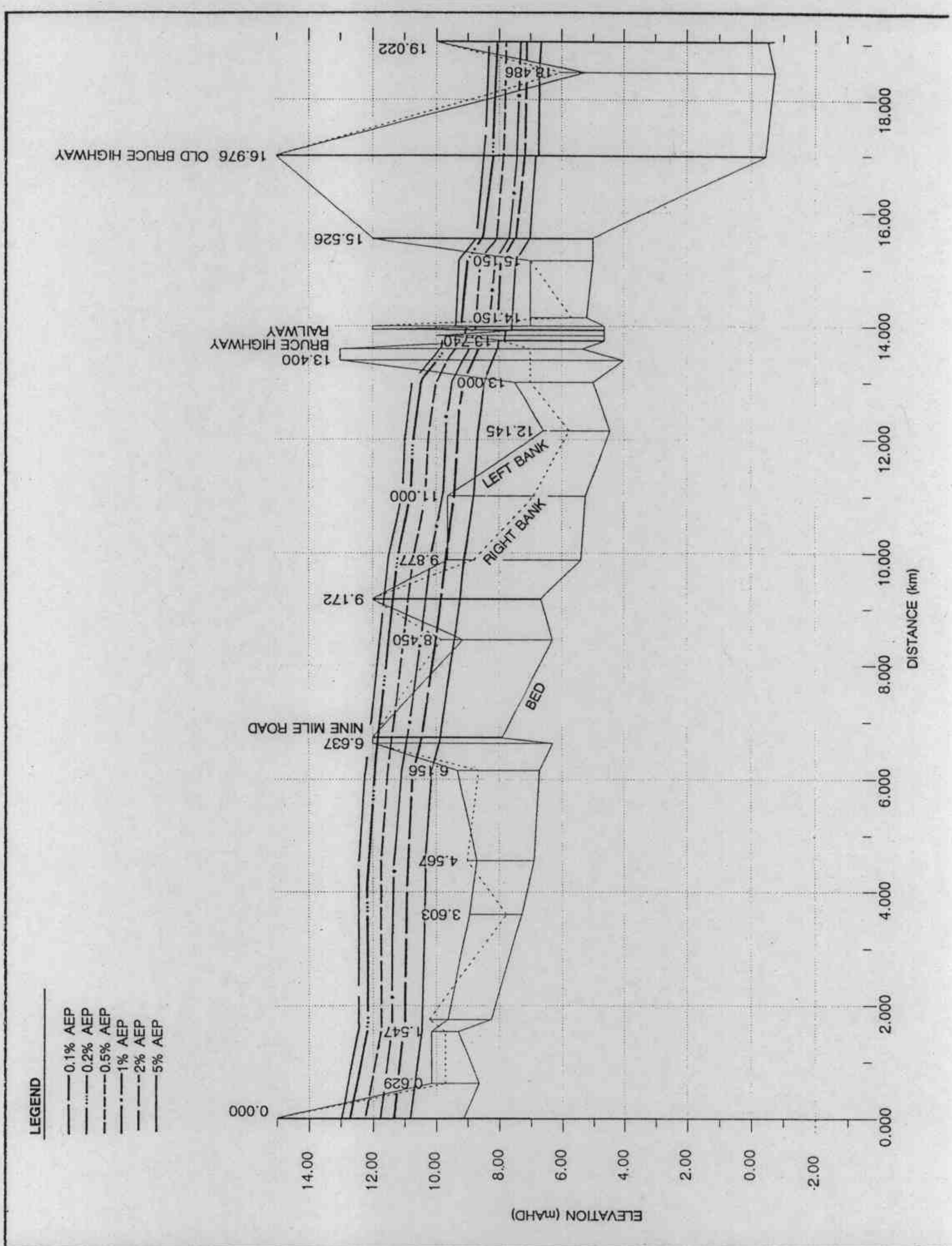
CAMP SCOTT FURPHY PTY. LTD.

ROCKHAMPTON FLOOD MANAGEMENT STUDY
LONGITUDINAL PROFILE FOR DESIGN FLOODS
FITZROY RIVER - PINK LILY TO EDINDA LANE

FIGURE 2-22



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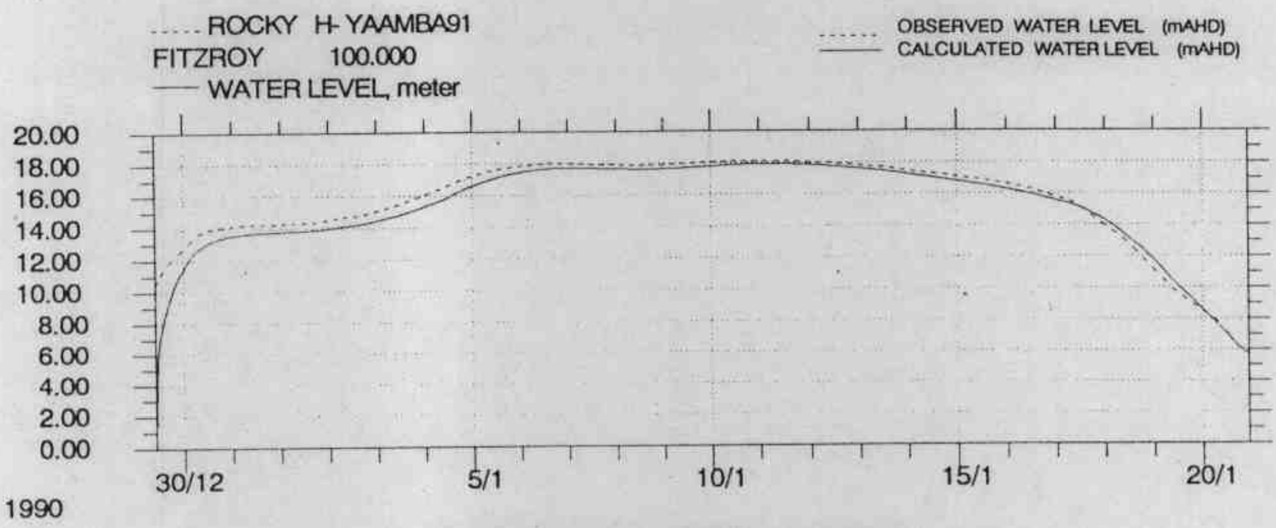
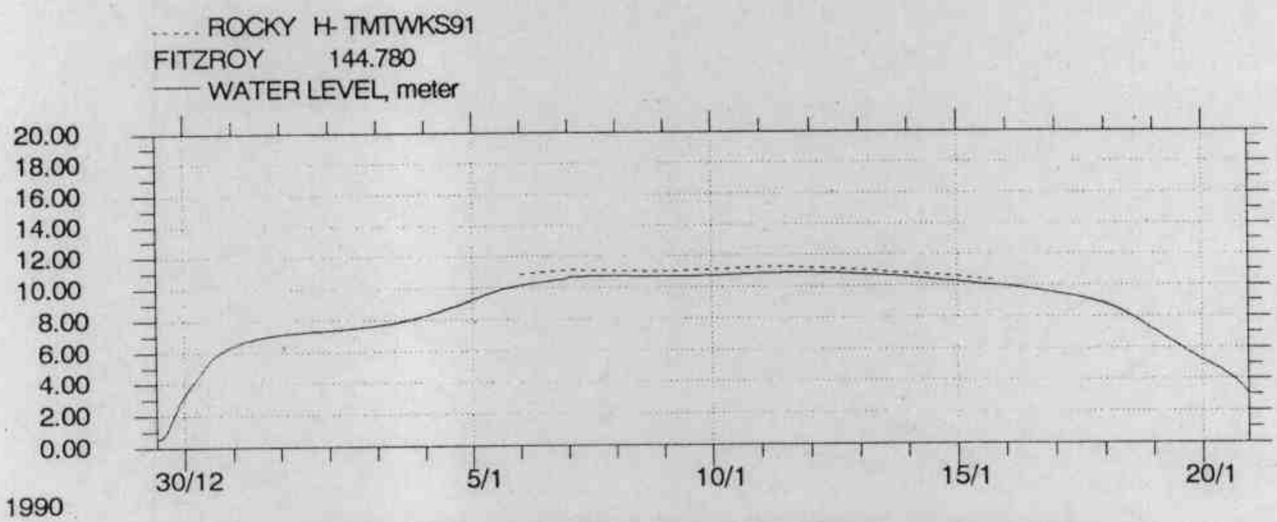
CAMP SCOTT FURPHY PTY. LTD.

ROCKHAMPTON FLOOD MANAGEMENT STUDY
LONGITUDINAL PROFILE FOR DESIGN FLOODS
MAIN FLOODWAY

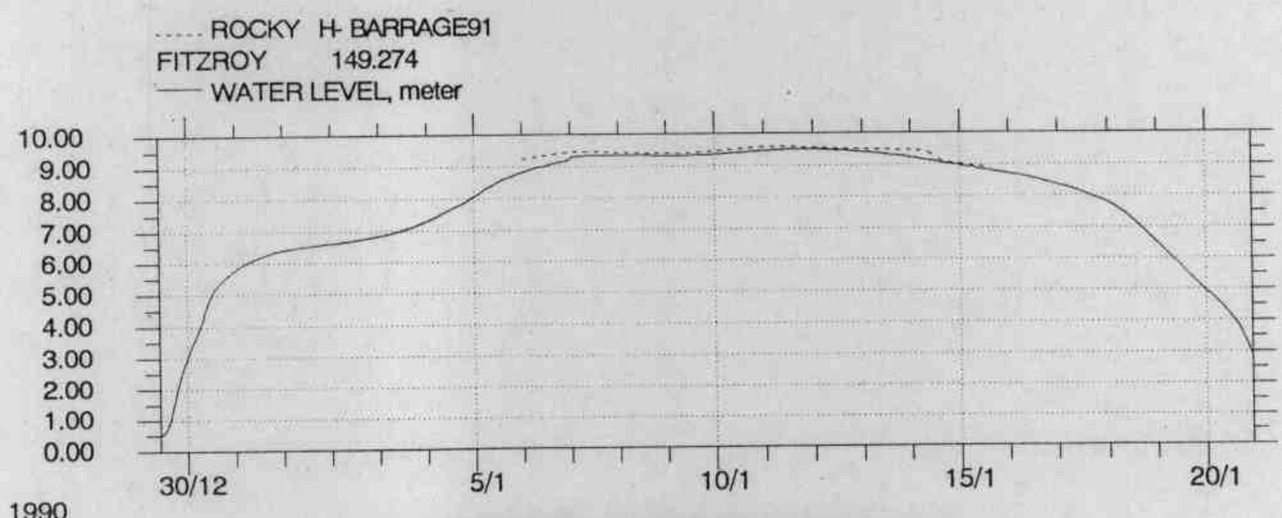
FIGURE 2–23



FITZROY RIVER - Yaamba



FITZROY RIVER - Water Treatment Works



FITZROY RIVER - Barrage

CMPS&F CAMP SCOTT FURPHY PTY. LTD.	**ROCKHAMPTON FLOOD MANAGEMENT STUDY** **1991 FLOOD CALIBRATION** **FLOOD LEVEL HYDROGRAPHS AT YAAMBA, WATER TREATMENT WORKS AND BARRAGE**	**FIGURE 2-4**

SECTION 3

3. OPTIONS FOR FLOOD MITIGATION

3.1 INTRODUCTION

As discussed in the Phase 1 Report, flood damages may be divided into direct damages ie. those caused by the physical contact with floodwaters, and indirect damages. The latter being those damages consequent to, but not directly caused by floodwater.

In the case of Rockhampton, it was established that a significant proportion of the damage in the 1991 flood was caused by indirect losses which resulted from the closure of road, rail and air links effectively isolating the city for nearly 2 weeks.

Whilst most of the flood mitigation options considered address direct damages, some of the works considered at Yeppen Crossing, and to a lesser extent at Rockhampton Airport, are aimed primarily at reducing indirect losses, by reducing closure times of the major transport links.

In order to put the flood mitigation options considered in this section into perspective, it is worth recalling here the magnitude of flood damages under current conditions, the distribution of direct flood damages throughout the flood liable area, and the relative scale of direct and indirect flood damages, so that flood mitigation works can be concentrated where they will be of greatest benefit. The following summary given in Tables 3-1 and 3-2 is drawn from various tables in the Phase 1 Report.

TABLE 3-1

Summary of Flood Damages

Damage Type		Mean Annual Damage $ million
A	Direct Flood Damage	
	Residential	0.2
	Commercial	0.8
	Public Sector	1.0
	Total	2.0
B	Indirect Flood Damage	3.2
	TOTAL	5.2

It should be emphasised that the cost of doing nothing to reduce flood damages may well not be the cheapest option, and it costs an average of $5.2 million each and every year ie. the long term average taking account of the range of flood magnitudes and their associated probabilities. Whilst this cost does not fall totally on the Local Authorities or State Government, it is a cost to the Australian economy. The cost of implementing flood mitigation works should be viewed in this context and in regard to the reduction of this average flood damage cost.

TABLE 3-2

Distribution of Direct Flood Damages

Zone No.	Location	Mean Annual Damage ($,000's)				% of Total
		Residential	Commercial	Approx Public Sector	Total	
	South of River					
10	Crescent Lagoon	9.2	60.3	37.8	107.3	9
11	Gladstone Road	5.7	225	28.8	259.5	23
12	Port Curtis	12.6	38.1	28.8	79.5	7
13	Central CBD	13.1	138	10.0	161.1	14
14	Lower CBD	36.8	189	10.0	235.8	21
15	Depot Hill	8.2	20.4	10.0	38.6	3
16	D/s Barrage	5.4	6.9	9.2	21.5	2
17	Pink Lily	9.4	1.4	10.0	20.8	2
18	U/s Barrage	0.8	4.3	7.0	12.1	1
	North of River					
20	D/s Barrage	3.5	13.5	8.5	25.5	2
21/22	Splitters Creek	21.6	13.4	15.0	50.0	4
23	Moores Creek	7.0	26.1	25.0	58.1	5
24/25	Lakes Creek Road	26.9	32.3	8	67.2	6
	TOTAL	160	769	208	1,137.0	100

Note: Public Sector Costs are based on RCC costs as only the urban area is considered herein.
MAD costs for RCC based on total cost of $1.52 million for 1991 flood and weighted average of residential and commercial MAD of 13.7% of 1991 damages.

Table 3-2 shows the approximate distribution of mean annual damage (MAD) throughout the urban area. It is clear from these values that the highest priority areas in terms of reducing flood damage costs are in the Gladstone Road/Lower Dawson Road area, and the lower part of the CBD, which together account for some 44% of MAD. The Central CBD accounts for a further 14% of MAD but is at risk to only rare floods.

In the case of Rockhampton, there is no easy solution to flooding, and because of its location, it will never be possible to eliminate flooding. The consideration of a wide range of flood mitigation options in the Phase 1 Report discounted several options including those which could reduce flood damages by means of reducing the peak flood flows, whether by storage or diversion. Hence, flood mitigation in Rockhampton must deal with the full natural flow of the Fitzroy River during flood.

In order to put the practical range of options considered herein in perspective, it is instructive to first consider two extreme scenarios, neither of which are regarded as being desirable or acceptable:

- firstly, to eliminate flows in the floodway to the south of the city by means of constructing levee banks along both banks from upstream of the city to well downstream. The effect of this would be to raise flood levels in the river considerably, at high cost. The consequences of overtopping if and when the design flood is exceeded would be considerable. The cost of such a scheme would be high, although this has not been considered in detail. Basically constraining the river to run between raised banks would not be considered a wise course of action;

- the opposite end of the spectrum of possible measures is the construction of a major floodway to the south of the city. This would need to carry about 50% of the total river flow in order to be effective in reducing flooding. The cost would be high, of the order of $200 million. Modelling, see Section 2.7.8, has shown this to be impractical.

Neither of the above are considered to be practical, nor do they offer least cost or high benefit-cost ratio solutions. The more practicable options fall generally within these two extremes.

The bulk of the flooding of the high risk areas arises from the flood flow in the Pink Lily – Yeppen – Gavial Creek floodway, with only the lower part of Quay Street and relatively minor flooding on the north bank of the river resulting directly from river levels exceeding bankfull in the areas subject to flooding from the immediate vicinity. This suggests that reduction in flood flows and/or levels in the floodway is likely to provide the most appropriate means of reducing the flooding problems in Rockhampton.

The options considered are discussed in the following paragraphs. The findings of the hydraulic model study are drawn upon in determining the impact of various options on flood levels (as discussed in section 2.7); costs given in the Phase 1 Report are updated and refined; and social/environmental impacts are discussed .

Options are initially considered separately and then in various combinations. Finally, recommendations are made in regard to preferred options. The options considered generally give protection to the 1% AEP flood level, except for upgrading of Yeppen Crossing where this is considered impractical. The worth of providing protection against more extreme floods is considered for the preferred options only.

The results from the hydraulic model studies of the options considered have already been outlined in Section 2.7. Figures in this section further summarise this information as well as adding costs and other considerations. In order to highlight the economics, hydraulic and other impacts of the various schemes, a number of diagrams have been included to emphasis the most salient points in order to try to simplify consideration of the options.

3.2 LEVEES

3.2.1 General

Levees are low earth embankments built to exclude flood waters. They have advantages and disadvantages which should be clearly understood by the community in deciding whether to proceed with any proposed levees. There a many examples of successful levee schemes in Australia and overseas.

Levees are often the most economically attractive form of protection to flood liable areas. They exclude all flood waters from the protected area for all floods up to some selected design flood. Their chief disadvantage results from this limitation in that they may overtop in some flood greater than that for which they are designed, unless designed to protect against probable maximum flood. This overtopping may be accompanied by failure of the levee. Subsequent damage in these circumstances is made all the worse because of the expectation of protection. This impact is minimised by good design which incorporates spillway sections in the levees to allow controlled overtopping in the event of extreme flood together with good construction practices and an appropriate level of maintenance. This allows time for evacuation and prevents catastrophic failure.

Levee construction should be accompanied by a community education and awareness program to ensure that the benefits and limitations of levees are realised.

Other negative impacts are the effects on flood levels elsewhere in the floodplain, and problems with internal drainage which requires storage, and in extreme cases may require pumped outlets to be provided.

In spite of these problems, which as stated above may be minimised by appropriate design and by community education, levees can provide a high level of community benefit.

For example, by preventing flooding over the full range of floods up to the design flood, significant reduction in flood damages can accrue. Furthermore, any land protected by the levee which was previously undeveloped because of its flood liable nature, may become available for development. Property values tend to rise due to rezoning and subsequent development of vacant land, and also values of existing property may increase due to the lowered flood risk. As property values rise, and/or land is developed, Council rates income increases. In Rockhampton, where there is little development potential close to the business district, this could be a substantial benefit, which has not been included in the benefit-cost analysis.

A summary of advantages and disadvantages of levee schemes is given below.

Advantages	Disadvantages	Overcome by
Reduction in mean annual flood damage	Failure due to overtopping	Design/maintenance
Reduction in social impacts of flooding	False sense of security	Education/warning
Improved property values	Increase in flood levels elsewhere	Compensatory works if increase unacceptable
Scope for additional development		

The above are taken into account in regard to the various options considered.

Cost estimates for levee schemes have been revised and refined on the following basis, but are still regarded as preliminary pending geotechnical investigation and final route selection. The basis for cost estimation assumed the following:

- side slopes of 1 vertical to 4 horizontal, with 4 m crest width. Passing lanes of 6 m crest width were allowed every 250 m allow passing of maintenance vehicles, whilst the former allows use of other than first grade materials for construction. A 4 m crest width is regarded as a minimum, 6 m is preferable but at additional cost. The provision of passing lanes is an appropriate compromise in this regard;

- a freeboard of 0.6 m above design flood level over and above making allowance for any increase in flood levels caused by the levee itself;

- generally the 1% AEP flood has been defined as the design flood. Whilst the freeboard allowance does not strictly give protection above the design flood as it allows for subsidence of the crest, and wave action, in practice some further protection is provided in this way. Spillway sections would be designed to overflow at the design flood level;

- alignment accommodated by easement thereby minimising land acquisition costs;

A higher level of protection may be provided at relatively low cost as the difference in level between the 1% AEP flood and the 0.1% AEP flood is only of the order of 1 m. This is considered where appropriate after the primary consideration of options.

The following paragraphs discuss the individual levee schemes considered. Their location is shown in Figures 3-1 to 3-3. A summary of schemes, their costs and impacts are given in Figure 3-4.

Cost estimates are given in Appendix G, which also gives revised flood damage reduction figures derived by re-running the ANUFLOOD model developed in Phase 1 with the revised flood height/probability curve derived from the design runs of the hydraulic model.

3.2.2 Protection of Port Curtis, Depot Hill and the lower CBD

As outlined in the Phase 1 Report, it would be possible to protect the whole of Port Curtis, Depot Hill and the lower CBD by a single levee. Alternatively, separate schemes could protect i) Port Curtis, ii) Depot Hill and the lower CBD. As the Phase 1 Study showed a combined levee to be the most economic solution, a separate Port Curtis levee has not been considered further. Consideration in Phase 2 has been limited to 2 schemes, namely the single levee protecting all the areas named above, and a second scheme excluding protection to Port Curtis.

A levee protecting Port Curtis, Depot Hill and the CBD would eliminate all flooding in these frequently flooded areas up to the selected design flood, which would be at least 1% AEP, and would eliminate flooding along Lower Dawson Road to the same limit.

Further consideration has been given to the alignment of these levees, as shown in Figure 3-1. These alignments will minimise disruption to existing landowners whilst providing maximum protection to the flood protection area. The alignments have not, however, been finalised.

The revised cost of these schemes are as follows, breakdowns for which are given in Appendix G:

- Depot Hill – Lower CBD only (Option A1) to 1% AEP flood level plus freeboard, cost $5.7 million.

- Lower Dawson Road – Port Curtis – Depot Hill – CBD (Option A2) to 1% AEP plus freeboard, cost $6.9 million.

Option A1 would reduce mean annual damage (MAD) by $300,000 p.a., which has a net present value (NPV) of $5.8 m at 5% discount rate, reducing to $4.3 million at 7% discount rate. The benefit–cost ratio (BCR) of this scheme is thus 1.01 at 5%, (0.75 at 7%) assuming all construction costs are in year 1 and ignoring maintenance costs.

Option A2 is considerably more attractive in economic terms. Construction cost has been estimated to be $6.9 million, with a reduction in MAD of $0.49 million. This gives an NPV of $9.3 m at 5% discount rate ($6.9 million at 7%) and corresponding BCR of 1.33 (1.0), which is high for a flood protection scheme.

As well as the economic advantages of this scheme, the social benefits would be high as they would greatly reduce the risk of flooding in the protected areas, and also encourage further development in these areas.

As discussed in Section 2.7, construction of the Depot Hill/lower CBD levee would have very little effect on flood levels, but the Port Curtis part of the combined levee would have a substantial effect as it closes off a significant flow path. The result of the above would be to raise levels in the floodway by 0.9 m downstream of the Yeppen crossing for a 1% AEP flood and by 0.4 m on the upstream side of Yeppen, and about 0.15 m at Fairybower Road. As these increases are not likely to be acceptable, the overall performance in combination with other options was considered. This is discussed in Section 3.6.

A levee to protect Depot Hill and the CBD would offer substantially economic benefits, but would have only a marginal impact on flood levels. However, this would be disadvantageous to the residents of Port Curtis whose current sense of isolation from the community would be heightened. Flood hazard at Port Curtis would remain high as it is as present. There is, therefore, a significant social cost in excluding Port Curtis should a levee be constructed to protect Depot Hill and the CBD.

If constructed, the Port Curtis to CBD levee would commence on the city side of the Yeppen crossing. It would be tied into higher ground along and adjacent to Blackall Street. The levee would then be built along the southern side of Blackall Street to its junction with Lower Dawson Road. A section of Lower Dawson Road from Yeppen bridge to the Blackall Street/Jellicoe Street junction would be ramped to tie in with the levee crest, with a ramp down on the city side. This would be a fairly flat grade. If this option were built in conjunction with raising the road level across the Yeppen Crossing, this ramp would be less marked on the southern side. The levee would then be constructed along the southern edge of Jellicoe Street to the railway crossing. The railway crossing will have to be raised from the Yeppen bridge to the levee crest level at Jellicoe Street, and regraded towards Port Curtis junction. Alternatively, this section could be left at its existing level and fitted with flood gates but this would not be preferred. The levee would be built along the southern edge of Jellicoe Street to near the junction with the Old Burnett Highway, where it would divert to the south so that it could cross the Old Burnett Highway and pass on the floodway side of Hastings Deering Pty Ltd. The levee would then turn northwards past the last house in Port Curtis (along the Old Bruce Highway) and pass to the east of Depot Hill. The route in this section has not been determined but would be as far to the east as possible in order to maximise the area available for storage of internal drainage (or for development). The levee would then pass close to Gavial Creek (between the sewage treatment works and the creek), turn along the right bank of the river, terminating near Derby Street. There is generally sufficient room for this section of levee, although where space is restricted short sections of retaining wall may be required. These details would be determined at the design stage. Flood valves will need to be provided on all drainage outlets along the levee route, to prevent backflow up the drainage lines. They would prevent the flooding of a large part of Allenstown which currently can be flooded by backwater from the main drain. At least two spillway sections would be incorporated into the levee, probably one on the river side upstream of the Gavial Creek Junction, and one on the opposite side between Depot Hill and Gavial Creek. This would allow controlled flooding to occur in the event of the levee being overtopped in an extreme flood and thereby enable water to accumulate behind the levee to prevent failure if the entire levee overtops. It should be noted that even in such an extreme situation flood, damages would be no worse than under existing conditions.

A separate levee for Depot Hill/lower CBD would need to start in the Port Curtis Junction area and would need to include local raising of the highway and railway and be tied into higher ground next to the highway to avoid overflow into the Allenstown area.

3.2.3 Protection of Rockhampton Airport

As discussed in the Phase 1 Report, the protection of Rockhampton Airport would require levees. In order to protect the road access to and from the airport, the levees should be extended to encompass the adjacent residential areas. The indicative preferred location is shown in Figure 3-2.

The levee has only been shown for the protection of the existing airport, as no details are available about the extension being considered. The preferred option is to tie the levee to high ground to the south of the airport and to build the levee on the western side of the runways and then close to the southern bank of Lion Creek, passing behind the residential properties and terminating along the river bank upstream of the barrage. This would provide considerable protection to the adjacent residential areas as well as maintaining the airport in operational order for the 1% AEP flood (or higher).

This would however have a negative impact on flood levels on the small number of houses in the floodplain. Levels along Nine Mile Road would be increased by a maximum of 0.3 m for 2% AEP, and 0.56 m for 1% AEP, so the houses adjacent to the proposed levee would need to be raised. The increase in flood level along the Rockhampton-Ridgelands Road would be 0.06 m for 2% AEP and 0.12 m for 1% AEP, which is regarded as being acceptable.

In this way the airport could be protected to 1% AEP or higher. Protection to 1% AEP would require a levee about 2.75 m high (including allowing 0.6 m freeboard and for local increase in flood level) along the boundary adjacent to Lion Creek.

The estimated cost given in the Phase 1 Report included protection of the proposed extended runways. In regard to protection of the existing airport, the revised cost is $4.3 million, a breakdown of which is given in Appendix G. Using the approximate reduction in MAD of $102,000 p.a. given in Table 14-1 of the Phase 1 Report, the NPV at 5% is $1.94 million and at 7% is $1.43 million, giving BCR of 0.45 and 0.33 respectively. Thus this scheme is not viable in purely economic terms, but may be justifiable in terms of reduction in social impacts which would result from being able to maintain the airport in operation condition in a 1% AEP flood.

3.2.4 Protection of other areas

Levees along the north bank of the Fitzroy River in the Splitters Creek area was briefly investigated. The Splitters Creek levee has some merit in regard to closing off a minor flow path, so that this area would be limited to flood storage (ie. low velocity) flooding.

The breakout could be closed off by a partial levee as shown on Figure 3-3 for a cost of about $140,000 (see Appendix G for details). This would not prevent flooding in the Splitters Creek area but would limit this to backwater flooding. Full levee protection requiring a floodgate across Splitters Creek would present practical difficulties resulting from its proximity to the barrage and has not been considered in detail. Assuming the reduction in MAD to be a third of that from the total protection this would give a BCR of about 1.2 at 5% discount rate (0.9 at 7%).

Phase 1 studies had shown levee protection of other areas such as Lakes Creek Road and the Moores Creek area to be not cost-effective and these were not pursued in Phase 2. However, as the bulk of flooding in these areas is by backwater from the river, it would require only the provision of flood gates on the major creeks and stormwater drains to given some measure of flood protection (to about 2% AEP). This has been included as a low priority item in the recommended works summarised in section 5.

3.3 UPGRADING OF YEPPEN CROSSING

As discussed in the Phase 1 Report (section 13.5), the highway and railway crossings of the Fitzroy River floodplain to the South of Rockhampton, known as the 'Yeppen Crossing', were reconstructed in the 1980's. The design flood immunity of the crossing is 8.5% AEP (12 year ARI). The actual performance of the crossing in the 1988 and 1991 flood is consistent with this design criterion. The average time of submergence was estimated to be 0.58 days per year. As the 1988 flood was the fourth highest since discharge records begin in 1914 it is apparent that had the current floodplain conditions existed throughout this century only the major floods of 1918, 1954 and 1991 would have caused closure. The hydraulic model has shown that times of submergence for these floods would have been 15, 12.5 and 11.5 days respectively ie. a total of 39 days in 78 years or an average of 0.50 days per year. Thus on this basis also the crossing is performing as designed.

Notwithstanding the above, it is clear that the indirect losses caused by closure of this crossing are high and could be substantially reduced by further upgrading of the flood immunity of the crossing. A statement from the Department of Transport in regard to their position concerning this upgrading proposal is given in Appendix L.

A summary of the options considered in regard to upgrading of Yeppen crossing are given in Figure 3–5. The hydraulic impacts of these options have been outlined in Section 2.7.

It was apparent from the investigation of individual options for Yeppen Crossing that only those combining an increase in waterway area with an increase in embankment height would be able to improve the flood immunity of the crossing without negative impact on flood levels. Hence, only these combinations are considered further here.

This paragraph will only consider options for improvement of the flood immunity and flood level impact of Yeppen Crossing and not combinations of the above plus other possible schemes. Such combinations are considered in section 3.6.

The existing bridge and embankment structures across the floodplain at Yeppen comprise 4 road and 4 rail bridges.

These structures cause significant afflux during major floods. Although reduction in afflux would be beneficial to flood levels in the Fairybower area and to a lesser degree at the airport, flood damages in these areas alone are not sufficient to warrant works to reduce afflux by increasing bridge waterway area.

Also simply raising the embankments without increasing waterway area has a negative impact on upstream levels but very small reduction to submergence times.

However, the combination of increased waterway area and raised embankment height offers significant reduction in submergence time together with some improvement in flood levels. The options considered in this regard (B5 and B7) would both maintain flood free conditions for 2% AEP flood (eg. the 1991 flood) with time of submergence for 1% AEP being reduced from about 12.7 days under existing conditions to 6.8 days for Option B5 and 8 days for Option B7. Average closure time would be reduced to 0.15 days per annum.

Under Option B5, each of the bridges would be doubled in length to double the bridge waterway area, and the embankment would be raised so as to give constant road and rail heights across the entire length of the crossing. It is emphasised that, whilst doubling of bridging length is shown by the hydraulic model studies to be appropriate, this should not be taken as final design dimensions of these structures. The individual bridges will need to be designed to ensure that they meet design criteria for velocity and afflux. This is outside the scope of the current study.

The cost of upgrading as outlined above has been estimated to be $16.5 million.

Option B7 represents a lower cost alternative in which the additional waterway area would be obtained by excavating an average of 2 m from upstream of the highway bridges through to downstream of the railway bridges. The hydraulic model runs showed this to be almost as beneficial as doubling bridge length, in conjunction with raising embankments. An initial consideration of the structural implications of this has shown this to be feasible. In the case of the highway bridge, DOT have indicated that no bridge strengthening would be required, but in the case of the railway bridges the pile caps would be exposed, requiring some structural works and possibility the installation of some additional piles. However, detailed structural calculations in this regard, are outside the scope of the study.

It would also be necessary to provide some protection works in the lowered sections in order to prevent continuing erosion. Gabions/reno mattresses would be suitable in this regard. This option could have a relatively high maintenance cost, as small floods may cause siltation in the lowered section. This tendency would be minimised by limiting the slope of the downstream ramp. As floodplain flows occur only on a frequency of 1 year in 7 on average, this should not be a major problem. The lowered sections would be drained to Scrubby Creek to prevent permanent water below the bridges.

The cost of this option, at $13.0 million, offers a substantial saving over Option B5. This cost includes for bridge strengthening measures expected to be sufficient. However, this is an inferior solution which would result in increased maintenance.

Estimates given in section 8 of the Phase 1 Report in relation to the 1991 flood show direct damage to the combined Yeppen crossing of $1.2 million and indirect damage resulting from road and rail traffic delays of about $5 million, giving a total of about $6.2 million. In addition to the above, the closure of this route is one of the main causes of indirect losses to the commercial/industrial sector. Assuming these losses are in proportion to the traffic delay losses for north and south links (ie. 75% of the total), this would result in an indirect loss of about $22 million (using the adopted value of $32 million for indirect losses to the commercial/industrial sector given in Table 8-11 of the Phase 1 Report). On the above basis, which is only approximate, the total losses caused by the closure of the crossing in the 1991 flood were about $28 million. Assuming, further, that the damages are proportional to the time of submergence, this gives a total damage of cost of about $2.5 million per day of closure.

The preferred options B5 and B7 would produce a flood free crossing at 2% AEP with reduced times of submergence of 6.8 and 8 days at 1% AEP. These times vary slightly when these measures are combined with others. The damage values and times of submergence were used to estimate a damage probability curve as given in Figure 3-6.

Times of submergence (TOS) for the range of design floods (5% AEP to 0.1% AEP) together with the existing design of zero TOS at 12.5% AEP (as per the current design and as evidenced by the 1988 flood) were used to prepare a curve relating TOS to event probability (Figure 13-6 a). From this a damage/probability curve was derived assuming that indirect losses caused by the crossing being closed are proportional to closure time. This may be conservative for short duration of closure but is considered to be reasonable for the longer durations. It should be noted that for the purposes of this report TOS is defined as the time for which there is flow across the road. This is not necessarily the time of closure.

The area under this curve was integrated to estimate the mean annual damage. This was estimated to be $1.75 million to $2.1 million depending on the curve adopted from Figure 3-6. The higher value relates to the TOS given by the design runs but this is based as the shape and hence duration of the design hydrographs which are based only on observed hydrographs for the 1991 flood (which was used for AEP to 1%) and 1918 used for more extreme floods. The lower value relates to an interpolated curve given in Figure 3-6 which may be more realistic. This above figures compare to $1.6 m used in Phase 1, when extrapolation to the more extreme floods was not available.

The reduction of MAD by Options B5 and B7 is shown in Figures 3-6. These are very similar, the residual MAD being $0.45 million and $0.43 million respectively. The latter should be reduced to reflect the increased maintenance cost of say $0.1 million per annum. On this basis, the reduction in MAD is $1.3 million for Option B5 and $1.18 million for Option B7.

Net present value (NPV) of Option B5 is thus, $24.3 million at 5% discount rate ($18.2 million at 7%), with BCR of 1.50 (1.10). The corresponding NPV for Option B7 is $22.4 million ($16.5 million) and BCR of 1.72 (1.27). Thus even accounting for increased maintenance, Option B7 has a preferable BCR.

As well as these schemes, particularly B7, being justifiable economically, they would also have a significant social impact as these schemes would not only greatly reduce the disruption to the movement of persons and goods into and out of Rockhampton during floods, but would also significantly improve the sense of isolation caused by the closure of the major crossings.

The Department of Transport subsequently advised that Option B7 involving lowering of the bridge inverts would not be acceptable.

3.4 CONTROL OF PINK LILY BREAKOUT

The necessity for works to stabilise the right bank of the Fitzroy River in the breakout section at Pink Lily has been discussed in detail in the Phase 1 Report. The level at which the banks should be stabilised was to be considered as part of the hydraulic modelling studies in Phase 2.

For a given river level the breakout at Pink Lily controls the flow of water in the floodplain. Hence raising or lowering the control level would alter the distribution of flows between the river and the floodplain. A number of options were investigated in regard to varying this control to offset the impact of existing structures and of those being considered in the current study. These options are summarised in Figure 3-7.

It was found that a minor raising of control levels of, say, 1 m throughout the breakout zone was ineffectual (Option D1). The banks at Pink Lily could be raised so as to eliminate overflow for 2% AEP (Option D2) and this would cause the existing Yeppen crossing to remain open even at 1% AEP. However, this would have a major impact on flood levels and hence flood damages in the city area, where levels would be raised by about 0.5 m. At the barrage, levels would be raised by 1.1 m for 2% AEP and 2.3 m for 1% AEP, with similar increases to upstream of Pink Lily. Even flood levels at Yaamba would be raised by 0.11 m for 2% AEP, and by 0.15 m at 1% AEP. These increases were regarded as unacceptably high, and this option was not pursued. A compromise raising of 2.5 m (Option D3) was also briefly investigated.

Whilst none of the above were found to be acceptable on their own, the lower degrees of raising in the 1 m to 2.5 m range were thought to be of possible use in combination with other measures. If such a scheme were to be promoted, it would probably be a combination of raising the control level and reducing the length of the overflow path to ensure damaging overtopping did not occur. Lowering of the control level was not considered as it is not practical and would worsen levels in the lower floodplain. Increasing floodway capacity by means of a major channel is considered in the following paragraph.

As none of these options are considered to be worthwhile, except possibly in conjunction with other measures, it is recommended that the banks at Pink Lily be stabilised at their current levels.

3.5 MISCELLANEOUS OPTIONS

3.5.1 Major Improvements to Floodway Capacity

The option of a major floodway to the south of the city was briefly investigated using the hydraulic model. This option had been discounted in Phase 1 due to limited effectiveness, high cost and high environmental impact. It was, however, given further consideration in Phase 2 as a result of having been raised in the Community Consultation process.

A summary of the findings are given in Figure 3–8. Even with a channel with 1,000 m base width and a further 1,000 m width in the right overbank area once a depth of 3 m was reached, such a channel would only carry 3,500 m^3/s in a 2% AEP flood and 4,600 m^3/s in a 1% AEP flood. This general channel shape was used so that a potential benefit of such a scheme could be to provide levee protection, or allow filling of the left bank area, ie. the area between the channel and the city. However, even with a channel of this size, levels through the city reach would be increased by 0.3 m at 1% AEP, thereby rendering the proposal ineffectual. Due to the lack of hydraulic performance this was not considered further. As such a scheme would be of very high cost, would also cause severe environmental damage and involve resumption of a substantial land area, it was concluded that this did not warrant further consideration.

3.5.2 Minor Improvement to Floodway Capacity

The old Burnett Highway bridge across Scrubby Creek and the associated causeway across the Yeppen floodplain is still in existence close to the downstream side of the new Yeppen railway crossing.

Also the disused railway embankment adjacent to the Old Bruce Highway between Port Curtis and Roopes bridge is still mainly intact. Both of these structures impede the passage of floodwaters. Whether there is any benefit in removal of these structures was considered using the hydraulic model. This was found to have only a marginal impact on flood levels (Option F3) on its own, but subsequently was found to be effective in partly offsetting the increase in flood levels caused by the levees around Port Curtis being considered.

The cost of these works was estimated to be $500,000.

A partial floodway was also considered in relation to the above, by means of excavating a channel from downstream of Yeppen Crossing to discharge downstream of Gavial Creek. However, because of flat grades and the wide nature of the floodplain, this was found to have very little impact on flood levels and was not considered further.

3.5.3 Effect of Commonage Landfill

The effect of the Commonage Landfill was investigated using the hydraulic model. As the flow in the adjacent overbank section was very small, the impact on flood levels of increasing the size of the landfill was found to be insignificant.

However, the environmental concerns in regard to the presence of the landfill in the floodplain, as expressed in the Phase 1 Report are reiterated.

3.5.4 Lowering of the Capricorn Highway

Whilst a high level of flood immunity for the Capricorn Highway between Gracemere and the Bruce Highway is not a high priority due to the existence of an alternative flood free route, the question had been raised during the Community Consultation Phase of the impact on flood levels of this section of the Capricorn Highway being of the order of 1 m above surrounding ground level in places.

The effect of this was investigated in the model by lowering the relevant sections by 1 m (Option M3). This was found to have virtually no impact on peak flood levels, so action in this regard is not warranted.

3.6 COMBINATIONS OF OPTIONS

Consideration of the individual flood mitigation options in the sections above has demonstrated that upgrading the Yeppen Crossing by both raising embankment height to improve flood immunity and increasing bridge waterway area to counteract the increase in water levels which would otherwise occur, is the most cost effective means of reducing indirect damages.

Further hydraulic model runs were carried out with a range of combined options in order to determine whether the negative impact of the proposed levee schemes could be offset by the other measures under consideration. These combinations are summarised in Figures 3-9 and 3-10.

Of the levee schemes considered, the Port Curtis - Depot Hill - CBD levee is very beneficial in terms of reducing flood damages. Together with the option of lowering the bridge inverts, this would cause increase in flood levels immediately upstream of the Yeppen Crossing in a 1% AEP flood but not in the Fairybower Road area. Protecting the smaller area of Depot Hill and the lower CBD only avoids this problem but to the detriment of the residents of Port Curtis. In the option with bridge duplication, no such increase in level would occur. The proposed levee around the airport is believed to be beneficial, if not justifiable on purely economic terms, due to its enabling the airport to continue operation during a 1% AEP flood (or possibly higher). Construction of this levee would result in flood levels being increased close to the airport but reduced downstream.

Therefore the following (Options C8/C9) are put forward for final consideration:

- Construction of a levee from Port Curtis - Depot Hill - Quay Street, together with upgrading of Yeppen Crossing, removal of the Old Burnett Highway bridge and the disused railway embankment adjacent to the Old Bruce Highway, levee protection of the existing airport and the adjacent residential area, and levee protection to prevent breakout from the river in the Splitters Creek area.

The cost of the total combined scheme is estimated to be \$23.9 million if the airport and Splitters Creek levees are excluded or \$28.2 million with these included (Option C9). Overall BCR for these alternatives are 1.40, 1.26 at 5% discount reducing to 1.04, 0.93 for 7% discount rate.

The preferred option should be compared to the cost of the 'do nothing' option. If no flood mitigation works are constructed, even though there may be no flood and hence no flood damages for some years, the long term average damage cost has been estimated to be \$5.2 million per annum, together with significant social disruption during and after each major flood. Floods occur in a random sequence, so the fact that a major flood has occurred recently is no guarantee that there will not be a flood of similar or greater magnitude in the near future.

As the preferred schemes would substantially reduce the long term average flood damage, and also substantially reduce the social impacts of flooding they are recommended for implementation. Possible funding of such works is discussed in Section 5.

In the above combinations the levee components are based on protection against 1% AEP flood. Costs and benefits associated with the recommended levees were then considered for a range of higher levels in order to determine the most appropriate level of protection.

In regard to the CBD levee, the cost differential between protection to 1% AEP and 0.5% AEP is about \$1.45 million ie. a total cost of \$8.35 million. The benefits however, increase considerably as the MAD reduction is improved significantly from \$0.49 million to \$0.63 million. The BCR at this level is increased to 1.43 at 5%, 1.05 at 7%, compared to 1.35 (1.0) respectively at 1% AEP. However, a levee giving protection to 0.5% would have some adverse impact on flood levels, for example raising the level upstream of Yeppen Crossing by 0.2 m to 0.3 m depending on whether the airport levee is also constructed.

Further increase in protection level beyond 0.5% AEP would result in a significant cost increase in order to prevent direct breakout from the river in the city area (which first occurs between 0.5% AEP and 0.2% AEP). This would require construction of a retaining wall along Quay Street to at least the Fitzroy Street bridge. MAD reduction would rise to $0.77 million, and the estimated capital cost would be $10.1 million. This would give a BCR of 1.45 at 5% and 1.07 at 7% discount rate. Such a levee would also have a significant impact on flood levels in the floodplain for floods in excess of the 1% AEP event.

It is apparent on the basis of the above that protection to levels higher than 1% AEP flood have marginally beneficial BCR's at higher initial cost. Levees to 0.5% AEP would keep out a flood greater than the 1918 flood, which is a reasonably high degree of protection. However, as protection above the 1% AEP level starts to have a negative impact on flood levels in the floodplain for events between 1% AEP and levee overtopping, it is recommended that protection should be limited to the 1% AEP flood.

In regard to the airport levee, the additional cost of raising from 1% AEP flood level to 0.5% AEP flood level would be $1.3 million, raising the cost to $5.6 million. Further raising to the 0.2% AEP level would incur a substantial cost increase to $7.4 million. As this levee cannot be justified even to 1% AEP on economic grounds, a decision to raise the levee to above 1% AEP level would need to be based on disaster relief considerations, as the need for an operational airport could become more important as flood magnitude increases. However, again as increasing protection above 1% AEP causes detrimental impact on flood levels, this is not recommended.

The Yeppen Crossing upgrade has been based on maintaining flood free access at 2% AEP, as that is the maximum which can be practically achieved without major cost over and above that already considered. This is because providing a greater level of immunity would require raising of the deck level of the existing bridges, as well as any new works. This has not been considered.

3.7 EXTREME FLOODS

The operation of the preferred scheme under floods more extreme than the design flood was considered to ensure that the works would not be detrimental under such circumstances.

The hydraulic model was run with the 0.1% AEP flood assuming that the levees were not overtopped, with the following results compared to the corresponding values for existing floodplain conditions:

- peak levels in the Pink Lily to barrage reach would be increased by up to 0.29 m;
- peak levels in the city reach would be reduced by 0.09 m at the flood gauge;
- peak levels in the upper part of the floodway ie. Pink Lily to Nine Mile road would be up to 0.45 m higher than under existing conditions, and up to 0.59 m along Lion Creek;
- peak level at Fairybower Road raised by only 0.02 m;
- peak level upstream of Yeppen Crossing would be raised by 0.39 m, and by a maximum of 0.56 m downstream.

These impacts whilst significant in the floodplain are in areas of low density of occupation and represent increases in flood depth but not increases in flood frequency. The above figures are conservative in that they assume that the levees are not overtopped in such an event. As the recommended level of protection is for 1% AEP, more extreme floods would cause the levees be overtopped, whereupon the peak levels would be reduced from those reported above.

As considerable devastation would occur in any event in a flood of this magnitude, the increase in damages which would result from the presence of the works as a result of increased flood levels would be small. On the contrary, the presence of the flood mitigation works would allow adequate time for evacuation prior to an extreme flood with consequent reduction in flood damages and social impact. Hence, it is believed that the presence of the works would be beneficial even when the design flood is exceeded.

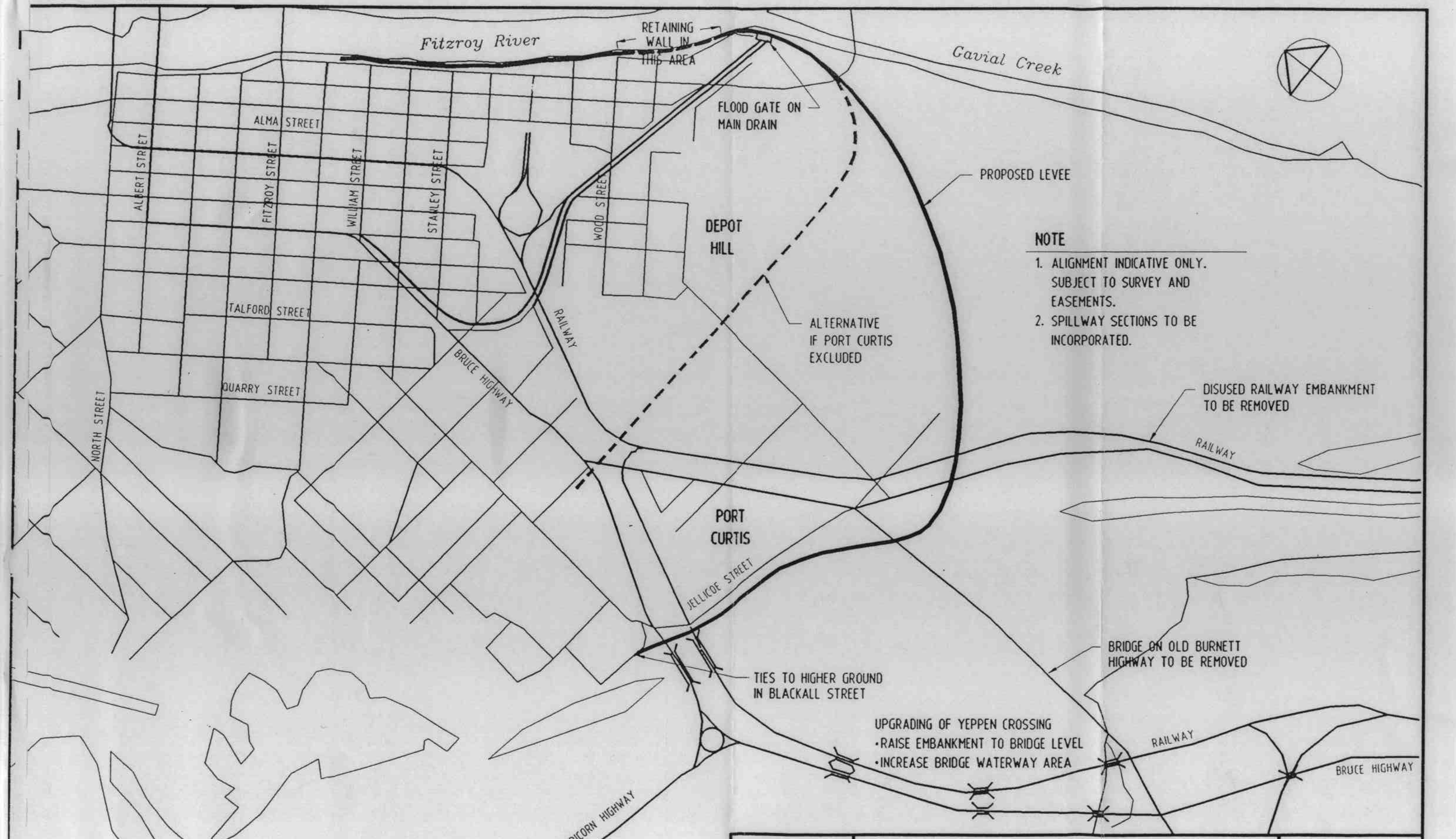
The effect on flood levels of 0.5% and 0.2% AEP floods for Options C8 and C9 were also considered. There are given in Tables J-30a and J-31 a (Volume 3).

3.8 RECOMMENDATIONS

Having taking account of the costs, benefit cost-ratio (BCR), impact on flood levels and of social consequences of flooding and flood protection, it was concluded that the most beneficial works for flood mitigation in Rockhampton are:

- Upgrading of Yeppen Crossing to provide flood free passage to both the Bruce Highway and the North Coast Railway at 2% AEP and to reduce times of submergence for larger floods. This will significantly reduce indirect flood damages;
- Construction of a levee to protect Port Curtis, Depot Hill, and the more flood liable part of the CBD from floods up to 1% AEP. This will significantly reduce direct flood damages and social costs without negative impact on flood levels compared to existing conditions. This also requires the removal of disused rail embankments and a disused highway bridge;
- Construction of a levee to protect Rockhampton Airport and the adjacent residential areas. As well improving the flood immunity of the airport this will reduce direct damages to the adjacent residential area;
- Construction of a levee to prevent overflow from the river in the Splitters Creek area.

These proposals have a marked positive social impact in regard to those members of the community who will be protected by the levees. However, a small number of persons outside the levees, particularly those in the area between the Rockhampton-Ridgelands Road and Lion Creek, and to a lesser extent those in the Fairybower area will suffer a negative impact because levels in any given flood will be increased, although the frequency of flooding will be unaffected. The impact on the latter groups will be offset to some degree by the proposed upgrading of the flood warning network which will enable, for the first time, forecasting of levels in the floodplain. All residents and businesses will benefit significantly from the marked reduction in social and business disruption resulting from improving the flood immunity of the Yeppen Crossing.



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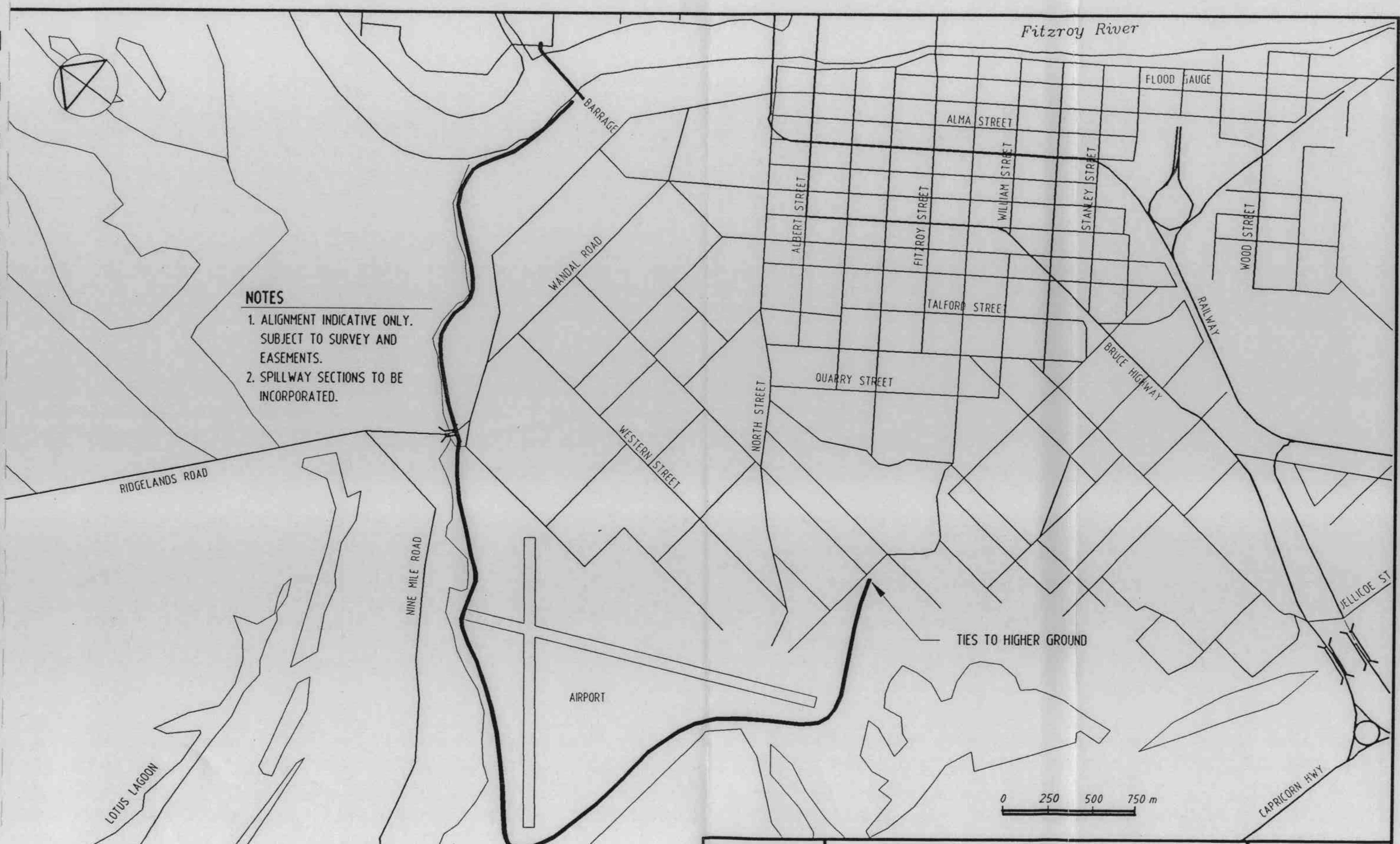
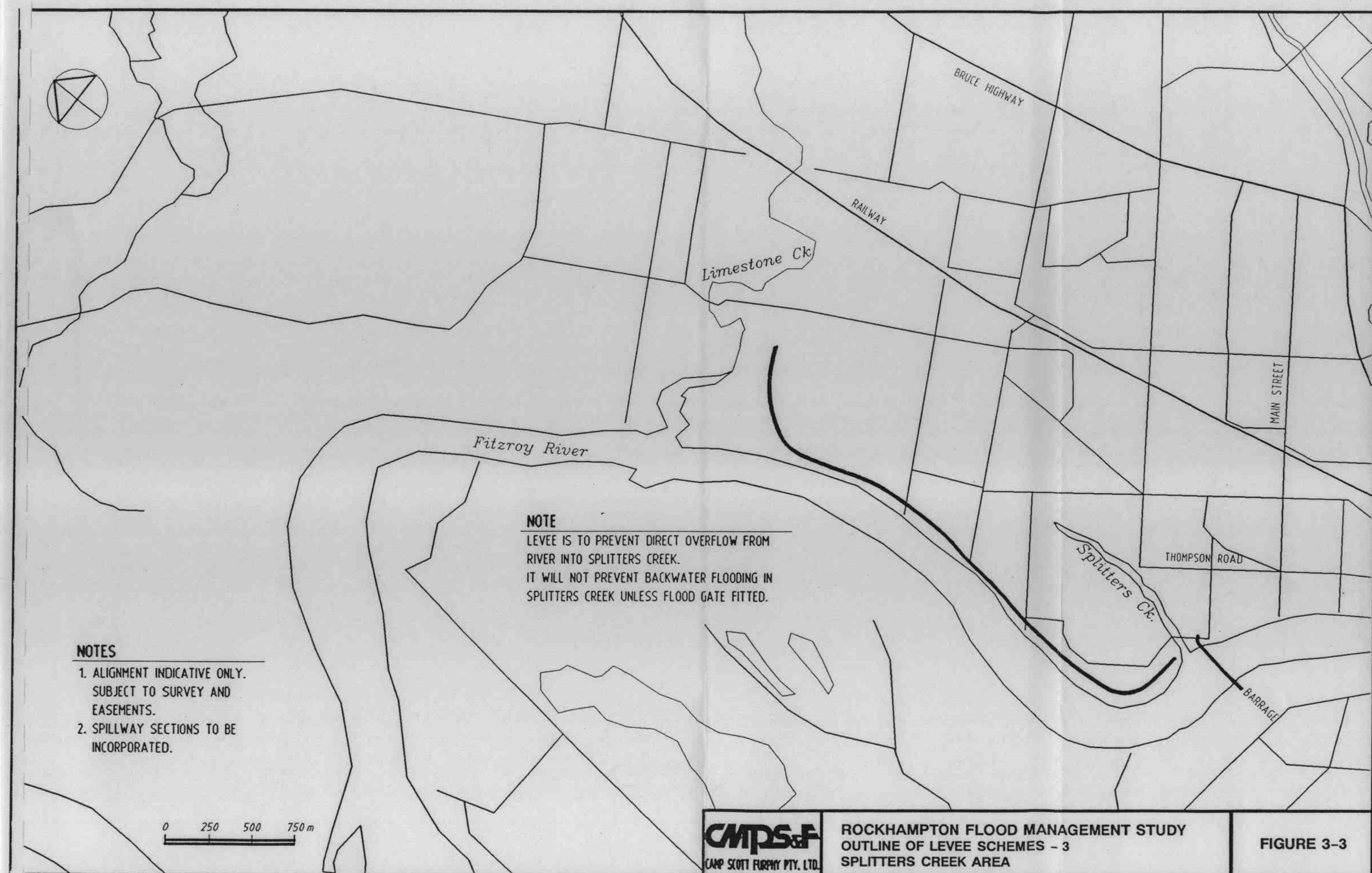
ROCKHAMPTON FLOOD MANAGEMENT STUDY
OUTLINE OF LEVEE SCHEMES - 1
PORT CURTIS - DEPOT HILL - CBD

FIGURE 3–1



NOTES
1. ALIGNMENT INDICATIVE ONLY. SUBJECT TO SURVEY AND EASEMENTS.
2. SPILLWAY SECTIONS TO BE INCORPORATED.

CMPS&F CAMP SCOTT FURPHY PTY. LTD.	ROCKHAMPTON FLOOD MANAGEMENT STUDY OUTLINE OF LEVEE SCHEMES - 2 ROCKHAMPTON AIRPORT AND ADJACENT AREAS	FIGURE 3-2



NOTE

LEVEE IS TO PREVENT DIRECT OVERFLOW FROM RIVER INTO SPLITTERS CREEK.
IT WILL NOT PREVENT BACKWATER FLOODING IN SPLITTERS CREEK UNLESS FLOOD GATE FITTED.

NOTES

1. ALIGNMENT INDICATIVE ONLY. SUBJECT TO SURVEY AND EASEMENTS.
2. SPILLWAY SECTIONS TO BE INCORPORATED.

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ROCKHAMPTON FLOOD MANAGEMENT STUDY
OUTLINE OF LEVEE SCHEMES - 3
SPLITTERS CREEK AREA

FIGURE 3-3

LEVEE OPTIONS

Option	Description	Table J–1
A1	Levee Depot Hill, Lower CBD	

(to 1% AEP)
Cost: $5.7 million
Reduction MAD: $0.30 m p.a.
BCR: 1.07 (0.75) **NPV:** $5.8 m ($4.3 m)
Impact on levels:
Minimal as not flow path.
Eliminates flooding to design level in area protected which suffers from high frequency flooding. Port Curtis still in floodway.

Option	Description	Table J–2
A2	Levee Port Curtis – Depot Hill – Lower CBD	

(to 1% AEP)
Cost: $6.9 million
Reduction MAD: $0.49 m p.a.
BCR: 1.35 (1.0) **NPV:** $9.3 m ($6.9 m)
Impact on levels:
u/s Yeppen Crossing +0.30 m, +0.42 m at 2%, 1% AEP.
d/s Yeppen Crossing +0.61 m, +0.90 m at 2%, 1% AEP.
Eliminates flooding to design level in protected area which suffers high frequency flooding.
Impact on levels too great as a stand alone measure – needs to be combined with other measures.

Option	Description	Table J–3
A3	Levee – Rockhampton Airport	

(to 1% AEP)
Cost: $4.3
Reduction MAD: $2.1 m p.a
BCR: 0.45 (0.33) **NPV**: $1.94 m ($1.44 m)
Impact on levels:
Increases levels along Lion Creek (outside levee) by max of 0.37 m at 2% AEP, 0.58 m at 1% AEP.
Reduces levels u/s Yeppen by 0.04 m, 0.08 m for 2%, 1% AEP.
Increase levels city reach of river by 0.03 m, 0.05 m for 2% AEP, 1% AEP.
Major benefit – keeps airport open to 1% AEP for emergency relief.

Option	Description	Table J–4
A4	Levee – Airport with proposed runway extension	

Details not available, modelled approximately, little change from A3.

Note: NPV at 5% (7%)

Option	Description	Table J–5
A5	Splitters Creek	

(to 1% AEP)
Cost: $140,000
Reduction MAD: $9,000 p.a.
BCR: approx 1.2 (0.9)
Impact on levels:
Negligible – eliminates minor flood path.

Option	Description
A6	Moores Creek

Impact on levels:
Negligible as flood storage only.

Option	Description
A7	Lakes Creek Road

Impact on levels:
Increases flood levels negligible as flood storage only.

Options shaded thus are carried forward for further consideration.

Summary of Levee Options
Figure 3–4

YEPPEN CROSSING

Option	Description	Table J-6
B1	Double bridge width	

Impact on levels:
Reduces flood level u/s of crossing by 0.27 m for 2% AEP, 0.29 for 1% AEP.
Reduces flood levels Airport, Fairybower Road by 0.08, 0.14 m respectively for both 2% and 1% AEP.
Reduces levels Depot Hill by 0.06 m, 0.1 m for 2% and 1% AEP.
TOS: 9.75 d, 11.95 d (current 11.6, 12.7 d)

Option	Description	Table J-9
B5	Combine B1 + B4	

Cost: $16.5 million
Reduction MAD: $1.3 m p.a.
Flood Free at 2% AEP
NPV: $24.7 m ($18.2 m)
BCR: 1.50 (1.10)
Impact on levels:
Reduces flood level u/s crossing by 0.17 m, 0.05 m for 2%, 1% AEP.
Reduces flood levels Airport by 0.05 m, 0.02 m for 2%, 1% AEP.
Reduces flood level Fairybower Road by 0.09 m, 0.02 m.
Reduces level at Depot Hill by 0.08 m, 0.15 m.
TOS: 0 at 2% AEP, 6.8 d at 1% AEP

Option	Description	Table J-8
B4	Raise road/rail to bridge level	

Impact on levels:
Increases flood u/s of crossing by 0.38 m for 2% AEP, 0.31 m for 1% AEP.
Increases level Fairybower Road by 0.23 m, 0.19 m for 2% AEP, 1% AEP.
Reduces level Depot Hill by 0.04 m, 0.06 m for 2%, 1% AEP
TOS: 7.67 d, 9.63 d for 2%, 1% AEP

Option	Description	Table J-11
B7	Combine B6 + B4	

Cost: $13.0 million
Reduction MAD: $1.28 m p.a
Flood Free at 2% AEP
NPV: $24.3 m ($17.9 m)
BCR: 1.87 (1.38)
Impact on levels:
Increases flood level u/s of crossing by 0.01 m for 2% AEP, 0.27 m for 1% AEP.
Increases flood level Airport by 0 for 2% AEP, 0.09 m for 1% AEP.
Increases flood level Fairybower by 0, 0.16 m for 2%, 1% AEP.
TOS: 0 for 2% AEP, 8.d for 1% AEP

Option	Description	Table J-10
B6	Increase waterway area by lowering invert by 2 m	

Impact on levels:
Reduces level u/s of crossing by 0.21 m for 2% AEP, 0.22 m for 1% AEP.
Reduces level Fairybower Road by 0.11 m 2% and 1% AEP.
Reduces level Depot Hill by 0.03 m, 0.05 m for 2%, 1% AEP
TOS: 10.1 d, 11.4 d for 2%, 1% AEP

Options shaded thus are carried forward for further consideration.

Note: NPV at 5% (7%) **Summary of Flood Mitigation Options - Yeppen Crossing**

Figure 3-5

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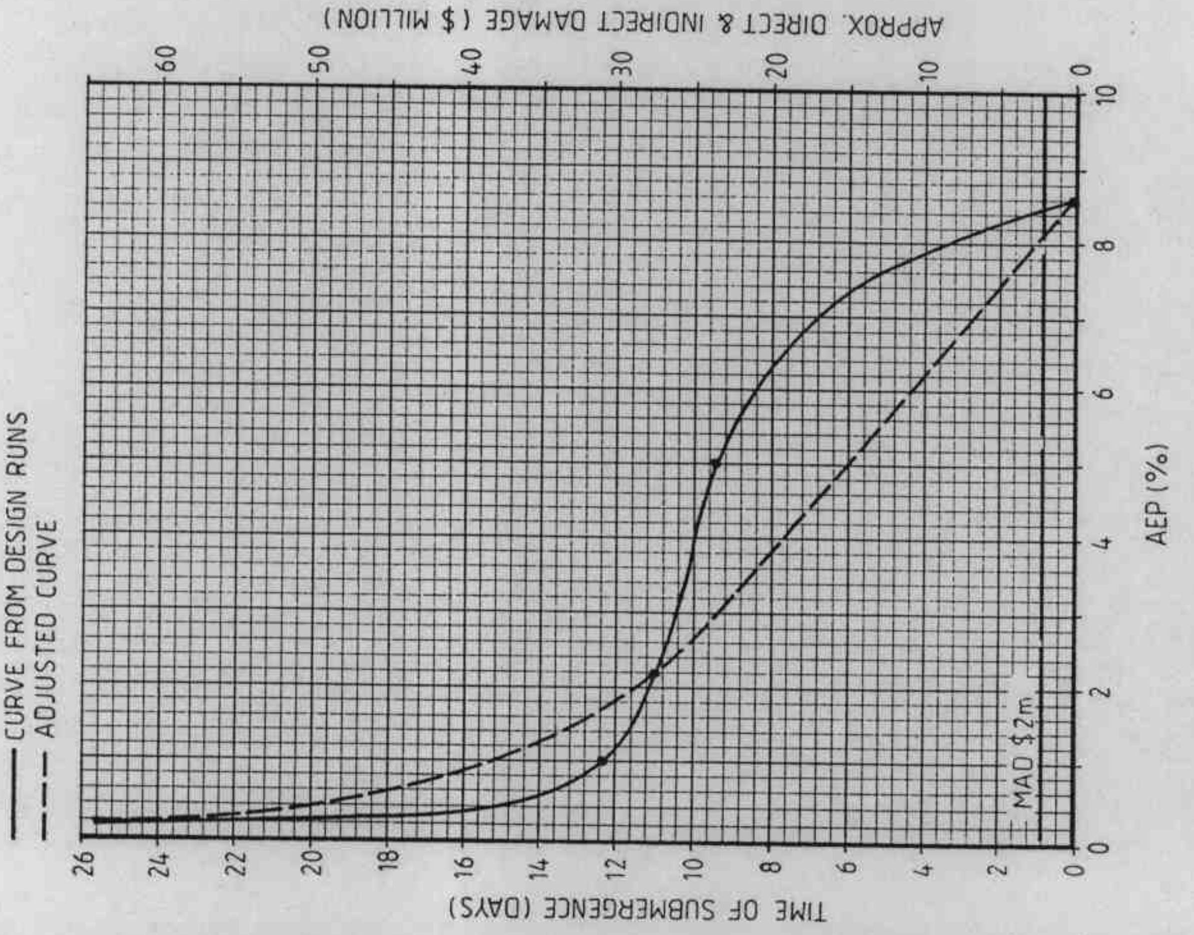
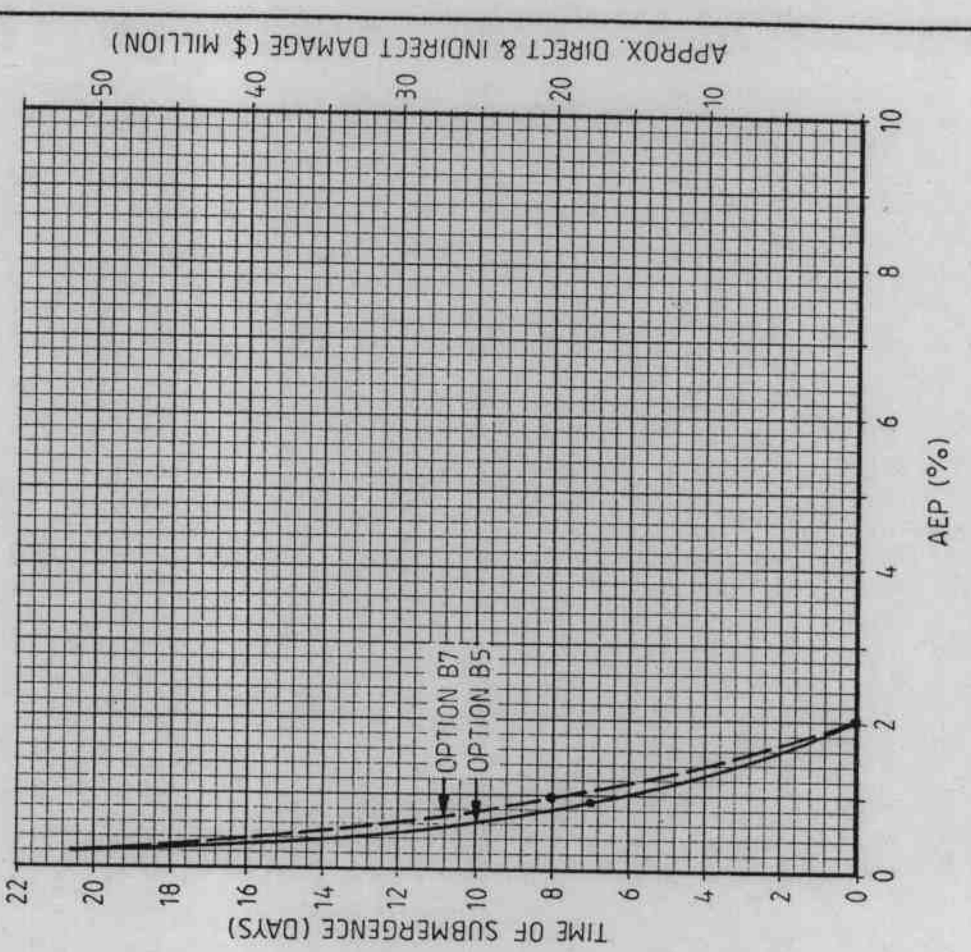


TIME OF SUBMERGENCE & DAMAGE COST vs
PROBABILITY FOR EXISTING YEPPEN CROSSING



TIME OF SUBMERGENCE & DAMAGE COST vs
PROBABILITY FOR UPGRADED YEPPEN CROSSING

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ROCKHAMPTON FLOOD MANAGEMENT STUDY
TIME OF SUBMERGENCE AND DAMAGE PROBABILITY CURVES - YEPPEN CROSSING

FIGURE 3-6

BREAKOUT CONTROL

Option	Description	Table J-14
D1	**Raise Breakout Level at Pink Lily by 1.0 m**	

Impact on Flood Levels:
Increases level at Yaamba by 0.03 m 2% AEP, 0.01 m 1% AEP
Increases level at barrage 0.32 m for 2% AEP, 0.12 m for 1% AEP
Increases level at City Flood Gauge 0.13 m, 0.06 m for 2%, 1 % AEP.
Reduces levels Nine Mine Road by 0.21 m, 0.1 m for 2%, 1% AEP.
Reduces levels Airport by 0.44 m, 0.14 m for 2%, 1% AEP.
Reduces levels Fairybower Road by 0.26 m, 0.12 m for 2%, 1% AEP.
Reduces levels Yeppen Crossing by 0.22 m, 0.09 m for 2%, 1% AEP.
Negligible impact at Depot Hill.
NOT EFFECTIVE

Option	Description	Table J-16
D3	**Raise Breakout Level at Pink Lily by 2.5 m**	

Compromise between D1 and D2 gives signficant reduction in time of submergence at Yeppen to 6.5 d at 2% AEP, 9 days at 1% AEP. Levels at Yeppen reduced by 0.71 m for 2% AEP, 0.49 m for 1% AEP.
But raises levels at Yaamba by 0.07 m, 0.06 m for 2%, 1% AEP, at Barrage by 0.71 m, 0.55 m and at City flood gauge by 0.27 m, 0.19 m.

Option	Description	Table J-15
D2	**Raise breakout level at Pink Lily to prevent overflow in 2% AEP**	

Flood Free at Yeppen 1% AEP

Impact on levels:
Increased level at Yaamba by 0.11 m, 0.15 m for 2%, 1% AEP.
Increased level at Stanwell PS by 1.3 m, 1.66 m for 2%, 1% AEP.
Increased level at Pink Lily by 1.82 m, 2.22 m for 2%, 1% AEP.
Increased level at Barrage by 1.14 m, 2.35 m for 2%, 1% AEP.
Increased level at City Flood Gauge by 0.49 m, 0.51 m for 2%, 1% AEP.
Reduces level at Airport by 1.81, 1.48 m for 2%, 1% AEP.
Reduces level at Yeppen by 1.57 m, 1.27 m for 2%, 1% AEP.
TOS: Yeppen 0, for 1% AEP
GIVES FLOOD FREE CROSSING AT YEPPEN FOR 1% AEP BUT RAISES LEVEL IN RIVER BY UNACCEPTABLE AMOUNT.

Option	Description	Table J-16
D4	**Raise breakout threshold at Gavial Creek to reduce tailwater at Yeppen**	

This was investigated for 2% AEP only.
As a means of reducing levels at Yeppen Crossing this was ineffective, reducing levels by only 0.08 m but raising levels in the river by up to 0.7 m at the Gavial Creek junction.
NOT CONSIDERED FURTHER.

None of the above were considered to warrant further consideration as stand alone measures, but could be useful in conjunction with other measures.

These measures were not costed.

Measures to Vary Breakout Control
Figure 3-7

MISCELLANEOUS OPTIONS

a) PINK LILY - YEPPEN - GAVIAL CREEK FLOODWAY

Option	Description	Table J-19
E1	Pink Lily - Gavial Creek	

Investigated only briefly, the aim of this would be to carry a signficant part of the flood flow to the south of the city. Modelling showed that a channel 1,000 m wide for 3 m depth and 1,000 m wide right overbank section would only be able to carry about 3,500 m^3/s at 2% AEP, and 4,600 m^3/s at 1% AEP. This would cause increased levels in the city reach of the river of up to 0.3 m at 1% AEP. The cost would be prohibitive and environmental impact on the lagoon system would be high. It would however, allow filling of adjacent land for development.
NOT CONSIDERED FURTHER

Option	Description	Table J-17
F3	Remove Old Burnett Hwy bridge and disused rail embankment	

Cost: $0.5 million
Impact on levels:
Reduction u/s of Yeppen crossing - 0.06 m, - 0.07 m for 2%, 1% AEP floods.
NOT WORTHWHILE AS A STAND ALONE MEASURE BUT MAY BE USEFUL IN CONJUNCTION WITH OTHER OPTIONS - SEE FIGURE

Option	Description	Table J-18
F4	F3 + Enlarge Channel d/s Yeppen	

The effectiveness of excavating a channel from the downstream side of the Yeppen Crossing in addition to the works in Option F3 was investigated as a means of reducing tailwater levels and hence increasing the capacity of the existing structures.
This was found to be ineffective.
NOT CONSIDERED FURTHER.

b) OTHER

Option	Description	Table J-20
M1	Commonage Landfill	

The effect of the commonage landfill on flood levels was investigated.

This was found to have an insignifcant impact on flood levels.

Option	Description	Table J-22
M3	Lower Capricorn Highway by 1.0 m	

The impact of lowering those sections of the Capricorn Highway above ground level by 1.0 m was investigated.
This was found to be ineffective.
NOT CONSIDERED FURTHER

Options shaded thus are carried forward for further consideration.

Miscellaneous Flood Mitigation Measures
Figure 3-8

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a) Options with Port Curtis – Depot Hill – CBD Levee

Option C8 — **Description** (B5a + A2) Yeppen upgrade + levee Port Curtis to CBD — Table J-30

Cost: $23.9 m
Reduction MAD: $1.77 m
NPV: $33.6 m ($24.8 m)
BCR: 1.40 (1.04)

Yeppen flood free at 2% AEP
Port Curtis, Depot Hill, Lower CBD flood free to 1% AEP

Impact on levels:
Flood level u/s Yeppen reduced by 0.28 m at 2% AEP and 0.02 lower at 1% AEP. Levels at airport reduced by 0.09 m at 2% AEP, and 0.02 m at 1% AEP.
Levels at Fairybower Road reduced by 0.15 for 2% AEP and by 0.02 m at 1% AEP.
TOS zero 2% AEP, 6.5 days at 1% AEP.
THIS OPTION PROVIDES PROTECTION TO GREATEST FREQUENTLY FLOODED AREA AND RESULTS IN REDUCTION IN LEVEL U/S OF YEPPEN AT 1% AND 2% AEP.

Option C9 — **Description** (C8 + A3 + A5) ie. as C8 + Levee Airport + Splitters Creek — Table J-31

Cost: $28.4 million
Reduction MAD: $1.88 m p.a.
NPV: $35.7 m ($26.3 m)
BCR: 1.26 (0.93)

Yeppen flood free at 2% AEP
Airport, Port Curtis, Depot Hill, CBD flood free to 1% AEP

Impact on levels:
Flood level u/s Yeppen reduced by 0.33 m at 2% AEP compared with existing and 0.09 m at 1% AEP compared to existing. Raises level Nine Mile road by 0.11 m, 0.18 m for 2% AEP, 1% AEP.
Level at Fairybower Road reduced by 0.25 m, 0.17 m for 2%, 1% AEP. Small increase in level in river Pink Lily – Barrage, reduction d/s of barrage of 0.02 m, 0.05 m at City gauge for 2%, 1% AEP.
TOS zero 2% AEP, 3.5 days at 1% AEP.
THIS OPTION PROVIDES PROTECTION TO AIRPORT AND SPLITTERS CREEK AREA AS WELL AS PORT CURTIS - CBD - SMALL REDUCTION IN LEVEL U/S YEPPEN AT 2% AEP, BUT INCREASE OF 0.17 M AT 1% AEP.

Option C10 — **Description** as C9 + Raise Pink Lily Breakout by 1.25 m — Table J-32

Yeppen flood free at 2% AEP
Airport, Port Curtis, Depot Hill, CBD flood free to 1% AEP.

Impact on levels:
Breakout level at Pink Lily offsets increased levels at Fairybower, Yeppen caused by levee, at expense of raising river levels by 0.02 m, 0.01 m at Yaamba, 0.44 m, 0.27 m near WTW (2%, 1% AEP) 0.28, 0.16 at Barrage, 0.12 m, 0.02 m at City Flood Gauge.
Reduction at Fairybower Road 0.44 m, 0.18 m, and at Yeppen Crossing 0.40 m, 0.01 m for 2%, 1% AEP.
TOS zero 2% AEP, 5.4 days 1% AEP.

Option B5a — **Description** as B5 + removal of bridge on Old Burnett Highway & disused railway embankment — Table J-9a

Yeppen TOS 0 for 2% AEP, 3.0 d 1% AEP

Impact on levels:
Flood level u/s Yeppen Crossing reduced by 0.34 m for 2% AEP, and by 0.16 m for 1 % AEP. Respective reductions Fairybower Road 0.18 m, 0.09 m.
Levels in City reach reduced by 0.04, 0.05 m for 2%, 1% AEP.
Levels at Airport reduced by 0.11 m, 0.05 m.
Levels at Depot Hill reduced by 0.13 m, 0.25 m for 2% , 1% AEP.
Yeppen Crossing flood free at 2% AEP
TOS: 3.0 d for 1% AEP

Preferred Options

Note: NPV at 5% (7%)

NOTE: The above options relate to increasing waterway area of Yeppen bridges by doubling the bridge length.

b) Options with Depot Hill – CBD Levee only (ie. excluding Port Curtis)

Option C6 — **Description** (B5 + A1) Yeppen upgrade + levee Depot Hill to CBD — Table J-28

Cost: $22.2 million
Reduction MAD: $1.62 m
NPV: $30.8 m ($22.7 m)
BCR: 1.38 (1.02)

Yeppen flood free at 2% AEP
Depot Hill, Lower CBD flood free to 1% AEP

Impact on levels:
Improved flood level impacts compared to C8 at expense of not protecting Port Curtis area. Levels u/s Yeppen reduced by 0.16 m for 2% AEP compared to existing, and by 0.07 m at 1% AEP.
Reduction at Airport 0.05 m at 2% AEP, 0.01 m at 1% AEP.
Reduction at Fairybower Road 0.09 m 2% AEP, 0.03 m at 1% AEP. TOS zero 2% AEP, 6.9 days at 1% AEP.
REDUCED IMPACT U/S YEPPEN COMPARED TO OPTION C8, BUT AT EXPENSE OF NOT PROTECTING PORT CURTIS.

Option C7 — **Description** (C6 + A3 + A5) – as C3 + levee Airport & Splitters Creek — Table J-29

Cost: $26.7 million
Reduction MAD: $1.73 m
NPVE: $32.9 m ($24.2 m)
BCR: 1.23 (0.91)

Yeppen flood free at 2% AEP
Airport, Depot Hill, Lower CBD flood free to 1% AEP

Impact on levels:
U/s Yeppen flood level reduced by 0.20 m at 2% AEP 0.13 m at 1% AEP and at Fairybower Road by 0.18 m, 0.19 m respectively.
Raises level Nine Mile Road by 0.11 m, 0.19 m for 2%, 1% AEP.
TOS zero at 2% AEP, 6.4 days at 1% AEP.
BENEFICIAL U/S YEPPEN AND FAIRYBOWER AREA, PREVENTS FLOODING 1% AEP TO AIRPORT, DEPOT HILL, CBD, BUT NOT PORT CURTIS.

'Do Nothing' Case

Cost:
MAD of $5.2 million per year

Closure Frequency Yeppen Crossing 8.5% (12 year ARI)

Flooding Frequency
Port Curtis – Depot Hill – Lower CBD
8% – 10 % AEP (10 – 12 year ARI)

0080G803.B02

Summary of Combined Flood Mitigation Options
Figure 3-9

SECTION 4

4. FLOOD MAPPING

4.1 INTRODUCTION

Flood maps showing the extent of flooding for a range of flood levels, on a probability basis, are a necessary pre-requisite to the development of planning controls for flood liable land. The delineation of the flood liable area into high and low hazard categories is a further aid in the development of planning controls.

This section describes and presents the flood maps produced as part of the current study, together with the limitations to their accuracy, and makes recommendations in regard to planning controls which the Local Authorities should incorporate into their Floodplain Management Policies.

As previously indicated in the Phase 1 Report, flood maps are only being prepared for that part of the flood liable area for which contour information is available. Whilst this covers most of Rockhampton City, there are still some areas for which flood maps have not been prepared. Similarly, no flood maps have been prepared for the adjacent areas of Fitzroy Shire and Livingstone Shire. In those areas, flood levels predicted from the hydraulic model provide the only information available for planning purposes.

4.2 FLOOD MAPS

A flood map has been prepared at a scale of 1:10,000 to show the extent of inundation in 2%, 1% and 0.5 % AEP floods. A reduced version of the flood map is presented as Figure 4-1 in this report.

The flood maps, however, are of a low level of accuracy because of significant anomalies between the observed flood inundation extent in 1991 (2% AEP) and that determined by available contour information.

This problem was identified in Phase 1. Considerable effort has been put into eradicating these problems in Phase 2, by means of preparing new contour mapping for the flood liable part of the urban area of Rockhampton to the north of the river which was the worst in this regard. However, as this was done from existing photography, with the exception of the Queen Elizabeth Drive/Kershaw Gardens area for which new photography was obtained, there are still a few anomalies of a minor nature in this area. Funds were insufficient to prepare new mapping for the south side but unfortunately significant anomalies have been found in this region as well.

Because of these limitations, the following approach has been adopted, namely:

- the 1991 inundation line which has been determined from information provided by the City Council, and from aerial photography, has been adopted as the 2% AEP extent (the Yaamba discharge as revised by WRC is now equal to 2% AEP). This is regarded as the most accurate information available;

- the difference between levels in 2%, 1% and 0.5 % AEP floods was obtained from the relevant design run of the hydraulic model and used to prepare the extent of flooding for 1% and 0.5% AEP events.

Whilst the 2% AEP flood line is believed to be reasonably accurate, the 1% and 0.5% AEP events are regarded as indicative only. They should not be used, therefore, to determine whether or not a particular block is flood liable at 1% AEP. The flood maps have been marked to clearly display this limitation.

The accuracy of the maps is also dependant upon the accuracy of the modelled flood levels. This is expected to be of the order of ±0.2 m at the 1% AEP level. The extent of such variation on the ground can be substantial where gradients are low.

If the works recommended in this study are constructed, the necessity for improving the accuracy of the flood maps will diminish, because most of the areas where there is some doubt as to the extent of flooding will be protected by the various mitigation measures.

However, should the recommended works not proceed, it is recommended that the accuracy of the flood maps be improved by actually establishing on the ground, the 1% AEP levels determined from the hydraulic model. This should be done prior to final adoption of the flood maps.

Prior to adoption of the maps for planning purposes, we recommend that the maps be issued in draft form for public comment. This will enable any minor anomalies in relation the 1991 flood extent to be identified and resolved. The maps could then be adopted as interim documents until they can be refined as discussed above.

4.3 FLOOD HAZARD

4.3.1 Delineation of Floodways and Flood Storage Areas

Floodways are those parts of the floodplain in which a significant volume of water flows during floods. They are areas which, even if only partially blocked, would cause a significant redistribution of flood flows which may in turn adversely affect other areas. They are often but not necessarily, the areas with deeper flow or areas where high velocities occur.

In the current context, these include the bulk of the floodplain flow area between Pink Lily and Midgee and much of the area adjacent to the river in which overbank flows occur.

In contrast to floodways, flood storage areas are those parts of the floodplain important for the temporary storage of floodwaters during the passage of a flood. If the capacity of flood storage areas is substantially reduced, for example, by the construction of levees, or by landfills, flood levels in nearby areas may rise, and peak discharge downstream may be increased. Substantial reduction in flood storage area capacity may also cause a significant re-distribution of flood flows. These effects are generally adverse, although those from individual developments may in themselves be small.

The other category of flood liable land is that regarded as 'flood fringe' which is the remaining area of land affected by flooding once floodways and flood storage areas have been identified. Development in flood fringe areas does not have a significant effect on the pattern of flood flows and levels.

The delineation of flood liable land into the above categories is based on hydraulic and hazard considerations. Initially the following criteria are generally adopted:

- Floodways – those areas in which the product of depth (metres) and flow velocity (m/s) is greater than 1.0; and/or those areas where removal of the cross-section area available to pass flood flows will cause adjacent upstream flood levels to increase by more than 0.1 m during major flooding.

- Flood storages – those areas outside floodways which, if completely filled, would cause peak flood levels to increase anywhere (upstream or downstream) by more than 0.1 m and/or cause the peak discharge anywhere downstream to increase by more than 10%.

- Flood fringe – flood liable areas not within the above categories.

4.3.2 Flood Hazard

The term flood hazard is used here as a measure of the overall adverse effects of flooding. It incorporates the concepts of threat to 'life and limb'; the difficulties and danger of evacuating people and their property during the flood; the potential for damage to structures and contents; social disruption; damage to public property and loss of commercial production.

Flood hazard varies across flood liable areas as a result of the mix of property types, the depth and velocity of flood waters, and the variation of problems of evacuation. Towards the edge of floodwaters, depths are generally shallow and velocities low, and such areas have a relatively low hazard. By way of contrast, areas in main floodways where floodwaters may be deep and velocities high are generally high hazard areas.

This hazard categorisation, is based initially on the hydraulic considerations discussed above. Other aspects such as difficulties of evacuation are then considered as to whether the initial classification should be varied. Low hazard rating is usually applied where persons and their possessions could be evacuated by trucks; where able-bodied adults would have little difficulty wading; where potential damage is low; where risk to 'life and limb' is low.

In high hazard areas, on the other hand, floodwaters make evacuation difficult and dangerous; structural damage to buildings may occur; there may be danger to 'life and limb'; social disruption and financial losses could be high.

The NSW Floodplain Development Manual (NSW Government 1986) lists six categories of flood liable land namely; floodway, flood storage and flood fringe each with low and high hazard rating and provides guidelines as to appropriate levels of development in each category.

The flood hazard map, Figure 4–2, has been prepared using this approach. In many areas the category is clear, but in some areas either the boundary between various categories, or the category to be assigned is not well defined. As for the map of flood extent, this map should be regarded as a draft at this stage.

4.4 DEVELOPMENT GUIDELINES

The development guidelines given in the NSW Floodplain Development Manual (1986) are reproduced in Appendix K.

It is recommended that the provisions of these guidelines be adopted in regard to planning and the consideration of development applications in the flood liable areas of Rockhampton City, and where applicable to the adjacent flood liable parts of the Fitzroy Shire and Livingstone Shire.

The primary recommendation in this regard is that no new residential, commercial or industrial development be permitted in designated floodways. As stated in 4.3.1 floodways have been defined on the basis of the product of depth and velocity in a 1% AEP flood being equal to or greater than 1 m^2/s. Problems of access have also been considered in this regard.

It should be noted that flood damages in floodway areas such as the Pink Lily area, have resulted from development which has been allowed to take place in such areas. Unless development control is amended, further development in these areas would result in increased levels of flood damages in future floods. No such increase has been accounted for in the modelling of flood damages. The general principle to be followed in regard to floodways should be to work towards a reduction of intensity of development, so that, at the least, flood damage levels are not made any worse.

The primary requirement in regard to new residential dwellings where they are permitted is for a minimum habitable flood level of 0.5 m above the design flood (1% AEP). It is recommended that this level be adopted. Access routes to any new development should be trafficable in the design flood.

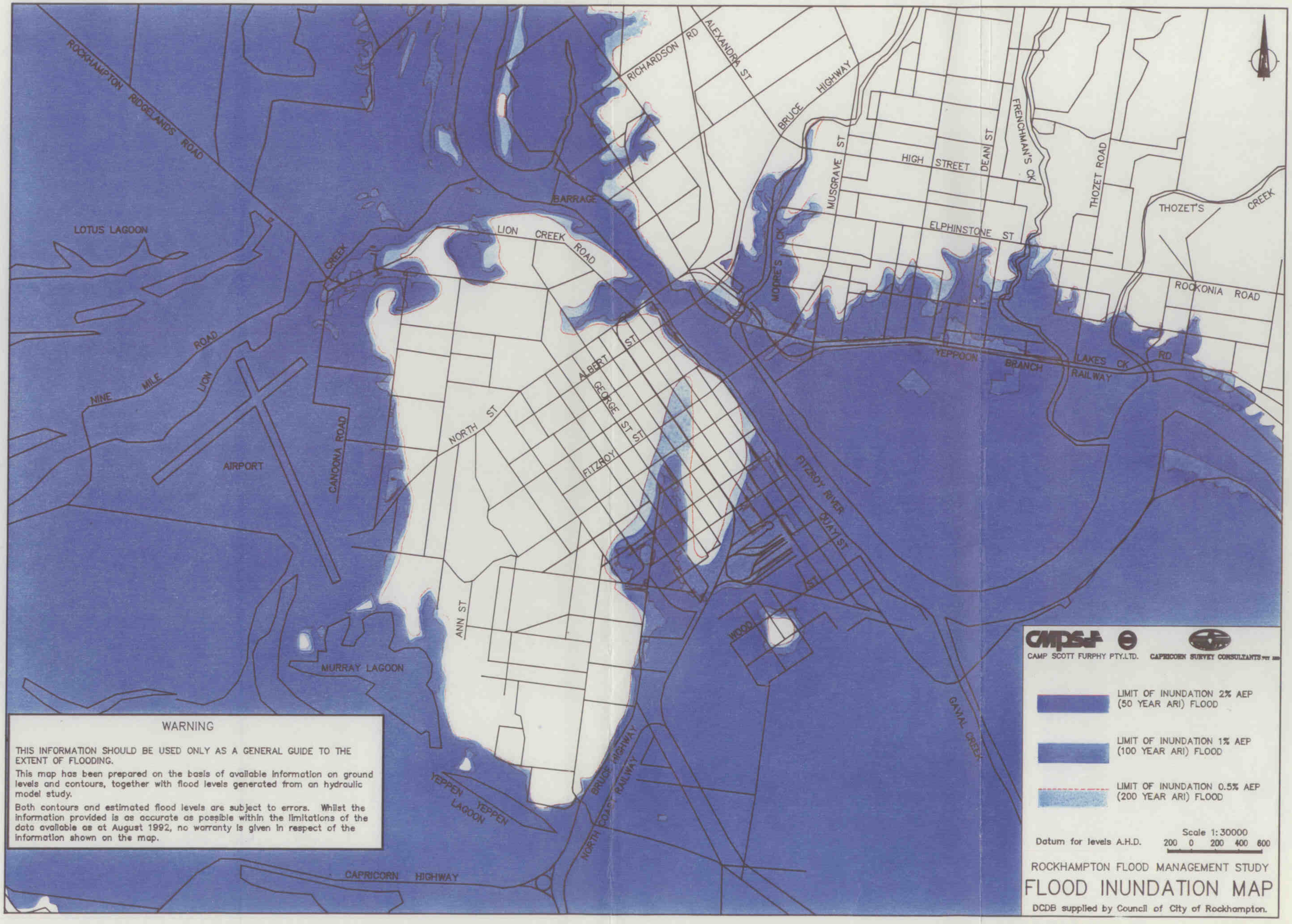
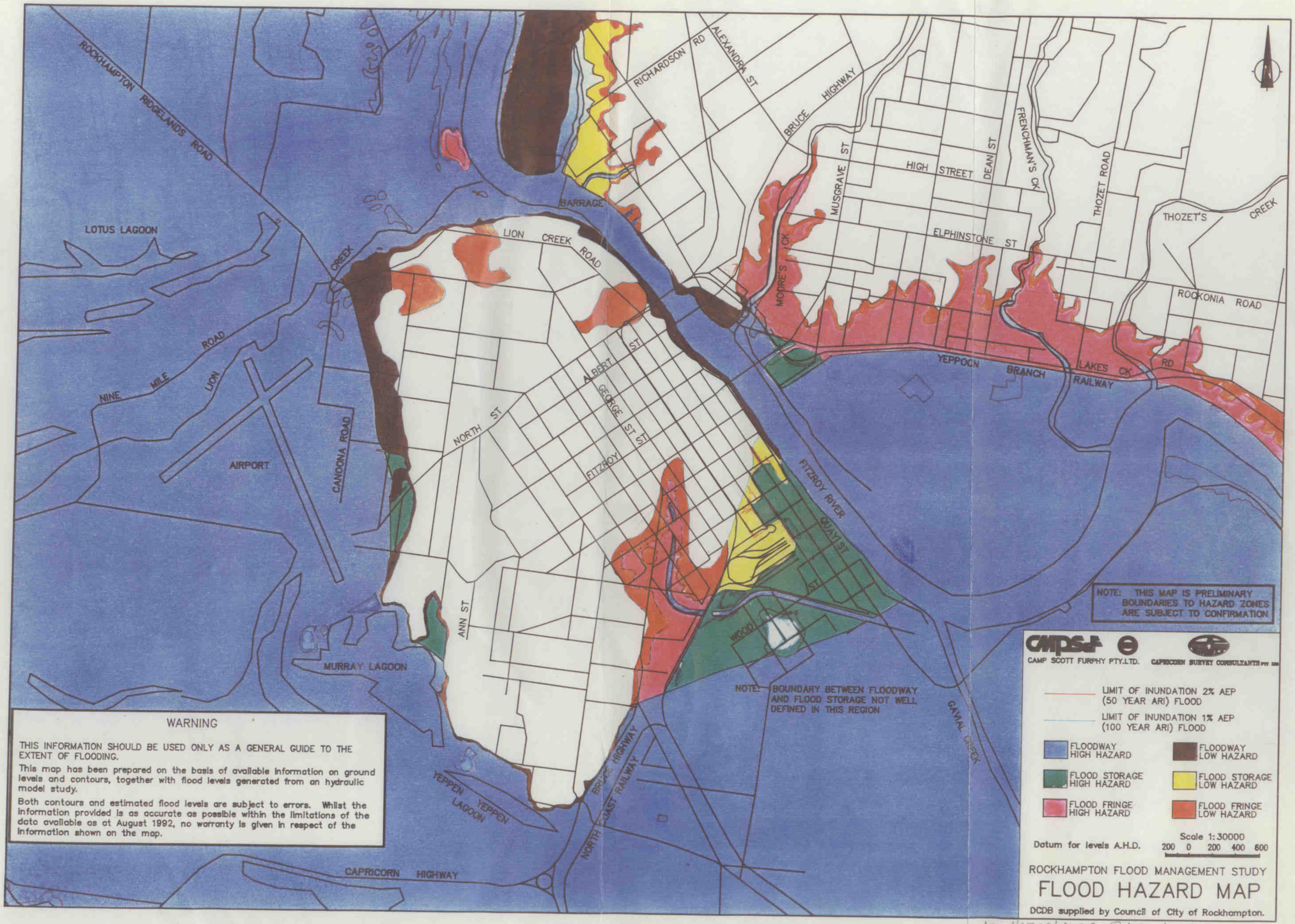

ROCKHAMPTON RIDGELANDS ROAD
LOTUS LAGOON
CREEK
NINE MILE ROAD
LION
AIRPORT
CANOONA ROAD
BARRAGE
LION CREEK ROAD
RICHARDSON RD
ALEXANDRA ST
BRUCE HIGHWAY
MUSGRAVE ST
HIGH STREET
DEAN ST
FRENCHMAN'S CK
THOZET ROAD
THOZET'S CREEK
ELPHINSTONE ST
MOORE'S CK
ROCKONIA ROAD
YEPPOON BRANCH RAILWAY
LAKES CK RD
NORTH ST
ALBERT ST
GEORGE ST
FITZROY ST
FITZROY RIVER
QUAY ST
WOOD ST
ANN ST
MURRAY LAGOON
YEPPEN YEPPEN LAGOON
BRUCE HIGHWAY
NORTH COAST RAILWAY
CAPRICORN HIGHWAY
GAVIAL CREEK
WARNING
THIS INFORMATION SHOULD BE USED ONLY AS A GENERAL GUIDE TO THE EXTENT OF FLOODING.
This map has been prepared on the basis of available information on ground levels and contours, together with flood levels generated from an hydraulic model study.
Both contours and estimated flood levels are subject to errors. Whilst the information provided is as accurate as possible within the limitations of the data available as at August 1992, no warranty is given in respect of the information shown on the map.
CMPS&F
CAMP SCOTT FURPHY PTY.LTD.
CAPRICORN SURVEY CONSULTANTS
LIMIT OF INUNDATION 2% AEP (50 YEAR ARI) FLOOD
LIMIT OF INUNDATION 1% AEP (100 YEAR ARI) FLOOD
LIMIT OF INUNDATION 0.5% AEP (200 YEAR ARI) FLOOD
Datum for levels A.H.D.
Scale 1:30000
200 0 200 400 600
ROCKHAMPTON FLOOD MANAGEMENT STUDY
FLOOD INUNDATION MAP
DCDB supplied by Council of City of Rockhampton.

FIGURE 4-1



WARNING

THIS INFORMATION SHOULD BE USED ONLY AS A GENERAL GUIDE TO THE EXTENT OF FLOODING.

This map has been prepared on the basis of available information on ground levels and contours, together with flood levels generated from an hydraulic model study.

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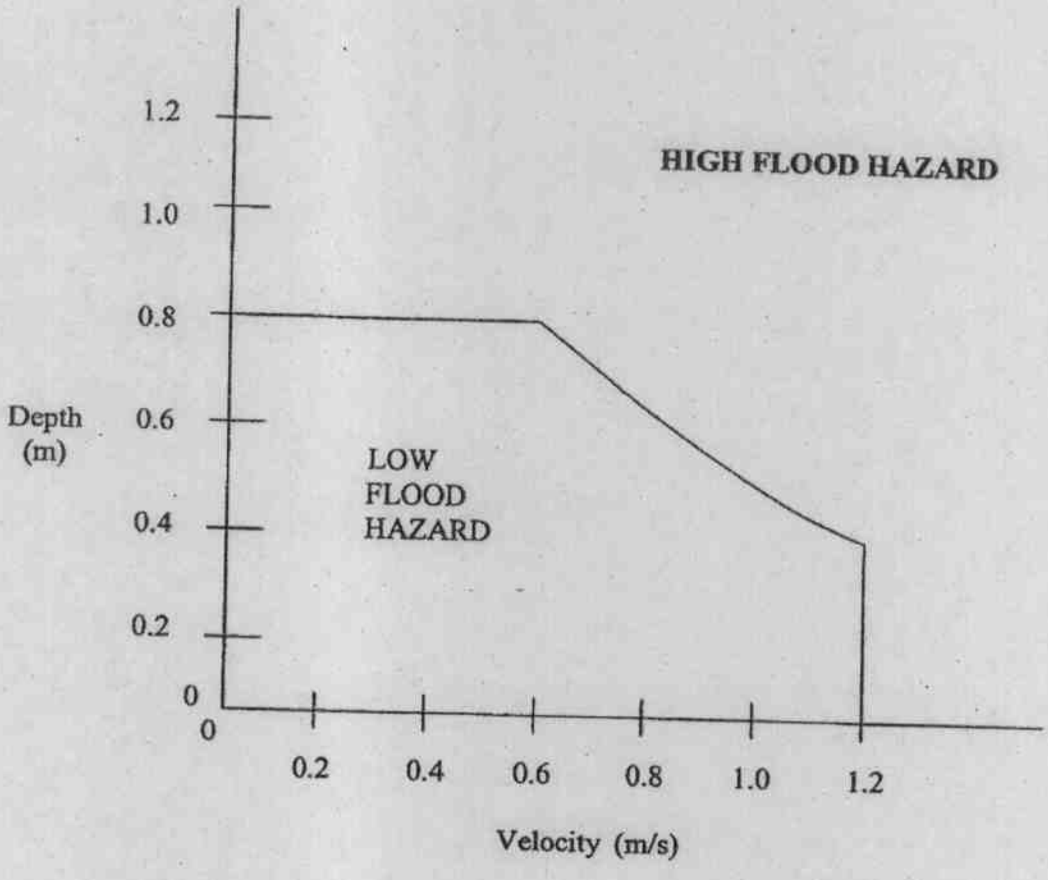
Low Hazard > 0.8m @ 1.2m/sec.

FIGURE 4-2

Determination of Flood Hazard Categories

(Applicable to a Q100 ARI flood event)



SECTION 5

5. SUMMARY OF RECOMMENDATIONS

5.1 GENERAL

This section summarises the recommendations made in Phases 1 and 2 of the study for improvement of flood management in Rockhampton. The latter incorporates both the structural flood mitigation options discussed above and the non-structural measures recommended in the Phase 1 Report. The consideration of a combination of such measures is in line with the guidelines given for works to be funded under the Federal Water Resources Assistance Program (FWRAP).

This section also briefly addresses possible funding for these works.

It is recommended that those items of the relatively low cost non-structural measures identified as being of first priority be implemented by Rockhampton City Council, Fitzroy Shire Council and Livingstone Shire Council as appropriate, as soon as possible and prior to awaiting the outcome of any funding application, as although these do not give any physical protection against flooding they will ensure that damages are minimised should another major flood occur prior to the construction of the flood mitigation works.

The total estimated cost of the recommended works is about $35 million of which the non-structural works comprise only $0.3 million. The structural works have been broken down into four priority levels. These priorities may be used in phasing the works according to budget constraints.

5.2 SUMMARY OF RECOMMENDATIONS

5.2.1 Non-Structural Measures

The following is a summary of the non-structural measures which were recommended in the Phase 1 Report, which should be consulted for further detail. These are measures recommended for immediate implementation.

a) Formulation and adoption of a floodplain management policy to be formalised by the adoption of appropriate planning instruments. The flood inundation map and flood hazard map produced as part of this study provide the basis for these controls. For the preparation of the floodplain management policy allow $30,000;

b) Upgrading of the flood warning system:
   - installation of telephone telemetry at the Rockhampton flood warning gauge, cost $20,000;
   - installation of a new river level station with telephone telemetry at Pink Lily to provide information regarding floodplain flows, cost $15,000;
   - installation of rainfall recorders at existing river level stations equipped with telephone telemetry (Riverslea, The Gap, Neerkol Creek) cost 3 @ $1,000 ie. $3,000;

- installation of a water level and a rainfall recorder with telephone telemetry in the Alligator Creek catchment, cost $16,000.

Annual maintenance and operation on the above, allow $20,000. It is possible that some of the cost of the above upgrading could be met by the Bureau of Meteorology.

c) Installation of permanent flood markers throughout the urban area and the floodplain to show the 1991 flood level, allow $25,000 (1,000 markers @ $25);

d) Establishment of a recorded message telephone service for flood warnings at the Local Emergency Operations Centre (LEOC). The cost of this is difficult to determine without investigation of the PABX system currently installed but may be of the order of $20,000 – $30,000 if queuing facilities have to be provided compared to just a few thousand for the recorded message facility itself. An indicative cost of $30,000 has been included herein. The warning messages should be frequently updated and should contain information on levels at Tartrus, Riverslea, The Gap, Yaamba, and the new floodway reference gauge as well as Rockhampton. The message should repeat so that information missed on the first pass may be reheard. Multiple telephone lines should be provided;

e) Instigation of a programme of raising community flood awareness and preparedness, by means of:

   i) making the flood maps available for sale to the public;
   ii) preparation of a flood awareness pamphlet;
   iii) inclusion of a flood awareness page in the local telephone directory;
   iv) encouragement to local business operators to prepare flood action plans;
   v) establishment of the LEOC as a single point of contact;
   vi) raising media awareness of their role in flood warning dissemination;
   vii) improvement to road closure reporting (RACQ/LEOC).

   The costs of preparation of the community flood awareness material would be approximately $25,000.

The total cost of these measures outlined above would be $143,000 plus annual maintenance costs of about $30,000. The improvement in flood warnings and the way in which the community can relate the warnings to their own circumstances would be expected to result in a substantial reduction in direct flood damages. If this results in only a 10% reduction in actual damage, this is worth of the order of $200,000 p.a. (mean annual direct damage approximately $2 m) so this expenditure is clearly worthwhile. These measures are further summarised in Table 5-1.

The Phase 1 report also contained a recommendation in regard to a pilot study of the feasibility of flood proofing commercial premises in Rockhampton. This may be supported by local business groups. The aim of such a study would be to look at the practicalities of flood proofing a small number of existing buildings of a range of types and industry types, together with a detailed examination of the damage reduction such measures would produce in order to enable evaluation of the cost effectiveness of this approach. There is very little detailed information in this regard, hence support for a pilot study would be very worthwhile. The cost of this study would be about $40,000. Business operators should also be encouraged to prepare flood contingency plans, or flood action plans, so that they can minimise damage and disruption caused by any future floods.

Whilst the responsibility for flood forecasting lies with the Bureau of Meteorology, there would be merit in establishing a flood forecasting model for the lower Fitzroy River which would be operated locally. This could be developed from the MIKE II model set up for the current study and would allow the operators of the LEOC to have improved information of a more detailed nature than that provided by the bureau. The cost of developing this model would be about $50,000 plus $30,000 for computer software and hardware. It is recommended that consideration be given to developing this system.

5.2.2 Structural Measures

The following structural measures are recommended. The priority of each component is shown. Should the works be constructed in a phased manner, the order of construction should follow the priority rating. A phased approach will allow the highest level of benefits to be achieved early during the works programme. Works of priority 1 to 4 may be envisaged, for example, as a 4 year work programme. This timing must be determined by the Local Authorities in regard to their budgets and also in regard to possible funding.

As discussed in Section 3, the recommended works comprise the following, a summary of which is given in Table 5-1.

a) Priority 1

- upgrading Yeppen crossing by raising embankment height to bridge height for the full width of the floodplain crossing, together with doubling bridge waterway area by increasing bridge length to about 840 m from the existing 420 m.

  The cost of these works is estimated to be $16.5 million.

  This would raise the flood immunity of the southern road and rail approaches to Rockhampton to 2% AEP, with significantly reduced closure times for more extreme floods.

  The damage reduction has been estimated to be about $1.3 million per annum on a long term average basis, with a benefit-cost ratio of 1.5 for these alternatives assuming a 5% discount rate (1.1 for 7%).

  The Department of Transport's position regarding funding this upgrade is given in Appendix L.

- Construction of a levee to protect the lower Dawson Road/Gladstone Road, Port Curtis, Depot Hill areas and the lower part of the CBD. This would extend from Blackall Street to the north of Yeppen Yeppen Lagoon along Jellicoe Street to Port Curtis, across to Depot Hill, to near the Gavial Creek junction with the Fitzroy River, then along Quay Street to Derby Street. If protection were provided to 1% AEP, the cost would be about $6.9 million, with a BCR of 1.35 at 5% (1.0 at 7%). Raising the level of protection to 0.5% AEP would increase the total cost to $8.35 million with a BCR of 1.43 (1.05), and to 0.2% AEP the cost would be $10.1 million with a BCR of 1.45 (1.06). However, as the 0.5% AEP level of protection would have some negative impact on flood levels in the floodplain, it is recommended that the levee be designed to give protection to the 1% AEP flood, with controlled flooding for more extreme events.

- Removal of the bridge/causeway along the section the Old Burnett Highway between Jellicoe Street and the new Bruce Highway, together with removal of the disused railway embankment adjacent to the Old Bruce Highway between Port Curtis and Roopes Bridge at a cost of approximately $0.5 million.

The latter measure is necessary to help offset the adverse impact of flood levels which would otherwise be caused by the proposed levee. The measures outlined above should be regarded as a total package and should preferably be constructed concurrently. If phasing is necessary due to financial constraints, the Yeppen Crossing upgrade should be regarded as being the highest priority.

The combination of the above will have no adverse impact on flood levels in occupied areas at 2% AEP and at 1% AEP compared to existing conditions.

This scheme will have a very high positive social impact. It will allow complete protection from flooding (apart from local runoff) for the areas within the levee up to at least 1% AEP with consequent reduction of the trauma effects of isolation during flooding. The community awareness programme should include discussion of the limits of flood protection but this should be balanced against the benefits. This scheme will also allow development within the protected areas, although sufficient area should be retained for storage of local flood waters, and should result in a rise in property values. It is considered that there is little or no negative environmental impact of these works.

The proposed upgrading of Yeppen Crossing will also have a substantial positive social impact as it will significantly reduce the frequency of closure of the southern road and rail approach to Rockhampton, with consequent reduction in disruption to social and business activity. The proposed scheme is considered to have negligible environmental impacts.

b) **Priority 2**

At a slightly lower priority, construction of a levee to protect Rockhampton Airport, and the adjacent residential areas is recommended. One end of this levee would be near the Barrage. It would then pass close to Lion Creek, around the airport and then to higher ground near Denham Street (Extended). This would cause a significant increase in flood levels in that part of the floodplain between Pink Lily and Lion Creek. This is a maximum of 0.3 m at 2% AEP and 0.6 m at 1% AEP. A small number of houses along Nine Mile Road may need to be raised to compensate for this effect. The increase in level along the Rockhampton–Ridgelands Road is 0.05 m at 2% AEP and 0.12 m at 1% AEP, which is regarded as being acceptable.

Social impact will be positive overall with the protection of the airport and the adjacent residential areas, although it will be negative for the small number of houses where flood levels are adversely effected. However, as these houses are within a current floodway, their lot is not significantly worsened. The cost of raising these houses should be considered as part of the scheme. Land use controls should be utilised to prevent additional development in the floodway as discussed in section 4.

The cost of this levee system, with protection to 1% AEP is estimated to be $4.3 million rising to $5.6 million at 0.5% AEP. The direct benefits are relatively low with BCR for 1% AEP at only about 0.35 at 5% (0.26 at 7%). However, a significant intangible benefit would be obtained from keeping the airport open to traffic during such circumstances by allowing emergency and flood relief services to operate far more effectively than is currently possible. There is a substantial cost penalty of raising protection to 0.2% AEP as the total cost would then be $7.4 million. Because of negative impacts on flood levels for protection against floods greater than 1% AEP, the 1% AEP level is recommended.

c) **Priority 3**

- The construction of a levee to prevent direct overflow from the Fitzroy River into Splitters Creek. The levee would extend from near Limestone Creek to near Splitters Creek. The purpose of this levee is to prevent the direct overflow and hence reduce flood hazard. The cost would be $0.14 million. The social impact would be positive as a result of reduction in flood hazard.

- The stabilisation of control levels at Pink Lily was investigated as described in Sections 2.7 and 3.4, whereupon it was determined that no alteration to the control levels could be justified. However, as discussed in the Phase 1 Report, section 13.4.3, it would be advisable to stabilise the outer bank of the Pink Lily meander so that the breakout threshold level does not reduce with time. It is not possible to estimate direct flood mitigation benefits from this measure. Hence these stabilisation works are included as a low priority item at an estimated cost of $900,000 on the basis of battering the existing bank, placement of a rockfill toe and revegetation of the banks.

d) **Priority 4**

Priority 4 items are those which should be undertaken in the longer term. These are measures to reduce flooding in flood fringe areas and comprise the fitting of flood gates on creeks and flood valves on stormwater drainage outlets to prevent backwater flooding. These will not prevent flooding in the relevant drainage areas when local flooding is coincident with river flooding, but will prevent river floodwater backing up these systems to between 2% AEP and 1% AEP level at which adjacent bank sections would start to overtop. Further long term measures to improve the immunity would be to raise the north bank levels by means of low levees. These have not been costed at this time.

These items have not been costed in detail, a sum of $500,000 has been allowed for floodgates for each major creek on the north bank ie. Splitters Creek, Moores Creek, Frenchmans Creek and Thozet Creek, and a further $500,000 in total for similar control on piped stormwater drainage outlets.

In addition to the capital costs outlined above, the Local Authorities and Government Departments responsible for the above works would need to meet maintenance costs. These costs are difficult to establish and a nominal cost of $100,000 per annum for Priority 1 works, $50,000 per annum for Priority 2 and Priority 3 works and $100,000 for Priority 4 works should be allowed. These would be substantially reduced if there is spare capacity in the existing maintenance labour force.

5.2.3 Design Stage

If the Local Authorities resolve to proceed with the measures outlined above, it will be necessary for detailed engineering studies to be carried out prior to construction. These costs have been allowed for in the estimates given, the approximate allowance of 10% of capital costs includes detail design, preparation of contract documentation and construction supervision. These costs may be included in the FWRAP funding application. Final alignment of the proposed levees would be determined in the design stage.

In the case of the non-structural works, these can proceed without further engineering input, except for determining the PABX requirements for the installation of the recorded telephone service.

5.2.4 Other Issues Requiring Action

This paragraph lists other issues raised in this report which require further investigation or action for their resolution. Due to budgetary and time constraints it was not possible to include the following in Phase 2, but all of the items listed warrant further study.

- Estimation of probable maximum flood;
- Scrubby Creek Diversion;

TABLE 5-1

Summary of Proposed Works Programme

PRIORITY 1 MEASURES	
NON-STRUCTURAL	
• Floodplain Management Policy	$30,000
• Upgrading of flood warning system	$53,000
• Installation of Flood Markers	$25,000
• Recorded message service	$30,000
• Community awareness programme	$25,000
SUB-TOTAL	$163,000
CAPITAL WORKS	
• Upgrade Yeppen Crossing to increase embankment height to that of the bridges, plus increase waterway area by increasing bridging length to 840 m (BCR 1.5)	$16.5 m
• Construction of levee from Blackall Street to Quay Street protecting Lower Dawson Road, Port Curtis, Depot Hill and the lower CBD (BCR 1.25)	$6.9 m
• Removal of disused railway embankment adjacent to Old Bruce Highway (material may be used in levee works) Demolition and removal of bridge/causeway on Old Burnett Highway	$0.5 m
SUB-TOTAL	$23.9 m
TOTAL PRIORITY 1	$24.063 m

PRIORITY 2 MEASURES	
NON STRUCTURAL	
• Development of Flood Forecasting model	$80,000
• Commercial Flood Proofing Pilot Study	$40,000
SUB-TOTAL	$120,000
CAPITAL WORKS	
• Construction of levee to protect airport extending from Savage Street to Denham Street Extd (BCR 0.45)	$4.3 m
TOTAL PRIORITY 2	$4.42 m

PRIORITY 3 MEASURES	
• Construction of levee to prevent overflow from River to Splitters Creek (BCR approximately 0.7)	$0.14 m
• Bank stabilisation works at Pink Lily	$0.9 m
TOTAL PRIORITY 3	$1.04 m

PRIORITY 4 MEASURES	
• Flood gates on Splitters Creek, Moores Creek, Frenchmans Creek and Thozet Creek	$2.0 m
• Flood valves on stormwater drainage outfalls	$0.5 m
TOTAL PRIORITY 4	$2.5 m

OVERALL TOTAL RECOMMENDED WORKS	**$32.023 m**
Note: BCRs at 5% discount rate.	

- Development of a geographic information system for counter disaster planning and operation;
- Detailed investigation of erosion and siltation in the lower Fitzroy River;
- Investigation of leachate from operational and closed landfills in the Fitzroy River floodplain and subsequent remediation if warranted.

5.3 FUNDING OF WORKS

In recent years flood mitigation works have been eligible for funding under the Federal Water Resources Assistance Program (FWRAP). From 1993/94 flood mitigation works and measures are expected to be eligible for funding under the National Landcare Program (NLP) which will integrate FWRAP and other programs.

In Queensland, it is the responsibility of the relevant Local Authority to apply for funding under the program to the State Government in the first instance through the Water Resources Commission, customarily by December each year. The State Government will integrate and prioritise applications and submit those programs it supports as part of a Partnership Agreement with the Commonwealth Government. Notification of successful applications is made following the Federal Budget each August.

Under this scheme funding is as follows:

- Federal Government 40%
- State Government 40%
- Local Government 20%

It should be noted that NLP funds are limited, and that submissions for funding are considered on their merits and cost-effectiveness and also on the basis of priority with other state projects as this program is placing increasing emphasis on well integrated land and water resource management projects and non-structural flood mitigation measures. However, due to the magnitude of flood damages in the recent flood and the isolation of a city of the size of Rockhampton which results from such floods, it may be expected that the chances of a support by the State would be high, but would of course depend on the State's priorities in the particular year. Criteria for Commonwealth support under the new NLP may evolve from those under FWRAP with increasing emphasis on Commonwealth funds being used to stimulate micro-economic reform or improvements in procedures and perceptions of natural resource management issues. Consequently, successful projects would need to engender new local and regional financing schemes and viable, beneficial, community-based flood management strategies.

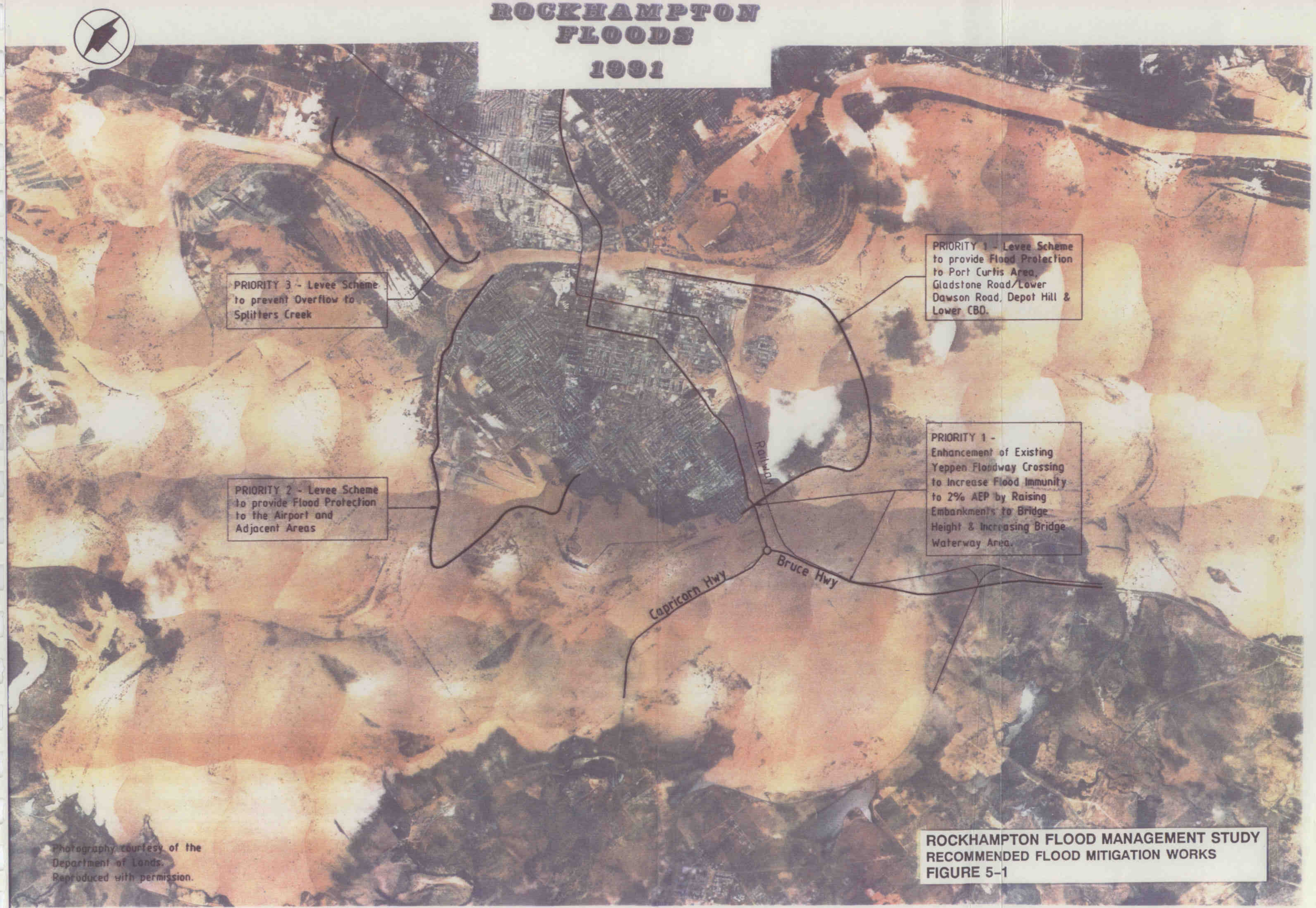
Thus if funding were obtained under NLP for all the first priority works, the Local Authority Contribution would be expected to be $4.8 million. However, if only the levee works and the non-structural works were funded in this way, for example, this would reduce to $1.5 million.

Whilst the proposed upgrading at Yeppen principally relates to flood mitigation in respect of reduction of indirect damages, it would be expected that part of the upgrading costs would be met by the Department of Transport. This would be the subject of negotiation between relevant Government Departments and Local Authorities. The Department of Transport's positioin in this regard is the subject of the statement given in Appendix L.

In regard to the airport levee, Rockhampton Airport is owned by Rockhampton City Council but is administered as a separate entity. Thus the costs attributable to protection of the airport will need to be separated from those for protection of the adjacent residential areas, so that the costs of protection the airport are not a direct cost on ratepayers. As for the Yeppen crossing, the distribution of costs will need to be negotiated should the scheme proceed.

Also the Bureau of Meteorology may contribute to funding of the flood warning system upgrade. Local business groups may be willing to fund the proposed flood proofing pilot study.

The priorities listed above should be followed in developing a phased programme of works to match Local Authority and funding agency budgets.



ROCKHAMPTON FLOOD MANAGEMENT STUDY
RECOMMENDED FLOOD MITIGATION WORKS
FIGURE 5-1

SECTION 6

6. REFERENCES

CAMP SCOTT FURPHY (1992), 'Rockhampton Flood Management Study – Phase 1 Report', prepared for the Water Resources Commission.

CAMERON McNAMARA (1981) 'Fitzroy River Erosion in the Pink Lily Area', prepared for the Queensland Water Resources Commission.

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