Johnstone River Flood Study
Final Report

Volume 1 of 2

Prepared For:	Johnstone Shire River Improvement Trust
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	Synopsis:	This document details the Johnstone River Flood and Floodplain Management Study. On the basis of this Study, the Floodplain Management Steering Committee recommends the inclusion of some of the options presented here for inclusion into the Floodplain Management Plan. The Plan exists as a separate document.

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FORWARD

The Queensland Department of Emergency Services is administrating the Queensland studies under the Federal Department of Transport and Regional Services' "Natural Disaster Risk Management Studies Program." The aim of the program is to identify, analyse and evaluate the risks from natural disasters and to identify risk management measures to reduce the risk to life and property.

Flooding was identified as a major risk on the floodplains of the North and South Johnstone Rivers funding for the study was obtained through this program to develop a Floodplain Management Plan. The Natural Disaster Risk Management Studies Program contributed 2/3 of the funding (1/3 from both Federal and State Governments) and 1/3 of the funding for the study was provided by the Johnstone Shire Council through the Johnstone Shire River Improvement Trust. During the study, the Queensland Department of Main Roads contributed funds in recognition of the future value of the flood model.

The publication "Floodplain Management in Australia – Best Practice Principles and Guidelines" (CSIRO, 2000) provides the framework for the development and implementation of a Floodplain Management Plan. The process outlined in CSIRO (2000) is described below.

Floodplain Management Process

Stage		Description
1	Flood Study	The nature and extent of the flood problem are determined.
2	Floodplain Management Study	Management options for the floodplain are investigated in respect of both existing and proposed developments. These options are evaluated based on the impact on flood risk, while considering social, ecological and economic factors.
3	Floodplain Management Plan	Following acceptance of Stage 2 recommendations, the preferred management options are documented in a plan.
4	Implementation of the Plan	Involves formal adoption by Council of the floodplain risk management plan and a process of implementation for the selected flood, response and property modification options.

This document comprises stage 1 and 2 of the process for the Johnstone Rivers. It defines the existing flooding problem and assesses a range of measures and their ability to reduce the impact of flooding in the Johnstone River area by controlling the flood risk and reducing flood damages. The impact on flooding of a number of past flooplain works are aslo assessed. Stage 3 is a separate document (WBM, 2003), and it summarises the preferred management measures identified in this report.

WBM Oceanics Australia was commissioned by the Johnstone Shire River Improvement Trust (JSRIT) to carry out this study.

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GLOSSARY

annual exceedance probability (AEP)	The chance of a flood of a given size (or larger) occurring in any one year, usually expressed as a percentage. For example, if a peak flood discharge of 500 m^3/s has an AEP of 5%, it means that there is a 5% chance (i.e. a 1 in 20 chance) of a peak discharge of 500 m^3/s (or larger) occurring in any one year. (see also average recurrence interval)
Australian Height Datum (AHD)	National survey datum corresponding approximately to mean sea level.
average annual damage (AAD)	Depending on its size (or severity), each flood will cause a different amount of flood damage. The average annual damage is the average damage in dollars per year that would occur in a designated area (e.g. the Innisfail area) from flooding over a very long period of time. In many years there may be no flood damage, in some years there will be minor damage (caused by small, relatively frequent floods) and, in a few years, there will be major flood damage (caused by large, rare flood events). Estimation of the average annual damage provides a basis for comparing the effectiveness of different floodplain management measures (i.e. the reduction in the annual average damage).
average recurrence interval (ARI)	The long-term average number of years between the occurrence of a flood as big as (or larger than) the selected event. For example, floods with a discharge as great as (or greater than) the 20yr ARI design flood will occur on average once every 20 years. ARI is another way of expressing the likelihood of occurrence of a flood event. (see also annual exceedance probability)
cadastral data	Property boundary data
catchment	The catchment at a particular point is the area of land that drains to that point.
design floor level	The minimum (lowest) floor level specified for a building.
design flood	A hypothetical flood representing a specific likelihood of occurrence (for example the 100 year or 1% probability flood). The design flood may comprise two or more single source dominated floods.
development	Existing or proposed works that may or may not impact upon flooding. Typical works are filling of land, and the construction of roads, floodways and buildings.
Discharge	The rate of flow of water measured in terms of volume over time (i.e. the amount of water moving past a point). Discharge and flow are interchangeable.
DEM/DTM	Digital Elevation Model or Digital Terrain Model - a three-dimensional model of the ground surface.
effective warning time	The available time that a community has from receiving a flood warning to when the flood reaches them.
flood	Relatively high river or creek flows, which overtop the natural or artificial banks, and inundate floodplains and/or coastal inundation resulting from super elevated sea levels and/or waves overtopping coastline defences.

flood awareness	An appreciation of the likely threats and consequences of flooding and an understanding of any flood warning and evacuation procedures. Communities with a high degree of flood awareness respond to flood warnings promptly and efficiently, greatly reducing the potential for damage and loss of life and limb. Communities with a low degree of flood awareness may not fully appreciate the importance of flood warnings and flood preparedness and consequently suffer greater personal and economic losses.
flood damage	The tangible and intangible costs of flooding.
flood behaviour	The pattern / characteristics / nature of a flood.
flood frequency analysis	An analysis of historical flood records to determine estimates of design flood flows.
flood fringe	Land that may be affected by flooding but is not designated as floodway or flood storage.
flood hazard	The potential risk to life and limb and potential damage to property resulting from flooding. The degree of flood hazard varies with circumstances across the full range of floods.
flood level	The height or elevation of floodwaters relative to a datum (typically the Australian Height Datum). Also referred to as "stage".
flood liable land	see flood prone land
floodplain	Land adjacent to a river or creek that is periodically inundated due to floods. The floodplain includes all land that is susceptible to inundation by the probable maximum flood (PMF) event.
floodplain management	The co-ordinated management of activities that occur on the floodplain.
floodplain management measures	A range of techniques that are aimed at reducing the impact of flooding. This can involve reduction of: flood damages, disruption and psychological trauma.
floodplain management plan	A document outlining a range of actions aimed at improving floodplain management. The plan is the principal means of managing the risks associated with the use of the floodplain. A floodplain risk management plan should be developed in accordance with the principles and guidelines contained in the CSIRO (2000). The plan will usually contain both written and diagrammatic information describing how particular areas of the floodplain are to be used and managed to achieve defined objectives.
floodplain management scheme	A floodplain management scheme comprises a combination of floodplain management measures. In general, one scheme is selected by the floodplain management committee and is incorporated into the plan.
flood planning levels (FPL)	Flood planning levels selected for planning purposes are derived from a combination of the adopted flood level plus freeboard, as determined in floodplain management studies and incorporated in floodplain risk management plans. Selection should be based on an understanding of the full range of flood behaviour and the associated flood risk. It should also take into account the social, economic and ecological consequences associated with floods of different severities. Different FPLs may be appropriate for different categories of landuse and for different flood plans. The concept of FPLs supersedes the "standard flood event". As FPLs do not necessarily extend to the limits of flood prone land, floodplain risk management plans may apply to flood prone land beyond that defined by the FPLs.

flood prone land	Land susceptible to inundation by the probable maximum flood (PMF) event. Under the merit policy, the flood prone definition should not be seen as necessarily precluding development. Floodplain Management Plans should encompass all flood prone land (i.e. the entire floodplain)
flood proofing	Measures taken to improve or modify the design, construction and alteration of buildings to minimise or eliminate flood damages and threats to life and limb.
flood source	The source of the floodwaters. In this study, the Johnstone River catchment is the primary source of floodwaters.
flood storages	Floodplain areas that are important for the temporary storage of floodwaters during a flood.
floodway	A flow path (sometimes artificial) that carries significant volumes of floodwaters during a flood.
freeboard	A factor of safety usually expressed as a height above the adopted flood level thus determing the flood planning level. Freeboard tends to compensate for factors such as wave action, localised hydraulic effects and uncertainties in the design flood levels.
historical flood	A flood that has actually occurred.
hydraulic	The term given to the study of water flow in rivers, estuaries and coastal systems.
hydrograph	A graph showing how a river or creek's discharge changes with time.
hydrology	The term given to the study of the rainfall-runoff process in catchments.
peak flood level, flow or velocity	The maximum flood level, flow or velocity occurring during a flood event.
photogrammetry	The technology used to obtain reliable measurements, maps, digital elevation models, and other GIS data primarily from aerial photography.
probable maximum flood (PMF)	An extreme flood deemed to be the maximum flood likely to occur.
probability	A statistical measure of the likely frequency or occurrence of flooding.
runoff	The amount of rainfall from a catchment that actually ends up as flowing water in the river or creek.
stage	See flood level.
stage hydrograph	A graph of water level over time.
TUFLOW	Fully two-dimensional unsteady flow hydraulic modelling software
URBS	Hydrological computer model software
velocity	The speed at which the floodwaters are moving. Typically, modelled velocities in a river or creek are quoted as the depth and width averaged velocity, i.e. the average velocity across the whole river or creek section.
water level	See flood level.

LIST OF ABBREVIATIONS

1D / 2D/ 3D	One dimensional / Two dimensional / Three dimensional
AEP	Annual Exceedance Probability
AHD	Australian Height Datum
ARI	Average Recurrence Interval
AR&R	Australian Rainfall and Runoff
BoM	Bureau of Meteorology
CBD	Central business district
CDC	Counter Disaster Committee
cm	centimetre
cumecs	cubic metres per second
DA	Development Application
DCP	Development Control Plan
DEM/DTM	Digital Elevation Model /Digital Terrain Model
DMR	Queensland Department of Main Roads
DNRM	Queensland Department of Natural Resources & Mines
DoT	Queensland Department of Transport
EIS	Environmental Impact Study
EPA	Queensland Environmental Protection Agency
ERA	Environmentally Referable Activity
FPL	Flood Planning Level
JSC	Johnstone Shire Council
JSRIT	Johnstone Shire River Improvement Trust
GIS	Geographic Information System
km	kilometre
m	metre
m^3/s	cubic metres per second
m AHD	Elevation in metres relative to the Australian Height Datum
PMF	Probable Maximum Flood
SES	QLD State Emergency Services

1 INTRODUCTION

1.1 Background

The Johnstone River system comprises the North Johnstone River and the South Johnstone River with their confluence being at the town of Innisfail. From the confluence, the river flows about 5 km to the ocean. The rivers have a combined catchment of about 1600 km^2 with the North Johnstone being the larger of the two with a catchment of about 1030 km^2. The locality of the river system is shown in Figure 1-1 (refer Volume 2 for all figures).

The headwaters of the rivers are in the high rainfall area of the Cardwell Ranges. The rivers flow from the range down through gorges to the lower fertile floodplains that are predominantly utilised for agricultural purposes including sugarcane and banana farming. There are a number of townships on the floodplains including the major centre of Innisfail and the smaller townships of South Johnstone, Mourilyan, Wangan and Mundoo.

There is a history of severe flooding on the floodplain with considerable damage to property, agriculture and public infrastructure. Innisfail is most affected being at the confluence of the rivers and with development on flood prone land. Flooding in and around Innisfail town occurs initially through backup of Saltwater and Sweeneys Creeks and then through overtopping of the banks around Innisfail and further to the north in larger floods. The construction of floodgates on Sweeneys and Saltwater Creeks has helped to reduce the frequency of flooding in Innisfail, although the floodgates are overtopped in larger floods.

The suburbs of Webb, East Innisfail and South Innisfail are affected by overtopping of the river banks and by back up from the Johnstone River through the Ninds Creek catchment. Parts of Innisfail Estate are affected in larger floods through overtopping of the river bank. Mourilyan is affected in larger floods when the South Johnstone River overtops its banks. These floodwaters pass through Mourilyan and into the Ninds Creek catchment before rejoining the Johnstone River at the confluence with Ninds Creek.

Consideration of options to reduce flooding impacts, and planning for future development requires an understanding of the flood behaviour. To develop a greater understanding of flooding, hydrological and hydraulic flood models were developed and calibrated to historical floods. These models were then used to simulate a range of design floods that were the benchmark for assessing both past and future works.

Once flood behaviour is understood, a strategic approach to controlling development on flood prone land, assessing the advantages and disadvantages of flood mitigation options, flood proofing properties and buildings, educating and safeguarding communities and protecting the natural environment can be carried out with confidence. This Study provides such assessments, and actions arising from the Study recommendations will be used to formulate a Floodplain Management Plan.

There have been a number of developments on the floodplain over the last 30 years that have raised concerns within the community as to their impact on flood behaviour. These include a levee on the northern bank of the river downstream of Innisfail known in the community as Carello's levee, filling of the town swamp and construction of floodgates to protect the town. The flood model was used to quantify these impacts.

1.2 Study Area

The study area is shown in Figure 1-1 and extends from about Upper Daradgee on the North Johnstone River and the town of South Johnstone on the South Johnstone River through to the ocean. Key features referred to throughout the report are shown in Figure 1-2.

1.3 Previous Studies

The most significant previous study was a floodplain management study by Cameron McNamara (Cameron McNamara, 1985). The study undertook hydraulic modelling using the one-dimensional software CELLS. The study recommended the construction of a major levee and pump system for Innisfail. However, the levee system was not implemented, probably because of concerns over the limited economic analysis undertaken in the study.

Fielding and Orpin (2000) is a study of the effects of Carello's levee on upstream flooding. The study was a desktop review and as such states "*the absolute result of the simplistic modelling analysis presented in this study is inconclusive*" but goes on to suggest that the potential for impacts should be recognised and investigated in any future flood studies.

1.4 Objectives

The key objectives of the study were as follows:

1. develop a state-of-the-art computer model of the Johnstone Rivers System within the study area to define the nature and extent of the flood hazard;
2. model the effects of existing developments and existing flood mitigation measures to determine their impact on flooding, including community concerns raised during the resident survey and community open sessions;
3. propose, assess and recommend possible flood mitigation measures with consideration given to social, ecological and economic factors;
4. prepare a report detailing the development of the model, the assessment of the effect of existing development and flood mitigation measures, addressing community concerns and detailing proposed flood mitigation measures;
5. prepare a Floodplain Management Plan.

1.5 Steering Committee

The Johnstone River Shire Improvement Trust formed a Steering Committee to oversee the Floodplain Management Study and to ensure that issues important to the Johnstone Rivers community have been addressed. The Steering Committee was comprised of:

- the River Trust

- community representatives;
- Chamber of Commerce representatives.

The Steering Committee had an important role in advising the Council and River Trust on recommendations for implementation in the Floodplain Management Plan. The mix of representatives provided a forum for the distillation and resolution of differing viewpoints before the plan is submitted to Council.

A series of discussion papers were presented and reviewed during the course of the study. These discussion papers represent the collective ideas of the consultant (WBM Oceanics Australia), the Steering Committee and the community.

Throughout the study, regular meetings were held in Innisfail with the Committee at which the findings documented in the papers were discussed and issues were resolved. The discussion papers outlined the essential information about each floodplain management measure and, based on this information, the Committee decided whether individual measures were to be incorporated into a Floodplain Management Scheme.

2 STUDY APPROACH

There were seven key stages in the study.

1. Data Collection
2. Flood Model Development and Calibration
3. Design Flood Analysis and Existing Flood Damage Assessment
4. Historical Floodplain Works Assessment
5. Floodplain Management Measures Assessment
6. Recommendation of Management Measures
7. Reporting

The remainder of Section 2 outlines the adopted approach for each of these stages. A detailed description of each of the stages is given in subsequent sections of the report.

2.1 Data Collection

2.1.1 Topographic and GIS Data Sets

Topographic survey was required for the development of a Digital Elevation Model (DEM), which is a three-dimensional model of the ground surface. The DEM forms the basis of the hydraulic model. Data for the DEM was obtained from photogrammetry, ground survey and bathymetric survey. Extensive GIS data sets were obtained including cadastral data, aerial photography and historical flood levels.

2.1.2 Resident Survey

An extensive survey of residents within the study area was conducted to gather historical data from those who have experienced Johnstone River floods and to identify local concerns within the region. The local knowledge of the flooding in both Innisfail and its surrounds was found to be invaluable. A number of flood heights were identified and surveyed.

2.1.3 Site Inspections

Consultants from WBM Oceanics Australia conducted numerous site inspections in the study area over the course of the study. They also attended regular meetings with the Steering Committee and used these opportunities to further investigate areas that became the focus of attention.

2.1.4 Historical Flood Data

Historical flood data was required for the calibration of both the hydrologic and hydraulic models. Data was collected from the Bureau of Meteorology (BoM), the Department of Natural Resources and Mines (DNRM), the Johnstone Shire Council and residents during the resident survey. Historical ocean tide data was required and was obtained from the Environmental Protection Agency (EPA).

2.1.5 Community Consultation

Two community open sessions were held at the completion of the preliminary calibration of the flood model. This gave the community an opportunity to meet the study team and to see and review the flood model to ensure that it replicated the historical floods as they recalled them and provide the study team with ideas for flood mitigation measures. This proved to be an invaluable process as the community identified some areas in the model that needed improvement. Additional field survey was obtained and the model modified with the result that the model replicated the historical flooding patterns as indicated by the community during the open sessions. It is believed the open sessions also gave the community confidence in the study approach.

A further two open sessions were held at the completion of the draft report to present the findings and recommendations of the study. This gave the community an opportunity to

2.2 Flood Model Development and Calibration.

The flood model comprises a hydrological model and a hydraulic model.

The hydrologic model determines the runoff that occurs following a particular rainfall event. The primary output from the hydrologic model is hydrographs at varying locations along the waterways to describe the quantity, rate and timing of stream flow that results from rainfall events. These hydrographs then become a key input into the hydraulic model.

The hydraulic model simulates the movement of flood waters through waterway reaches, storage elements, and hydraulic structures. The hydraulic model calculates flood levels and flow patterns and also models the complex effects of backwater, overtopping of embankments, waterway confluences, bridge constrictions and other hydraulic structure behaviour.

The Bureau of Meteorology (BoM) has established and calibrated an URBS hydrologic model of the Johnstone River catchment. This model was reviewed and adopted for the study. Some minor modifications to the model sub-catchments were done by WBM to match the locations of the hydraulic model boundaries.

The complicated nature of the floodplain flow patterns and importance of obtaining community confidence in the process required that state-of-the-art modelling techniques be adopted. For these reasons, TUFLOW, a fully 2D dynamic hydraulic modelling system was adopted. In total, the hydraulic model covers approximately 125 km^2 of the rivers and floodplain.

Information on the topography and characteristics of the catchments, rivers, creeks and floodplains were built into the models. The hydrologic and hydraulic models were calibrated/verified against the February 1999, March 1997 and March 1967 historical flood events to demonstrate the validity of the models. The calibration and verification illustrated the models' abilities to reproduce historic flood patterns collected during data collation and community consultation. Comparisons with comments on flooding patterns received during the historic flood information survey were also consistent with the hydraulic model's performance.

2.3 Design Flood Analysis and Existing Flood Damage Assessment

Design floods are hypothetical floods used for planning and floodplain management investigations. A design flood is defined by its probability of occurrence. It represents a flood which has a particular probability of occurring in any one year. For example, the 1% Annual Exceedence Probability (AEP) or 1 in 100 Average Recurrence Interval (ARI) flood is a best estimate of a flood which has 1 chance in 100 of occurring in any one year. It should be noted that planning for the 1 in 100 year ARI flood does not guarantee protection for the next 100 years.

Design flood levels in the Johnstone River system were assessed using an iterative approach comparing results of hydrologic/hydraulic model results for given rainfall events against results of frequency analyses on the North Johnstone at Goondi and Tung-Oil and on the South Johnstone at Central Mill. The final Johnstone River system design flows were determined by critical assessment of the results in consultation with the Steering Committee. Design flood levels, flows and velocities were determined for 100, 50, 20, 10, 5 and 2 year ARI floods.

The design floods were used to make an assessment of the financial losses to residential and commercial properties. These financial losses were then used as a basis to do an economic assessment of potential floodplain management measures. Historical damage to public infrastructure was documented where information was available.

2.4 Historical Floodplain Works Assessment

The hydraulic model was used to assess the impact of the following works on flood levels:

- Carello's levee;
- Floodgates on Sweeneys & Saltwater Creeks;
- Filling of the Town Swamp;
- Raising of the Bruce Highway at Mourilyan.

2.5 Floodplain Management Measures Assessment

Both structural and non-structural floodplain management measures were assessed. Structural measures are referred to as flood modification measures in this document and are those measures that alter flood behaviour, eg, levees and diversion channels. Non-structural measures are classified as property modification or response modification measures in this document. Property modification measures include development controls, voluntary house purchase and voluntary house raising. Response modification measures include flood warning, emergency planning and community awarenss. Input for potential structural measures was sought from both the community and the Steering Committee. The assessments considered economic benefits, intangible benefits and environmental considerations.

2.6 Recommendation of Management Measures

Each of the measures assessed by WBM was reviewed by the Steering Committee and viable measures selected for recommendation in the Floodplain Management Plan. The Steering Committee considered factors such as flood benefits and disbenefits, economic return, capital costs, intangible benefits and environmental impacts.

2.7 Reporting

Discussion papers detailing the methodology and findings at milestones throughout the course of the study were issued to each member of the Steering Committee and presented at Steering Committee meetings. The discussion papers were an important tool in ensuring that each member of the committee understood the study methodology and assumptions before "signing-off" on the study progress at each stage.

A Draft Flood Study Report was presented to the Steering Committee and the key findings of the report to the Community at Community Open Sessions. Feedback was obtained and incorporated into this Final Flood Study Report. The key findings and recommendations of the Steering Committee are summarised in a separate document called a Floodplain Management Plan. The Floodplain Management Plan is a simple, easy to read and view document, using maps and plans to illustrate the preferred scheme. It is designed for the lay-person and as a central tool for Council's day-to-day floodplain management activities.

3 DATA COLLECTION

3.1 Site Inspection and Resident Survey

Over the period 24-28 August 2000, a detailed site inspection of the Lower Johnstone River system and floodplain was undertaken by Cathie Barton (WBM) and Alan Dunne (JSRIT) to identify key features that significantly influence flood behaviour and to witness flood level data/marks to aid the data validation process. The site inspection included a four-day resident survey involving:

- personal interviews with 24 residents;
- discussions with Johnstone Shire Council and Department of Natural Resources and Mines officers and the Johnstone Shire Council Mayor;
- interviews with the Cairns Post and the Advocate;
- touring of the Johnstone River region by both land and water.

Further input was obtained from the steering committee and the community at the committee meeting and community open sessions on 27th March 2002 following the presentation of the preliminary calibration of the flood model. The resident survey and community open sessions had several benefits including:

- input to the study team through local knowledge and personal experiences in flooding patterns and obtaining historical flood height data;
- developing a better understanding of flood behaviour in the area and an appreciation of flooding issues, thereby improving the quality of hydraulic modelling;
- developing a relationship with the community so that they obtain ownership of the study.

The regional coverage of the residents interviewed is given in Table 3-1 and the historical coverage in Table 3-2. Important information was obtained from the residents relating to the flooding characteristics of the region, in particular:

- several new flood marks were discovered and surveyed;
- flow paths were confirmed by long-term residents;
- potential flow paths in extreme events were identified and in some cases confirmed by residents;
- differences in flow paths caused by differences in the size of flood events in the North Johnstone and South Johnstone Rivers were identified – these are particularly relevant around West Innisfail;
- historical changes to topography (for example, raising of roads, building of floodgates) were discussed.

Issues concerning residents were also discussed and summary is provided in Table 3-3.

Table 3-1 Regional Coverage of Resident Survey

Region	North Johnstone – Sundown, Daradgee, Belvedere	West Innisfail and the surrounding township	Downstream of Junction of Rivers	South Johnstone - Mourilyan to Wangan
Number of Residents Able to Provide Knowledge Of Region	5	6	8	5

Table 3-2 Historical Coverage of Resident Survey

Flood Events	1870 to 1960	1961 to 1970	1971 to 1980	1981 to 1990	1990 to 2000
Number of Residents Able to Provide Knowledge Of Event	6	15	9	15	24

Table 3-3 Historical Coverage of Resident Survey

Region	Issues
General	About 90% of people believed that river sedimentation needs to be addressed.
	About 90% of people expressed dissatisfaction at the flood warning provided. Most stated that it was non-existent
	About 70% of people stated that Innisfail was located on a floodplain, will always flood, and therefore if people built in low areas with low floor levels they should expect to be flooded.
	The people who have had reason to request a minimum habitable floor level were dissatisfied with the information provided, or not provided, by the Council
North Johnstone – Sundown, Daradgee, Belvedere	Concerns regarding the condition of Saltwater Creek were raised – it is believed that the creek needs to be cleaned out to remove Pannikin grass and sediment – at the moment the clogged creek causes too much water to be taken by the Town Swamp Drain.
	Some erosion of river banks is being experienced in those properties adjacent to the river.
West Innisfail and the surrounding township	The majority people interviewed with knowledge of Sweeneys Creek believed that the Sweeneys Ck floodgates are not operating properly during flooding. However, others believe that they are.
	Some expressed the view that filling of the Town Swamp has had an influence on flood levels in the town.
Downstream of Junction of Rivers	All people with flood affected property in this region believe that flood events which affect their properties are increasing in frequency. About 60% believe that this increase is due to a combination of the Carello's Levee and siltation of the river. The remaining 40% believe that Carello's Levee has minimal impact, if any, and instead suggested: 1. An increasing number of floodgates upstream in both the North Johnstone and the South Johnstone meant that more water came downstream. 2. Non-functioning floodgates in the Jodrell St area let the floodwater in. 3. Increases in the level of Coquette Point Rd which prevents flood water entering the large storage area in Ninds Creek and Bulguru.
	People living along the bank adjacent to Banana Island are experiencing bank erosion.
	Concerns were raised about changes in the Ninds Creek catchment including devegetation, draining of swamp land and artificial realignment of Ninds Creek.
South Johnstone - Mourilyan to Wangan	The Cellodonis breakout was identified as an important breakout point and the cause of much of the flooding in this area.
	The Celledonis believe that the culverts under the Bruce Highway are not large enough to take water to Ninds Creek and hence they experience flooding in local rainfall events as well as breakout events.
	Some properties on the river are experiencing bank erosion.
	Gracey Creek breakout causes flooding in Mundoo.
Other	Requests by some that any solutions recommended are both environmentally and economically sustainable.

3.2 Topographic Data

Topographic survey is required for the development of a Digital Elevation Model (DEM), which is a three-dimensional model of the ground surface. The DEM forms the basis of the hydraulic model. Data for the DEM was obtained from the following sources:

- 2001 photogrammetric survey obtained for this study using 1:8000 aerial photography;
- bathymetric survey from 1995 and 2001 provided by the Queensland Department of Transport;
- 2002 cross-section survey of the South and North Johnstone Rivers and Bamboo Creek in areas not covered by the bathymetric survey;
- digitised contours from 1985 photogrammetry supplied by the Johnstone Shire Council;
- photogrammetric survey and river cross-sections in the South Johnstone area supplied by the Department of Main Roads.

The coverage of each of these data sources is shown in Figure 3-1. The labels in this figure are coloured coded to match the data source type. For example, the area of the floodplain that was developed using Main Roads photogrammetry is coloured red.

The development of the hydraulic model required ground survey, undertaken in 2002, of the following features:

- road and rail drainage infrastructure data;
- road and rail top of embankment/rail level;
- Carello's levee;
- Floodgate levee heights and pipe details.

3.3 GIS Data Sets

An extensive GIS data set was obtained including:

- digital orthophotos;
- cadastral data;
- historical flood level and extent information.

The digital orthophotos were obtained for this study and were done in conjunction with the photography taken for the photogrammetry, but were taken at a higher flying height to reduce the number of photographs requiring rectification.

3.4 Historical Flood Data

Historical flood data was required for the calibration of both the hydrologic and hydraulic models. Data was collected from the Bureau of Meteorology, the Department of Natural Resources and Mines, the Johnstone Shire Council, the Environmental Protection Agency, and the community during the resident survey. The data types and their sources are summarised in Table 3-4. The peak floodplain flood level data was subject to a rigorous validation process to ensure its reliability for use in the flood model.

Table 3-4 Historical Flood Data Sources

Data	Source
Rainfall Data • 1999 flood • 1997 flood • 1967 flood (limited data)	URBS model from Bureau of Meteorology
Gauged River Heights • Goondi (112001a) • Tung Oil (112004a) • Central Mill (112101a) • Central Mill (112101b)	Bureau of Meteorology Department of Natural Resources and Mines
Peak Floodplain Flood Levels • 1999 flood • 1997 flood • 1967 flood • Other early significant events	Johnstone Shire Council Community Mr Alan Dunne
Historical Ocean Levels	Environmental Protection Agency
Flood patterns	Community
Flood photos and videos	Community

4 FLOOD MODEL DEVELOPMENT & CALIBRATION

The flood model comprises a hydrological model and a hydraulic model. URBS was the adopted hydrological model and TUFLOW the adopted hydraulic model. This Section documents the development and calibration of the flood model.

4.1 Hydrologic Model

The Bureau of Meteorology (BoM) has established and calibrated an URBS hydrologic model of the Johnstone River catchment. The model comprises 42 sub-areas as shown in Figure 4-1 and has been calibrated to eight floods including the February 1999 event. Full details of the model are in "*Johnstone River Revised URBS Model",* Bureau of Meteorology, Hydrology Section, 1998. The resolution of the discretisation is adequate for the purposes of this study. However, some changes to the catchment structure were undertaken to match the location of the hydraulic model.

The calibration events are given in Table 4-1 along with the BoM classification for each event. The BoM has also verified the model against the 1967 flood. The catchment has an established network of rainfall and river height stations and an ALERT system (installed in September 1989) as shown in Figure 4-1. In flood events since 1989, rainfall data has been available from up to 14 pluviograph stations as well as manual observations. Prior to September 1989, rainfall was based on daily totals and temporal distributions determined from the nearest pluviograph.

Table 4-1 URBS Calibration Events

Event	BoM Classification
April 1982	Moderate
January 1986	Major
March 1990	< Minor
January 1994	Moderate
February 1995	< Minor
March 1996	Moderate
March 1997	Moderate
February 1999	Major

The URBS model uses stream ratings at the locations given in Table 4-2. The ratings were either obtained from the Department of Natural Resources and Mines HYDSYS records or developed by the BoM using flows calculated by URBS and observed flood heights from manual or instrument readings.

For all events except 1982, the BoM obtained reasonable to good matches between the model predictions and the recorded hydrographs. Results from the model for the February 1999, March 1997, January 1994 and March 1967 floods are shown in Figure 4-2 to Figure 4-9. Model and recorded streamflows are provided for each flood at Tung-Oil on the North Johnstone and Central

Mill on the South Johnstone: the only exception is the 1999 flood for which Nerada results are presented rather than Tung-Oil because the Tung-Oil gauge was not operational during this flood. Model and recorded stream heights at Innisfail Wharf are provided in Figure 4-10 to Figure 4-12 for the 1999, 1997 and 1994 floods respectively.

Table 4-2 Stream Gauge Information

Stream	Station	(a) Highest Gauged Level (m)	(b) Highest Recorded Level (m)	Date Opened	Event	(a)/(b) (%)
Nth Johnstone	McAvoy Alert (112908)	Not Gauged	9.80	2000	12/02/1999	-
Nth Johnstone	Tung Oil (112004A)	9.78	10.81	01/10/1966	12/02/1999	78%
Nth Johnstone	Nerada (112905)	Not Gauged	11.35	1989	12/02/1999	-
Nth Johnstone	Innisfail (112900, 112901)	Not Gauged	8.09	1979	30/01/1913	-
Sth Johnstone	Central Mill (112101B)	6.55	10.84	1/10/1974	02/02/1986	34%
Sth Johnstone	Corsis	Not Gauged	8.63	1989	31/01/1994	-

The match between recorded and modelled is reasonable at most of these locations and it was concluded that the URBS model is satisfactory for the purposes of this study, although some adjustment to the catchment discretisation was required to match the hydraulic model boundaries.

4.2 Hydraulic Model

The complicated nature of the floodplain flow patterns and importance of obtaining community confidence in the process required that state-of-the-art modelling techniques be adopted. For these reasons, TUFLOW, a fully two-dimensional (2D) dynamic hydraulic modelling system was adopted. In total, the model covers approximately 125 km^2 of the rivers and floodplain. The DEM and extent of the 2D model is shown in Figure 4-14. During the course of the study, additional photogrammetry data was made available by DMR which allowed the fully two-dimensional model to be extended from its original boundary at about Mourilyan to south of South Johnstone as shown in Figure 4-14. However, the extended region of the model has not been calibrated and results from this part of the model should be used with caution.

The DEM was developed using the software package 12D. The survey data was obtained from a variety of sources as identified in Section 3.2. River and creek cross-sections were surveyed in areas not covered by the bathymetric survey. The maximum spacing between sections was about 500 m. This provided river bed levels at the location of the survey, but did not provide levels between the cross-sections to allow the development of the DEM. The river bed between the cross-sections was developed by WBM by connecting the cross-sections using breakline strings. There were typically five to six breaklines connecting the cross-sections: one running parallel to the toe of each bank; one along the centreline of the river and another two or three lines in between. The end result was a bed between the cross-sections that is in effect a linear interpolation of the surveyed cross-sections. This method of linear interpolation is adequate for the purposes of this flood study, but the resulting DEM should not be used to provide an accurate estimate of river bed levels for other purposes.

The hydraulic model is based on a 20 m square grid. Each square grid element contains information on ground topography sampled from the DEM at 10 m spacing, surface resistance to flow (Manning's n value) and initial water level. Eleven areas of different land-use type, determined from on aerial photography and site inspections, were identified for setting Manning's n values.

The following inflow boundaries were obtained from the URBS hydrological model:

1. North Johnstone River near Upper Daradgee
2. South Johnstone River at the Central Mill Gauge
3. Bamboo Creek – North and South
4. Scheus Creek
5. Cheeki Creek
6. Stewart Creek
7. flows from catchments draining into the floodplain north of the North Johnstone River
8. rainfall on area covered by model

The location of the above boundaries (except for 7 and 8) are shown in Figure 4-14. Boundaries 7 and 8 were internal boundaries rather than external boundaries.

Bridge structures were modelled by using width and height restrictions on 2D elements to represent the flow constriction caused by the bridges, plus the specification of additional losses for the bridge piers and vena-contracta losses if appropriate. Bridge decks were modelled as dynamically nested one-dimensional (1D) broad crested weirs to allow flow over the bridge. Small culverts were modelled as dynamically nested 1D culvert structures and larger culverts using 2D elements similar to bridges.

4.3 Selection of Calibration/Verification Events

The hydraulic model was calibrated/verified using three historical flood events. Selection criteria for calibration events were:

- the amount of good quality historical data available;
- the quality of boundary condition data such as the hydrological model calibration and historical ocean levels;

- the variability of events - preferably the events cover a range of flood conditions;
- changes to the floodplain;
- public perception and memory of floods.

The data availability for a number of floods is shown graphically in Figure 4-13 and the geographical location at which surveyed flood information is available for each of these floods is shown in Figure 4-15 to Figure 4-20. It can be seen from these figures that there is limited data available for the 1986 and 1996 floods. Of the other floods, those with the largest coverage of peak flood heights across the floodplain are the 1967, 1977, 1994, 1997 and 1999.

It was noted in Section 4.1 that the URBS model gave reasonable to good matches to recorded data in all events except 1982 which indicates that for most events, the hydrological model will provide reasonable boundaries for the hydraulic model. It is expected that the quality of the data will be better in the flood events following the installation of the Alert system in September 1989.

Recent floods will typically provide a better calibration because the current floodplain is used in the model: modifications can be made to the model representation of the floodplain for older events, but the reliability of the data decreases.

Public perception of floods can be an important factor in the selection of an event. For example, residents typically remember the 1967 flood as being the biggest flood in the last 40 years, and demonstrating a significant flood such as this on the model can be important in obtaining public confidence in the model.

Taking these factors into consideration, it was decided to select the three floods for calibration from the 1967, 1994, 1997 and February 1999 floods. The 1999 flood was selected because it was a recent major flood and the 1967 was selected because of the size of the flood and the public perception of it as a big flood. There was little difference between the 1997 and 1994 floods in terms of their value as a calibration event. It was decided to adopt the 1997 flood because it has additional data and is more recent than the 1994 flood.

4.3.1 Summary

A resident survey undertaken in August 2000 provided an opportunity to obtain important data on flood heights and flooding characteristics of the Johnstone River and its floodplains. It also provided an opportunity for the study team to ascertain concerns of the community in relation to flooding.

The BoM URBS model has been reviewed and found to be satisfactory for the purposes of this study, although some adjustment to the catchment discretisation and model parameters was undertaken during the joint calibration with the hydraulic model.

The three historical flood events selected for the calibration of the hydraulic model are the February 1999, the March 1997 and the March 1967 floods.

4.4 Model Calibration

4.4.1 Calibration Procedure

The general steps of the calibration and verification process were:

- process data for the selected events and set up boundary conditions for the hydrologic model;
- refine the existing BoM calibrated URBS hydrologic model discretisation to suit hydraulic model boundaries;
- run the URBS model using calibration parameters as supplied by the BoM to generate inflow boundaries for the TUFLOW hydraulic model;
- compile ocean tide boundaries for the hydraulic model for each of the three historical flood events;
- carry out initial calibration and verification of the TUFLOW model with parameters set at best estimated based on experience;
- continue calibration and verification of both hydrologic and hydraulic models using an iterative process which seeks to find the best combination of hydrologic and hydraulic parameters;
- present preliminary calibration to steering committee and community for review and feedback on flood extent and flooding patterns;
- finalise calibration based on feedback.

The tolerance for calibration to recorded levels is ± 0.2m. At all times, calibration parameters for both models are kept within conventional bounds.

4.4.2 February 1999 Calibration

The calibration of the model focussed on the recorded height-time history at the Innisfail Wharf Alert Station and surveyed peak flood levels along the banks of the river and across the floodplain, and general flooding patterns. Calibration parameters for both models were kept within conventional bounds, and consistency across the model was maintained.

The URBS hydrologic model allows the user the option of “matching” the hydrographs at river gauging stations for which a stream rating is available: a stream rating describes the variation in flow with river height at a particular location and hence allows the prediction of flow if the river height is known. River height-time histories and stream ratings were available at Nerada on the North Johnstone River and Central Mill on the South Johnstone River: Tung-Oil is closer to the boundary of the hydraulic model than Nerada, but was not available for this flood because the river height recorder malfunctioned. Figure 4-2 shows that at Nerada there is a good agreement between the streamflow predicted by the hydrologic model and that estimated using the recorded height and stream rating. Figure 4-3 shows that at Central Mill the agreement is not as good but is still reasonable. However, variation of this magnitude can impact on the calibration of a hydraulic model. Therefore, as part of the iterative process of jointly calibrating the hydrologic and hydraulic model, both matched and unmatched runs were trailed in conjunction with a range of Manning’s roughness coefficients. It was found that for this flood event, a better calibration of the hydraulic model was achieved if the hydrologic model predicted hydrographs were not matched with the recorded hydrographs at Nerada and Central Mill. This indicates that for this flood, the rating curves at the gauging stations were not of sufficient accuracy for the purposes of this study and that the runoff predicted by the hydrological model was more reliable.

The preliminary calibration presented to the Steering Committee and the community in March 2002 indicated that the model was well calibrated to the height-time history at Innisfail Wharf and to the peak flood heights in all areas except for West Innisfail. There was general agreement from the committee and the community that the animations and flood extent plots demonstrated that the model was replicating flooding patterns and flood extents. Some areas for improvement were noted including:

- The area to the south of Flying Fish Point Road near the crocodile farm did not show sufficient flood waters;
- The tramline downstream of the McAvoy bridge on the southern side of the river was not overtopped as indicated in the model;
- The model did not show water flowing from north to south on the western side of Sundown Hill (Dodd's property).

To improve the calibration and resolve the problems with flood extent, some additional survey was obtained including drainage structures and tramline levels. In particular the following changes to the model were undertaken:

- a bridge under the Queensland Government rail line at the northern end of Railway Street was added with a resulting improvement in the calibration in West Innisfail;
- a culvert under the tramline along the river bank to the west of Sundown Hill (near Dodd's property) was added which allowed water to flow to the south on the western side of Sundown Hill;
- the culverts under Flying Fish Point Road at the Crocodile Farm were incorporated resulting in an increase in the flow to the south of the road;
- some other culverts were included following receipt of survey data.

The flow across the tramline was difficult to resolve. Initially it was thought that the model was well calibrated at McAvoy Bridge because a good match was achieved with the flood level of RL 9.8 m AHD provided by the BoM. Since providing that level, BoM has changed their official record to RL 9.1 m AHD. The source for this level was Alan Dunne from the JSRIT. If the flood level at the bridge was 9.8 m AHD, then the tramline with a level of RL 9.1 m AHD must have overtopped. Further investigations by Alan Dunne lead to the conclusion that it was most unlikely that the tramline was overtopped, although the level of 9.1 m AHD could not be relied upon and it is likely to be higher. At this stage of the calibration process, a flood level of approximately RL 8.4 m AHD in the floodplain to the north of McAvoy Bridge became available. The model level in this area was RL 8.8 m AHD indicating the model was pushing too much water to the north, probably because of a high flood level in the river.

Given that the rest of the model was well calibrated and that this was achieved whilst maintaining internal consistency of parameters across the model, it was concluded that the conveyance of the river channel downstream of McAvoy Bridge may not be replicating the actual channel conveyance: the bed level in this area of the DEM was obtained by the linear interpolation of surveyed cross-sections and so there is some uncertainty in the waterway area. A number of trials were undertaken with the bed level in the river from the McAvoy Bridge down to Forrest Island being lowered to increase the conveyance. A reduction of 1.0 m in the bed level was found to reduce the flood level at the McAvoy Bridge to RL 9.45m. The tramline was not overtopped and a good match was obtained with the flood level in the floodplains to the north of McAvoy Bridge. A good match was achieved between modelled and recorded flood levels in the floodplain to the north of McAvoy Bridge. This is a strong indicator that the model is representing flow conditions in the river reasonably well because the flood level in a floodplain that is a storage area is strongly influenced by the flood level in the river system.

Figure 4-21 shows that good agreement was obtained between the recorded height-time histories and the hydraulic model results at the Innisfail Wharf Alert Station.

A comparison between the model peak flood levels and the surveyed peak flood levels is given in Figure 4-22 and in Table 4-3. Rather than displaying numbers in Figure 4-22, the difference between modelled and recorded flood levels are colour coded according to the legend in the drawing. The model is considered calibrated (yellow colour) if the difference falls within the range ± 200 mm. A red colour indicates that the model level is higher than the recorded level and a green colour indicates that the model is lower than the recorded flood level. It can be seen from this figure that the model is well calibrated over most parts of the floodplain.

Table 4-3 gives the recorded flood levels at each location shown in Figure 4-22 as well as the modelled level and the difference. A statistical analysis of the data in Table 4-3 is presented in Table 4-4. It shows that 92.5% of calibration points were within the accepted calibration range of ± 200 mm and that 55% of calibration points were within ± 50 mm. This analysis also indicates that the model is well calibrated.

The model is not inundated at location G26, which is a local drain, although there is a flood level recorded at this location. It is likely that floodwaters backed up this drain from the Ninds Creek area during the flood. However, the drain is relatively small in the context of this floodplain model and was not modelled in detail. Therefore, the model did not show water in this drain. This is not considered of consequence to the outcomes of this study.

The model was also compared to observations in the river around Carello's levee. The observations were documented in a field inspection report by Mr Errol Colman of DNRM. The letter states that the time of the inspection was 1:30 pm on Friday 12th February 1999. This was approximately 5-6 hours after the flood peak at the Innisfail Wharf. The letter states that the drop in water level at the bend in the river adjacent to Carello's levee is probably "*about 0.5 m over 300 m*". It should also be noted that the estimate was based on "visual indicators", not actual measurements.

To allow a comparison between the observed gradient and those in the model, the flood model levels at 2 pm (approximately the time of inspection) are shown in Figure 4-23. On the inside bend the model shows a drop in water level of 340 mm (RL 2.6 m AHD to RL 2.24 m AHD) over 340 m and over 640 m, a drop of 770 mm (RL 3.01 m AHD to RL 2.24 m AHD). On the opposite bank the change in flood level is less severe as was also observed by Mr Colman. This comparison indicates that the model is consistent with field observations.

There are no calibration points shown in the southern areas of the model. Initially it was not intended to model this part of the river system in the 2D hydraulic model. When the DMR photogrammetry data became available, the 2D model was extended to make use of this additional survey data. However, the model was not calibrated in this area and hydraulic controls such as the top of the river banks and roads were not represented in detail. Therefore, the model results should be used with caution in this area. If the model is to be used for detailed design work in this region, it is recommended that the model be reviewed and upgraded where necessary and some calibration data obtained.

Table 4-3 1999 Peak Flood Level Calibration Results

Flood ID	Recorded Peak Flood Level (m AHD)	Modelled Peak Flood Level (m AHD)	Difference (Modelled – Recorded) (mm)
G1	4.23	4.27	40
G2	4.24	4.27	30
G3	4.26	4.27	10
G4	4.25	4.44	190
G5	4.41	4.52	110
G6	4.39	4.53	140
G7	4.27	4.27	0
G8	4.24	4.27	30
G9	4.27	4.27	0
G10	4.24	4.27	30
G11	4.84	4.81	-30
G12	4.21	4.27	60
G13	4.24	4.27	30
G14	4.21	4.27	60
G15	4.73	4.70	-30
G16	4.28	4.27	-10
G17	4.82	4.85	30
G18	4.4	4.38	-20
G19	4.03	3.93	-100
G20	3.87	3.88	10
G21	4.72	4.49	-230
G22	4.9	4.87	-30
G23	3.94	3.94	0
G24	2.91	3.04	130
G25	2.89	3.04	150
G26	4.25	Not inundated in the model	
G27	5.35	5.49	140
G28	3.71	3.73	20
G29	3.33	3.56	230
G30	9.35	9.46	120
G31	4.94	4.88	-60
G32	3.617	3.63	10
G33	4.283	4.27	-10
G34	4.245	4.27	20
G35	4.401	4.52	120
G36	8.4	8.45	50
G37	8.39	8.45	60
G38	6.28	6.34	60
G39	7.45	7.05	-400
G40	8.4	8.44	40
G41	2.1	1.90	-200

Table 4-4 Statistical Analysis of 1999 Calibration

Range	Percentage of Calibration Points within Range (%)
-400 mm to –200 mm	5.0
-200 mm to -50 mm	7.5
-50 mm to +50 mm	55.0
50 mm to 200 mm	30.0
200 mm to 400 mm	2.5

4.4.3 March 1997 Model Calibration

A procedure for the calibration of the model to the 1999 flood was described in Section 4.4.2. The calibration of the model to the 1997 flood was done concurrently with the calibration of the 1999 flood. Decisions on adjustments to the model were made in the context of their impact on both floods.

Figure 4-24 shows that good agreement is obtained between the recorded height-time histories and the hydraulic model results at the Innisfail Wharf Alert Station. The differences on the rising limb at around 20:00 on the 22/03/97 and the peak occurring slightly early in the model indicate minor problems with the inflow boundaries generated by URBS.

A comparison between the modelled peak flood levels and the recorded peak flood levels given in Figure 4-25 and in Table 4-5. Figure 4-25 indicates that the model is well calibrated over most parts of the floodplain. It was found that for this flood event, a better calibration of the hydraulic model was achieved if the hydrologic model predicted hydrograph was matched to the recorded hydrograph at Central Mill and at Tung-Oil the model predicted hydrograph was adopted.

Table 4-5 gives the recorded flood levels at each location shown in Figure 4-25 as well as the modelled level and the difference. A statistical analysis of the data in Table 4-5 is presented in Table 4-6. It shows that 80% of calibration points were within the accepted calibration range of ± 200 mm. This analysis also indicates that the model is well calibrated.

The model is not inundated at location F10. This is the same location as G26 in the 1999 calibration event. As was explained for G26, this is a small drain in the context of this floodplain model and was not modelled in detail. Therefore, the model does not show water backing up this drain.

Table 4-5 1997 Peak Flood Level Calibration Results

Flood ID	Recorded Peak Flood Level (m AHD)	Modelled Peak Flood Level (m AHD)	Difference (Modelled – Recorded) (mm)
F1	4.17	4.31	140
F2	3.16	3.42	260
F3	3.23	3.42	190
F4	3.24	3.46	220
F5	3.29	3.42	130
F6	4.56	4.46	-100
F7	4.39	4.38	-20
F8	3.03	3.1	70
F9	4.34	4.44	100
F10	3.93	Not inundated in the model	
F11	3.42	3.53	110
F12	3.90	3.88	-20
F13	4.62	4.53	-90
F14	2.30	2.53	230
F15	3.52	3.53	10
F16	4.54	4.97	430
F17	3.49	3.56	70
F18	3.29	3.4	110
F19	4.50	4.43	-70
F20	4.37	4.44	70
F21	3.18	3.29	120
F22	4.35	4.44	80
F23	3.85	3.93	80
F24	8.00	8.26	260
F25	6.97	7.08	110

Table 4-6 Statistical Analysis of 1997 Calibration

Range	Percentage of Calibration Points within Range (%)
-400 mm to –200 mm	0
-200 mm to -50 mm	12.5
-50 mm to +50 mm	12.5
50 mm to 200 mm	54.2
200 mm to 400 mm	16.7
400 mm to 600 mm	4.1

4.4.4 1967 Model Verification

Given the lack of reliable data for input into the hydrologic model and potentially significant change to the topography since 1967, the 1967 flood was viewed as a verification event rather than a calibration event. This means that the model should generally match levels within the designated tolerance, but some discrepancies would be expected and that generally decisions on adjustment to parameters would not be based on the verification of this event. For this flood event, the hydrologic model predicted hydrographs were matched with the recorded hydrographs at Nerada and Central Mill.

The following adjustments were made to the model so that it was more representative of 1967 conditions:

- The height of Carello's levee was reduced;
- The floodgates were removed;
- The town dump was lowered to RL 2.6 m AHD;
- The McAvoy Bridge and its approaches were removed.

No time-height history is available for this flood at Innisfail Wharf, so only the comparison with the recorded peak flood levels given in Figure 4-26 and Table 4-7 is possible. The flood levels in the model were typically about 200 mm to 400 mm higher than the recorded levels as indicated in the statistical analysis given in Table 4-8.

One possible reason for the over-estimation of flood levels is that the river bathymetry in the model is not properly representing the 1967 conditions. Residents and members of the steering committee have suggested that sediment is building up in the river. If this is the case, the model will be under-representing the conveyance capacity of the main channel for the 1967 flood because the river bed levels would be too high. To test this theory, a sensitivity test was run with the river bed levels reduced by 500 mm. This improved the verification as shown in Figure 4-27, Table 4-9 and Table 4-10.

Table 4-7 1967 Peak Flood Level Calibration Results

Flood ID	Recorded Peak Flood Level (m AHD)	Modelled Peak Flood Level (m AHD)	Difference (Modelled – Recorded) (mm)
A1	4.86	5.20	340
A2	5.97	6.26	290
A3	5.21	5.10	-110
A4	4.26	4.45	200
A5	3.84	3.37	-470
A6	5.36	5.22	-140
A7	5.09	5.20	110
A8	4.90	5.20	300
A9	4.71	5.39	680
A10	4.91	5.20	290
A11	5.06	5.30	240
A12	9.50	9.23	-270
A13	3.44	3.49	50
A14	2.91	3.27	360
A15	4.72	5.31	590

Table 4-8 Statistical Analysis of 1967 Calibration

Range	Percentage of Calibration Points within Range (%)
-600 mm to –400 mm	6.7
-400 mm to –200 mm	6.7
-200 mm to +200 mm	33.3
200 mm to 400 mm	40.0
400 mm to 600 mm	13.3

Table 4-9 1967 Peak Flood Level Calibration Results – Bed Lowered 500 mm

Flood ID	Recorded Peak Flood Level (m AHD)	Modelled Peak Flood Level (m AHD)	Difference (Modelled – Recorded) (mm)
A1	4.86	4.91	60
A2	5.97	5.65	-320
A3	5.21	4.90	-310
A4	4.26	4.26	10
A5	3.84	3.00	-840
A6	5.36	5.00	-360
A7	5.09	4.91	-180
A8	4.90	4.91	10
A9	4.71	5.11	400
A10	4.91	4.91	0
A11	5.06	4.99	-70
A12	9.50	9.16	-340
A13	3.44	3.38	-60
A14	2.91	3.03	120
A15	4.72	4.99	270

Table 4-10 Statistical Analysis of 1967 Calibration – Bed Lowered 500 mm

Range	Percentage of Calibration Points within Range (%)
< -600 mm	6.7
-600 mm to –400 mm	0
-400 mm to –200 mm	26.7
-200 mm to +200 mm	53.3
200 mm to 400 mm	13.3
400 mm to 600 mm	0

4.5 Summary

A flood model comprising an URBS hydrological model and a TUFLOW fully two-dimensional hydraulic model has been developed for the Johnstone River and floodplain. The February 1999 and March 1997 floods were used as calibration events and the 1967 flood as a verification event. Overall, good agreement between recorded and hydraulic model flood levels was obtained for the calibration events, especially in the most recent February 1999 flood indicating that the model is reliably predicting the flooding behaviour of the current floodplain. It is recommended that results from the southern part of the hydraulic model be used with caution, as this part of the model was not calibrated.

5 DESIGN FLOOD ANALYSIS

In Section 4 it was demonstrated that the flood model of the Johnstone River reliably reproduces the flooding characteristics of the lower Johnstone River system. Therefore, the model can be confidently used to provide predictions of design flood events and assessment of floodplain works. This section details the analysis of design floods for the Johnstone River system.

5.1 Design Hydrology

Design floods are hypothetical floods used for planning and floodplain management investigations. A design flood is defined by its probability of occurrence. It represents a flood that has a particular probability of occurring in any one year. For example, the 1% AEP or 1 in 100 ARI flood is a best estimate of a flood which has 1 chance in 100 of occurring in any one year. It is important to acknowledge that the 100 year ARI event may occur more than once in a 100 year period as the definition of the event is that it occurs once, on average, in 100 years. Therefore, planning for the 1 in 100 year ARI flood does not guarantee protection for the next 100 years. Similarly, the 100 year ARI event may not occur at all within a 100 year period for the same reason. The 2 year, 5 year, 10 year, 20 year, 50 year and 100 year ARI were analysed.

There are two main methods of determining the magnitude of the flow for a design event. These are listed below and explained in the following sections:

- Flood frequency analysis (FFA)
- Design rainfalls with the URBS Model

5.1.1 Flood Frequency Analysis (FFA)

Flood frequency analysis (FFA) enables the magnitude of floods of selected ARI (Average Recurrence Interval) to be estimated by statistical analysis of recorded historical floods.

5.1.1.1 Stream Gauges

River heights have been recorded at several gauges in the Johnstone River catchment by the Department of Natural Resources and Mines (DNRM) as outlined in Section 3.4. Gauge data from the gauge sites listed in Table 5-1 was used to undertake the FFA. On both the North and South Johnstone, the gauge sites have been moved once during the recording history. On the North Johnstone the gauge site was moved in 1967 from Goondi to Tung Oil. On the South Johnstone, the gauge site was moved in 1974 from Central Mill to Upstream of Central Mill. In both cases the method of recording river levels was updated from manual to automatic recording.

Rating Curves

Gauges record river height. To convert this height to a flow, a conversion curve called a "rating curve" is used. For the gauge sites in the Johnstone River, two sources of rating curve are available. These are:

- Department of Natural Resources and Mines (DNRM)
- Bureau of Meteorology (BoM)

Table 5-1 Stream Gauge Stations

Gauge Number	River	Location	Maximum Gauged Stage / Flow (Date)	Period of Record	Catchment Area (km2)	Datum
112001a	North Johnstone	Goondi	1.35m / 110m^3/s (1935)	Oct-1928 to Jun-1968	936	SD
112004a	North Johnstone	Tung Oil	9.78m / 4060m^3/s (1967)	Jan-1967 – Current	925	SD
112101a	South Johnstone	Central Mill	4.38m / 400m^3/s (1945)	Jun-1916 – Oct-1974	401	AHD
112101b	South Johnstone	Upstream of Central Mill	6.55m / 680m^3/s (1977)	Oct-1974 – Current	400	AHD

In times of flood, DNRM may undertake a gauging, or rating, of the river flow by measuring flow with specialised apparatus, up to the maximum level reached by that particular flood. The DNRM rating curve is derived from these field measurements. Flow can only be measured up to the maximum flood level reached for that particular flood in which measurements were taken. If that particular flood is not a large flood, flow measurements are limited. Flows at river levels above the maximum must be estimated by extending the rating curve. DNRM rating curves at a gauge site may be updated by the DNRM over time when new field measurements are taken.

The Bureau of Meteorology (BoM) develops one rating curve for the gauge site that it believes is valid over the full period of record. The BoM rating curve is based on a mass balance undertaken using the URBS model in conjunction with the DNRM rating curve. BoM develop these curves to assist with flood forecasting. A BoM rating curve is not available for the Goondi gauge.

Rating curves at all gauges are provided in Figure 5-1 through Figure 5-4. Several DNRM rating curves may exist for a gauge site as mentioned previously.

The FFA has been undertaken using both the DNRM rated flows and the BoM rated flows to determine the sensitivity of the results to the rating curve.

Gauge Relocation

To achieve the best results for the FFA, the longest possible record length of flows is needed. In both the North and South Johnstone this is done by combining records from the old and new gauge sites. However, it is important to ensure that the flows at the new and old gauge sites are similar despite the relocation. If they are not similar, flows may need to be factored.

In relocating the gauge from Goondi to Tung Oil on the North Johnstone, the catchment area upstream of the gauge is reduced by 11km^2 to 925km^2 (about 1% reduction). This means that the Tung Oil location may receive less flow than the Goondi location. The Goondi and Tung Oil gauges were both in operation for a common 2 year period. This period is used to compare flows at both locations to determine if the relocation has an effect on flows. Table 5-2 summarises the findings of this comparison.

Table 5-2 Comparison of Common Record Periods After Gauge Relocation

River	Gauge Locations	Catchment Areas (km^2)	Reduction in Area due to Relocation (km^2)	Common Period of Record	Effect of Relocation on Flows
North Johnstone	Goondi	936	11 (1%)	Jun-1966 to Oct-1968	Increase in flows of 2% in common period
	Tung Oil	925			
South Johnstone	Central Mill	401	1 (0.25%)	Sept-1974 to Oct-1974	Not tested as change in area negligible & common period small
	Upstream of Central Mill	400			

Based on the common period of recording for Goondi and Tung Oil, it appears that the flows increased due to the relocation of the gauges. As the catchment area actually **decreased**, it is not expected that the flows would **increase**. Inspection of the annual maximum flows from each gauge, as shown in Figure 5-5, also appears to indicate that flows at Tung Oil may be higher than those at Goondi. It is not known however, whether this apparent increase in flows from 1967 onward may be a natural phenomenon, as DNRM believe that records from each gauge are good (personal communication with Alan Hooper from DNRM, 2000). In the absence of further information, the flows at each gauge must be considered accurate. However, factoring of the flows to account for the difference in catchment area is not undertaken.

In relocating the gauge on the South Johnstone from Central Mill to Upstream of Central Mill, the catchment area upstream of the gauges decreased by only 1km^2 (about 0.25%). In addition, the common period of recording was only 2 months for the South Johnstone gauges. A comparison of flows at the old and new sites over such a short common period may not produce a meaningful relationship. The small reduction in catchment area is also believed to have a negligible effect on flows and is therefore not investigated further.

For the purposes of the FFA, it is assumed that flows derived from height recordings at the old and new gauges on both rivers can be combined to produce continuous periods of record.

Stream Gauge Data Used for FFA

FFA is undertaken using the annual maximum flows. As mentioned previously in this section, two rating curves (DNRM & BoM) have been used to derive the flows. Annual maximum flows based on the DNRM rating curve at Goondi and the BoM rating curve at the other gauge sites are shown graphically in Figure 5-5 and Figure 5-6.

5.1.1.2 Analysis Techniques

Analysis techniques used are based on the recommendations from the proposed revision to Book 4 of ARR (2001) by Kuczera (2000). The L-Moment fitting method has been used to fit the data to the Generalised Extreme Value (GEV) theoretical probability distribution. This has been undertaken using the program HydroFreq 1.0 written by HydroTools Software in Canada. HydroFreq is also able to undertake a Maximum Likelihood fit to a Log Pearson Type 3 (LP3) distribution. Results from both the GEV and the LP3 distributions are provided for comparison. It most cases, the LP3 distribution appears to provide a better fit to the data at low and high ARIs. The fit through the bulk of the data is similar. As the GEV is soon to become the Australian standard, the GEV results are adopted.

5.1.1.3 North Johnstone FFA Results

FFA on the North Johnstone is initially undertaken for two scenarios:

1. Goondi (DNRM Rating*) + Tung Oil (BoM Rating*)
2. Goondi (DNRM Rating*) + Tung Oil (DNRM Rating*)

* Rating curve used to derive flows

The FFA results from the above two North Johnstone scenarios are compared in Figure 5-7. The effect of changing the Tung Oil rating curve (the only difference between Scenario 1 & 2) is small. The largest effect is evident in ARI events greater than the 100 year. Due to the minimal effect, Scenario 1 will be adopted as the North Johnstone FFA result as the development of the BoM rating curve is believed to be more rigorous.

Adopted FFA results are tabulated in Table 5-3 and plotted in Figure 5-9. These results allow the magnitude of major historical floods to be estimated as shown in Figure 5-8.

Table 5-3 North Johnstone FFA Results

ARI Event (years)	Goondi (DNRM Rating) + Tung Oil (BoM Rating) Flows (m^3/s)
2	1280
5	2200
10	2910
20	3660
25	3920
50	4760
100	5700
200	6740
500	8280

5.1.1.4 South Johnstone FFA Results

FFA on the South Johnstone is initially undertaken for two scenarios:

1. Central Mill + Upstream of Central Mill (Both using BoM Rating*)
2. Central Mill + Upstream of Central Mill (Both using DNRM Rating*)

* Rating curve used to derive flows

The FFA results the above two South Johnstone scenarios are compared in Figure 5-10. The effect of changing the rating curve is significant with the difference in 100 year ARI flows of more than 500m^3/s. This is due to the substantial difference between DNRM and BoM rating curves as evident in Figure 5-3 and Figure 5-4. DNRM rating curves at both South Johnstone gauge sites are not rated to large floods. That is, a substantial proportion of the curve is extrapolated (estimated beyond what has been measured). While this is also true for the BoM rating curves, the BoM curves are developed considering a mass balance undertaken using the URBS model for a range of historic events. In the absence of further information, the BoM rating curve is considered to be the most accurate method for converting heights to flows for the South Johnstone gauges. Thus, Scenario 1 will be adopted.

Adopted FFA results are tabulated in Table 5-4 and plotted in Figure 5-12. These results allow the magnitude of major historical floods to be estimated as shown in Figure 5-11.

Table 5-4 South Johnstone FFA Results

ARI Event (years)	Central Mill + Upstream of Central Mill (BoM Rating) Flows (m^3/s)
2	590
5	1020
10	1330
20	1660
25	1770
50	2120
100	2510
200	2920
500	3520

5.1.2 Design Rainfalls with the URBS Model

5.1.2.1 Design Rainfalls

Design **flood** events are produced using design **rainfall** events. To determine the intensity and distribution of rainfall that will produce a specified ARI design event, charts developed by the Bureau of Meteorology (BoM) are consulted. These charts are contained in a book called "Australian Rainfall and Runoff"(IEAust, 2001). The BoM produced these charts by analysing all historical rainfall information available around Australia to produce guidelines on the intensity and distribution of rainfall for each ARI event for each region of Australia.

5.1.2.2 URBS Model

Once the design rainfalls have been determined using the guidelines in IEAust (2001), these are input into the URBS hydrologic model. As explained during calibration, the URBS model determines how the rainfall is converted to runoff across the whole catchment. Thus, the URBS model takes a particular ARI rainfall event and produces the equivalent ARI flow.

Parameters used to calibrate the model during the calibration process are used to assist in the selection of parameters used for design hydrological model to ensure that the model is representing the true nature of the catchment. These parameters are listed below.

- Initial Loss = 50mm
- Continuing Loss = 4mm/hr
- m = 0.7

- $\alpha = 0.12$, $\beta=3$

The initial loss used during the calibration of the hydrological model varied from 0 mm to 140 mm and the continuing loss had some variation but was typically 4 mm/hr. m, α, and β were the same for all calibration runs and the same parameters were adopted for the design model.

IEAust (2001) recommends the use of an Areal Reduction Factor for medium to large catchments. This factor is designed to account for the fact that rainfall does not fall at the full intensity over an entire catchment. As would be expected, the reduction increases as the catchment area increases. A reduction factor of 0.9 was used for the Johnstone catchment.

5.1.2.3 Design Flows

Using the parameters listed above, the design flows were initially extracted at the gauge sites to provide a comparison with the results of the Flood Frequency Analysis (FFA).

5.1.3 Comparison of Design Flows with FFA Results

Design flows predicted by the URBS model at the gauge sites are compared with the results of the Flood Frequency Analysis (FFA).

5.1.3.1 North Johnstone

Figure 5-13 provides a comparison of the FFA results with the design flows at Tung Oil on the North Johnstone. Design flows predicted by the URBS model are higher than the flows produced from the FFA. Sensitivity of the FFA to several factors is tested to determine a reason for this difference. ARIs for major events can be estimated from the design event curve in the same way that they were estimated from the FFA results in Section 5.1.1.3 and Section 5.1.1.4. The estimations are provided in Table 5-5.

Table 5-5 Estimation of Major North Johnstone Historical Event ARIs

Major Historical Event	ARI based on FFA Results	ARI based on Design Events
1932	13	4
1935	18	5
1967	40	20
1979	12	5
1982	36	18
1986	36	18
1994	26	14
1997	18	9
1999	23	12

5.1.3.2 Sensitivity Testing of North Johnstone FFA

Sensitivity to Gauge Relocation

As mentioned in Section 5.1.1.3, the North Johnstone stream gauge has been relocated once in its recording history. It was noted that the relocation from Goondi to Tung Oil lead to an apparent increase in flows. It is not known whether this increase is a real natural phenomenon (flows may have been naturally higher since 1974) or if the apparent increase was caused by an inaccurate rating curve at either gauge. A FFA is undertaken on each gauge record separately to test the sensitivity of results to the apparent increase in flows.

Sensitivity to Non-Recorded Major Historical Events

Gauge recording on the North Johnstone began in 1929. This provides a continuous period of record of 72 years. Anecdotal evidence (Alan Dunne, 1999, and some subsequent minor adjustments to this reference during personal communication, Alan Dunne, 2003) suggests that some large flood events occurred in the very early part of the 1900s and in the late part of the 1800s. These events are obviously not included in the North Johnstone gauge records. A specialised feature of FFA is that it is able to include historical events, not part of the continuous period of record, if information on flows for these events is available. In this case, flow data is not available for these events. However, Alan Dunne has provided an estimate of the relative magnitude of the events and this is used to roughly estimate the magnitude of non-recorded historical events. It must be noted that this is very rough and is done for the purposes of sensitivity testing only.

The non-recorded historical events are shown in Table 5-6. An estimation of flows for these events is included in the FFA. Recorded flows are given an increased weighting based on the historical record extension length. FFA results estimated by extending the historic record length back to 1878 are shown in Figure 5-14.

Table 5-6 Non-Recorded Historic Events

Year	Estimated Flow* (m^3/s)	Comment from Alan Dunne
1878	9000	About 4.5 m higher at Innisfail than 1967
1894	5500	About 1.6 m higher at Innisfail than 1967
1911	5000	About 0.7 m higher at Innisfail than 1967
1913	5800	About 1.7 m higher at Innisfail than 1967
1927	4450	About equivalent to 1967

* rough estimate only

Sensitivity to Frequency Distribution

As described in Section 5.1.1.2, there are a number of analysis techniques that may be used to undertake a FFA. In this case, the GEV distribution has been used in conjunction with the L-moments fitting method. However, the LP3 distribution is also available. It is used is conjunction with the Maximum Likelihood fitting technique as a sensitivity check. Results are provided in Figure 5-14.

Discussion

The apparent increase in flows resulting from relocating the gauge from Goondi to Tung Oil has a significant impact on FFA results as shown in Figure 5-14. DNRM undertook rating at Tung Oil during the largest recorded flood. Thus, the Tung Oil rating curve is considered to be one of the most accurate. Rating at Goondi was undertaken in a small event and the majority of the rating curve is estimated (as shown in Figure 5-1). Thus it is believed that the Goondi flows may be inaccurate and apparently low.

Including the major historical floods also has a significant impact on the FFA results. Although these are a rough estimate, the general trend is that the FFA curve shifts upwards towards the design flow curve with the inclusion of these major events.

FFA results show some minor sensitivity to the distribution used. It is not considered significant enough to alter the adopted distribution.

5.1.3.3 South Johnstone

Figure 5-15 provides a comparison of the FFA results with the design flows at Upstream of Central Mill on the North Johnstone. The design flows are significantly higher than the results provided by the FFA. Further sensitivity testing of FFA results was undertaken in an attempt to understand the reason for the difference. ARIs for major events can be estimated from the design event curve in the same way that they were estimated from the FFA results in Section 5.1.1.3 and Section 5.1.1.4. The estimations are provided in Table 5-7.

Table 5-7 Estimation of Major South Johnstone Historical Event ARIs

Major Historical Event	ARI based on FFA Results	ARI based on Design Events
1932	23	6
1935	12	3
1946	46	12
1967	68	16
1986	47	12
1994	38	9
1996	20	5
1997	15	4
1999	14	4

5.1.3.4 Sensitivity Testing of South Johnstone FFA

Sensitivity to Gauge Relocation

As mentioned in Section 5.1.1.1, the South Johnstone stream gauge has been relocated once in its recording history. Unlike, the North Johnstone, the relocation does not produce a marked discrepancy in flows. However, a FFA is undertaken on each gauge record separately to test the sensitivity of results to the gauge relocation.

Sensitivity to Non-Recorded Major Historical Events

Gauge recording on the South Johnstone began in 1916. This provides a continuous period of record of 85 years. Anecdotal evidence (personal communication, Alan Dunne, 2000) suggests that some large flood events occurred in the very early part of the 1900s and in the late part of the 1800s. Some of these events are obviously not included in the South Johnstone gauge records. A specialised feature of FFA is that it is able to include historical events, not part of the continuous period of record, if information on flows for these events is available. In this case, flow data is not available for these events. However, Alan Dunne has provided an estimate of the relative magnitude of the events and this is used to roughly estimate the magnitude of non-recorded historical events. It must be noted that this is very rough and is done for the purposes of sensitivity testing only.

The non-recorded historical events are shown in Table 5-6. An estimation of flows for these events is included in the FFA. Recorded flows are given an increased weighting based on the historical record extension length. FFA results estimated by extending the historic record length back to 1878 are shown in Figure 5-14.

Table 5-8 Non-Recorded Historic Events

Year	Estimated Flow* (m^3/s)	Comment from Alan Dunne
1878	6000	About 4.5 m higher at Innisfail than 1967
1894	3000	About 1.6 m higher at Innisfail than 1967
1911	2500	About 0.7 m higher at Innisfail than 1967
1913	3200	About 1.7 m higher at Innisfail than 1967

* rough estimate only

Sensitivity to Frequency Distribution

As described in Section 5.1.1.2, there are a number of analysis techniques that may be used to undertake a FFA. In this case, the GEV distribution has been used in conjunction with the L-moments fitting method. However, the LP3 distribution is also available. It is used is conjunction with the Maximum Likelihood fitting technique as a sensitivity check. Results are provided in Figure 5-16.

Sensitivity to Rating Curve

Figure 5-10 indicates that the FFA results are sensitive to the differences in the BoM and DNRM rating curve. There is no further need to test the sensitivity of the results to the rating curve selection here. However, it is necessary to provide a reminder that the DNRM has not undertaken a rating of a major flood on the South Johnstone. Thus, the DNRM rating curves for the South Johnstone gauges are estimated for a substantial portion of the curve. The BoM has provided a sensibility check and revision of the DNRM curves based on mass balances using the URBS model. It is for this reason that the BoM rating curves are adopted for the FFA but it still remain that the BoM curve is based on the original DNRM curves.

Discussion

Flows produced by the FFA change slightly when considering the two gauges on the South Johnstone separately. Higher flows are produced for Upstream of Central Mill. However, the difference is not considered significant and may be more related to record length and time period than gauge relocation.

Inclusion of the major historical floods has a significant impact on the FFA results. The FFA skews upwards at the larger events. The FFA curve rises above the design event curve for events greater than about the 50 year ARI event. Although these are a rough estimate, the general trend appears to push the FFA toward the design event curve and some event magnitudes above it.

FFA results show some minor sensitivity to the distribution used. It is not considered significant enough to alter the adopted distribution.

5.1.3.5 Summary of FFA versus Design Events

North Johnstone

The FFA results indicate lower ARI flows than the design event flows as shown in Figure 5-13. However, after sensitivity testing the FFA analysis, it is believed that the design flows are realistic. The lower flows predicted by the FFA are due to a combination of:

1. Gauge Relocation – The Goondi rating curve is believed to be inaccurate due to the low flow at which it is rated. The inaccuracy is apparent in a comparison of flows and FFA at the Goondi and Tung Oil gauges. Flows at Goondi are believed to under-estimated.
2. Record Length – The continuous record length on the North Johnstone extends back to 1929. However, several major floods occurred in the 50 years preceding this record. When an estimate of these major floods is included in the FFA, flows increase to similar values as that given by the deisgn events.

South Johnstone

The FFA results indicate significant lower ARI flows than the design event flows as shown in Figure 5-15. After sensitivity testing the FFA analysis, it is still not clear which method of determining the design flows was more realistic. This matter was then referred to the Steering Committee for discussion. The adopted hydrology is discussed in Section 5.1.5.

5.1.4 Joint Probability Analysis

A joint probability analysis is used to determine the likelihood of two floods of similar magnitude occurring simultaneously in the North Johnstone and South Johnstone Rivers, or to use another term, to determine the correlation between the two catchments. For example the probability of a 100 year ARI flood occurring in both the North and South Johnstone Rivers at the same time may not be 1 in 100, but may be less likely. If that is the case, it would not be statistically correct to represent the 100 year ARI design flood as a combination of a 100 year ARI event in the North Johnstone and a 100 year ARI event in the South Johnstone. The same can be said for all other magnitudes of flood event.

IEAust (2001) refers to Laurenson (1974) for a methodology to determined the magnitude and associated probability of floods downstream of a confluence. The methodology requires that a FFA be undertaken on one arm, a conditional FFA on the other arm and that an empirical relationship amongst peak flows of each of the arms and downstream of the confluence be determined. The analysis could not be undertaken for several reasons:

1. the uncertainty in the FFA on the South Johnstone River as previously discussed;
2. lack of long-term hydrographic records of sufficient detail in both rivers - the analysis requires the full hydrograph for each flood so that the effects of timing can be considered;
3. the significant effect of floodplain storage on the peak flood flow downstream of the confluence in the Johnstone River.

Although a rigorous analysis could not be undertaken, a comparison of the ARI of corresponding floods and the North and South Johnstone Rivers was undertaken as detailed in the following section to obtain a "feel" for the correlation of the two catchments.

5.1.4.1 Comparison of Corresponding Floods

A comparison of corresponding floods in the North and South Johnstone may lead to the development of a relationship between the magnitudes in each river. As records exist in both rivers since 1929, a comparison of corresponding floods over the common period of record can be undertaken. The comparison is undertaken using by firstly comparing the ARI of the flood in each river and then the flow in each river.

The procedure used to undertake the ARI comparison is as follows:

1. Select one river as being the Primary River;
2. For each annual maximum flow on the Primary River, calculate the ARI of this flow based on the design event flows;
3. Determine the date at which each annual maximum flow occurred on the Primary River;
4. Extract the corresponding flow from Secondary River records by selecting the maximum flow over the 3 day period surrounding the date of the annual maximum flow on the Primary River;
5. For each corresponding flow extracted on the Secondary River, calculate the ARI of this flow based on the design event flows;
6. Plot the Primary River ARI versus the Secondary River corresponding ARI

7. Investigate a potential relationship between ARIs on each river.
8. Return to Point 1 – the Secondary River will become the Primary River.

Results of this process are presented in Figure 5-17 and Figure 5-18. As shown in these figures, there are some flood events in which there is no correlation between the ARI of each river, and in other events there is a strong correlation. From this analysis it can be concluded that the catchments are partially correlated.

The second analysis investigated the correlation between flows in each river rather than the ARI. This analysis is presented in Figure 5-19and Figure 5-20. Figure 5-19 is a plot of the annual maximum flow in the North Johnstone River and the corresponding flow, but not necessarily the peak flow for that year, in the South Johnstone and Figure 5-20 is vice versa. The correlation coefficient for the line of best fit is about 0.7 in both cases indicating partial correlation in the data. The correlation analysis indicates the following:

1. the peak flow in the North Johnstone is about 2.5 times that in the South Johnstone during the peak annual flow in the North Johnstone;
2. the peak flow in the North Johnstone is about 1.8 times that in the South Johnstone during the peak annual flow in the South Johnstone.

This correlation was then considered in the context of the average recurrence interval. For example, if the 100 year ARI flow in the North Johnstone is 6210 m^3/s, then the corresponding flow in the South Johnstone could be assumed to be 2480 m^3/s which corresponds to a 25 year ARI using the URBS analysis or 70 year ARI using the FFA analysis. This analysis for the full range of floods taking the North Johnstone River as the primary river is presented in Table 5-9 and with the South Johnstone River as the primary river in Table 5-10. For a flood of a particular ARI in the North Johnstone, the corresponding flood in the South Johnstone is likely to smaller, especially in larger events. However, for a flood of a particular ARI in the North Johnstone, the corresponding flood in the South Johnstone is likely to be of similar magnitude. This apparent dependence on which river is taken as the primary river may be a physical characteristic related to the storm patterns and the relative catchment size, but would also be influenced by the uncertainty in the URBS modelling and the flood frequency analysis which are used to assign the ARI.

Table 5-9 ARI Comparison – North Johnstone as Primary River

North Johnstone ARI (yrs)	North Johnstone Flow (URBS) (m^3/s)	Corresponding South Johnstone Flow (m^3/s)	South Johnstone ARI (yrs) (URBS)	South Johnstone ARI (yrs) (FFA)
2	2120	850	<2	4
5	3090	1240	2	8
10	3660	1460	3	13
20	4450	1780	6	30
50	5400	2160	12	50
100	6210	2480	25	100

Table 5-10 ARI Comparison – South Johnstone as Primary River

South Johnstone ARI (yrs)	South Johnstone Flow (URBS) (m3/s)	Corresponding North Johnstone Flow (m3/s)	North Johnstone ARI (yrs) (URBS)	North Johnstone ARI (yrs) (FFA-Goondi & Tung-Oil)
2	1170	2110	2	4.5
5	1680	3020	4.5	11
10	2000	3600	9	18
20	2410	4340	17	35
50	2900	5220	42	70
100	3330	6000	85	130

5.1.5 Adopted Design Hydrology

Because of the uncertainties in the design hydrology it was necessary to obtain the Steering Committee's position on the level of risk that should be incorporated into the analysis, particularly given that the potential impact on design flood levels, and hence habitable floor levels for future developments. These issues were discussed at the committee meeting on the 2 June 2002 and at that meeting the Johnstone Shire Council advised that a conservative approach (minimal risk) should be adopted in setting 100 year ARI flood levels. The following outcomes were then agreed:

1. The URBS design flows will be adopted on the North Johnstone River (refer Figure 5-21):

2. On the South Johnstone River, flows from the FFA (upstream of Central Mill only analysis) will be adopted for floods up to the 10 year ARI event, the URBS flow will be adopted for the 100 year ARI event, and for floods of magnitude between the 10 year and 100 year ARI the flows are to be interpolated (refer Figure 5-22);

3. Floods of equal magnitude are to be applied simultaneously on both rivers.

On the South Johnstone River, it was concluded that the FFA should be reliable up to the 10 year ARI event given the length of record analysed and hence it would be too conservative to adopt the URBS flows. However, the FFA did not include the large floods from the late 1800's and the early 1900's and hence it was considered that the FFA is underestimating the 100 year ARI flows.

The dynamic hydraulic model (TUFLOW) requires inflow hydrographs with the flow varying with time. The URBS model provides such boundaries, but the FFA only provides a peak flow. Therefore, on the South Johnstone River, the design flow hydrographs from the URBS model were factored to match the peak flow determined as described above. Peak flood level and extent for the 2, 5, 10, 20, 50 and 100 year ARI flood events are presented in Figure 5-23 to Figure 5-28 respectively.

The adopted peak design flows for each river are presented in Table 5-11.

Table 5-11 Adopted Peak Design Flows

ARI (Years)	Peak Design Flow (m^3/s)	
	North Johnstone River	South Johnstone River
2	2100	700
5	3060	1200
10	3630	1600
20	4400	2100
50	5340	2800
100	6140	3330

5.2 Design Hydraulics

The calibrated TUFLOW hydraulic model described in Section 4.2 was adopted as the design model. The model represents the current floodplain topography and as such, it was not necessary to update the model. Inflow boundaries for the hydraulic model were obtained from the URBS hydrological model.

In the calibration model, the ocean boundary was the recorded ocean levels during the flood event itself. For design runs it is necessary to synthesize an ocean boundary. An ocean boundary tidal cycle was synthesized using a pattern including the HAT, the MLWN, the MHWS and the MLWS tide levels. The HAT was timed to coincide with the peak of the flood wave in the lower reaches. In setting a tidal ocean boundary, consideration of storm surge and wave setup is required. The Beach Protection Authority Queensland publication "Storm Tide Statistics – Mourilyan Region" was used as a guide. This report shows that tide levels in this region are affected by storm surge for the 10 hours prior to and 10 hours following the passing of the eye of a storm over the coastline and provides a methodology for determining the surge and setup. The surge and setup was incorporated into the synthesized ocean boundary for floods greater than a 50 year ARI as recommended, although the size of the catchment means that the peak flood heights in the Innisfail area typically occur well after the effects of the storm surge have passed. Therefore, storm surge is unlikely to influence the peak flood levels.

Peak design flood levels, velocities, flows, and depths were generated for the 2 year, 5 year, 10 year, 20 year, 50 year and 100 year ARI design flood events. A peak flood level surface covering the 2D model area was generated for each of these runs. This peak flood level surface is a GIS layer that can be overlaid on digitised topographic maps.

The peak flood heights and extents for each of the design floods are mapped in Figure 5-23 to Figure 5-28 for the entire floodplain modelled in TUFLOW. Table 5-12 gives a comparison between the peak design flood levels and approximate historical flood levels.

Figure 5-29 to Figure 5-31, which are centred on Innisfail, compare the peak 100 year, 20 year and 5 year ARI flood heights from this study with the 1967 and 1999 recorded peak flood heights, and the peak 100 year ARI flood heights in Cameron McNamara (1985). Note that in Figure 5-30 and Figure 5-31, the peak 20 year and 5 year ARI levels from the current study are compared to the 100 year ARI Cameron McNamara (1985) peak flood heights. In these figures, a positive number indicates that the TUFLOW flood level is higher than the flood level to which it is being compared. For example, Figure 5-29 shows that the TUFLOW peak 100 year ARI level in the Innisfail CBD is:

- about 0.9 m higher than the Cameron McNamara 100 year ARI flood level;
- about 2.0 m higher than the 1999 flood level;
- about 1.3 m higher than the 1967 flood level.

Although the 100 year design flood levels in this study are higher than those in Cameron McNamara (1985), it is clear from the comparison with historical flood levels that the 100 year design flood levels are not unrealistic, and have been exceeded on several occasions in the last 125 years.

Table 5-12 Comparison Between Historical Floods and Design Floods

Flood (Year or ARI)	Flood Height at Innisfail Wharf Gauge (m AHD)*	Flood Height at Innisfail Wharf Gauge (m Gauge)
1878	9.0	11.0
1913	6.1	8.1
1894	6.0	8.0
100	5.4	7.4
1911 & 1935	5.1	7.1
50	4.9	6.9
20	4.5	6.5
1999	4.4	6.4
1967 & 1927	4.35	6.35
10	4.1	6.1
1997	3.85	5.85
5	3.8	5.8
2	2.7	4.7

*Some of the early floods are approximate levels only supplied by Alan Dunne (Dunne, 1999 and pers.comm.2003). The 1913 level is considered to be reasonably reliable as the level supplied by the BoM was independently verified by A. Dunne. In the Innisfail CBD the March 1967 flood levels were higher than the February 1999.

5.3 Summary

Uncertainties in the determination of the design hydrology required that input be obtained from JSC on the level of risk that should be incorporated into the hydrology. JSC advised that a minimal risk (conservative) approach should be adopted for the 100 year ARI event. The calibrated hydraulic model was then used to determine flood flows, heights, extent, depths and velocities for a range of design flood events. The design flood levels are consistent with historical flood levels.

6 EXISTING FLOOD DAMAGE ASSESSMENT

6.1 Background

To improve floodplain management on the Lower Johnstone River and to allow the effectiveness of management measures to be assessed, damages from flooding incurred on the floodplain need to be quantified. These damages establish the socio-economic costs to society and are used to quantify the benefits of certain mitigation measures (eg. levees).

The Lower Johnstone River region is a primary industry based economy serviced by a number of townships, the largest being Innisfail. The region comprises predominantly floodplain lands used for sugar cane, banana and pastoral activities. During flooding under existing conditions, agricultural activities sustain substantial flood damage, reflecting the location of these activities in the floodplain. Damages are not limited to the agricultural sector with significant damages also occurring to residential property, businesses and public infrastructure, particularly in larger floods.

Flood damages are classified as tangible or intangible, reflecting the ability to assign monetary values. Intangible damages arise from adverse social and environmental effects caused by flooding, including factors such as loss of life and limb, stress and anxiety. Tangible damages are monetary losses directly attributable to flooding. They may occur as direct or indirect flood damages. Direct flood damages result from the actions of floodwaters, inundation and flow, on property and structures. Indirect damages arise from the disruptions to physical and economic activities caused by flooding. Examples are the loss of sales, reduced productivity and the cost of alternative travel if road and rail links are broken.

For the purposes of this assessment, flood damages are classified into the following categories:

- Tangible
    - ⇒ Rural Damages
    - ⇒ Urban Damages (residential, commercial and industrial)
    - ⇒ Infrastructure Damages
- Intangible Damages

The flood damages assessment drew upon:

- the flood modelling results;
- ground level data of the study area;
- aerial photography to ascertain land use;
- previous damages assessments completed for the JSRIT by Cameron McNamara (1985).

6.2 Previous Investigations

Cameron McNamara undertook a damages assessment as part of the Johnstone River Systems Flood Management Study (Cameron McNamara, 1985). The 1985 study only calculated urban (residential and commercial) damages in Innisfail and Mourilyan. In 1985 dollars, the existing urban damages in

Innisfail in a 100 year ARI flood event were calculated to be $1,570,000 and in a 50 year ARI flood the damages were calculated to be $990,000. In Mourilyan, the damages were $160,000 and $70,000 respectively.

6.3 Tangible Damages

The methodology for the calculation of rural and urban damages broadly follows these steps.

1. Identify the areas inundated and the depth of inundation for the range of design flood events (2, 5, 10, 20, 100 year ARI) modelled using the TUFLOW hydraulic model.

2. Define land uses, building types and floor heights.

3. Apply damage relationships to the areas inundated.

4. Calculate the total damage for each design flood event and present the results in a probability-damage graph.

5. Determine the average annual damages (AAD).

The AAD is the average damage in dollars per year that would occur in a designated area from flooding over a very long period of time. In many years there may be no flood damage, in some years there will be minor damage (caused by small, relatively frequent floods) and, in a few years, there will be major flood damage (caused by large, rare flood events). Estimation of the AAD provides a basis for comparing the effectiveness of different floodplain management measures (i.e. the reduction in the annual average damage). The AAD is the area under the probability-damage graph.

The above methodology for the calculation of rural and urban damages is expanded and the calculations presented in Sections 6.3.1 and 6.3.2 respectively. Public infrastructure damages are not included in these calculations, but a discussion on these damages along with some historical data is provided in Section 6.4.

6.3.1 Rural Damages

Rural flood damages were calculated using the following steps.

- Identify the areas inundated and the depth of inundation for the range of design flood events (2, 5, 10, 20, 100 year ARI) modelled using the TUFLOW hydraulic model
- Define existing rural landuses:
  - ⇒ cane farming
  - ⇒ banana farming
  - ⇒ beef grazing
- Review latest research on flood damage to sugar cane, banana and beef grazing to determine damage relationships.
- Apply damage relationships to the areas inundated and present the results in a probability-damage graph.

- Determine the AAD.

The extent of the damages assessment was based on the coverage of the fully two-dimensional model (refer Figure 4-14). The extent of the 2D model was defined to extend beyond the outer limit of possible change in flood characteristics as a result of a structural mitigation option. Not all of the area covered by the model is inundated during floods, particularly during the smaller floods.

The area inundated for the range of design flood events was interpreted from the flood surface generated by TUFLOW and exported to a Geographical Information System (GIS). A summary of the total inundated area for different design floods is given in Table 6-1. This data was then further interpreted within the GIS to determine the areas of specific landuses inundated during the various floods. Other data that was extracted for the analysis included flood depth for calculating the damage to sugar cane and banana farms.

Table 6-1 Total Inundated Areas (Rural & Urban)

Flood Event (years ARI)	Total Area Inundated (ha)
100	10,000
50	9,200
20	7,370
10	6,100
5	5,220
2	3,810

6.3.1.1 Rural Landuse

Rural flood damage varies according to the different landuses across the floodplain. The Johnstone River floodplain is predominantly used for sugar cane and banana farming, but there is a small beef grazing industry. A description of landuse was obtained by interpreting the aerial photography obtained for this project. Analysis of this information in combination with the inundated areas described in Section 6.3.1 was carried out to determine the inundated areas for each landuse (see Table 6-2). The areas given in Table 6-2 are the area of land inundated that was used in calculation of the flood damage. For example, flood damage to sugar cane is assumed to occur where the flood depth (D) is greater than 1.2 m, as explained subsequently in Section 6.3.1.2. Therefore, the area quoted in this table is for locations where the depth > 1.2 m.

Table 6-2 Rural Landuse Areas Inundated

Flood Event (years ARI)	Total Area Inundated used in Damages Calculation (ha)	Breakdown of Land use Inundation		
		Sugar Cane (D>1.2 m) (ha)	Banana (D>2.5 m) (ha)	Beef (ha)
100	5,950	4,080	410	1,460
50	5,045	3,330	325	1,390
20	3,887	2,365	212	1,310
10	3,058	1,700	148	1,210
5	2,427	1,240	97	1,090
2	1,547	680	27	840

6.3.1.2 Sugar Cane Growing

The methodology adopted for this study is the same as that used by WBM on the Herbert River Flood Study, another study currently being undertaken by WBM. As part of that study, discussions were held with representatives from the BSES Canegrowers. The following points summarise these discussions.

1. BSES (1977) has been updated by Kingston et al (1999). The later report includes data from different regions and flood events and includes an algorithm to assist in the automation of the calculation of damages rather than using the nomograph given in BSES (1977). These investigations have found that sugar cane damage is a function of depth of inundation and duration of inundation above the growing point. During periods of sunshine, boiling of young sugar cane can occur if a shallow depth of water is around the cane for an extended period. This type of damage is normally associated with significant local runoff rather than flooding from the Johnstone River because Johnstone River flooding typically occurs from January through to March when the cane is sufficiently mature to be resistant to boiling.

2. Sugar cane is grown on a yearly cycle and an average height of cane in January would be 1.0 m and by April the average height would be 1.4 m.

3. Typically about 12% of caneland is fallow at any time.

4. An average yield would be 85 tonnes/ha with good seasons yielding 100 tonnes/ha and poor seasons 50 tonnes/ha.

5. An average price for cane is $25/tonne in 2002 dollars.

Kingston et al (1999) shows that for a stalk height of 1.2 m, the reduction in yield is not overly sensitive to duration of submergence for periods up to 6 days. More specifically, it specifies a reduction in yield of 11% for four days submergence and 13% for 6 days submergence. It is unlikely that sugar cane in the study area would be submerged for longer than 6 days, so a reduction in yield of 13% was conservatively adopted.

The calculation of damages only includes land inundated from Johnstone River flooding in the study area. Damage to sugar cane outside study area or within the study area but on higher ground is likely. Therefore the damages for the shire may be greater than indicated here.

6.3.1.3 Banana Farming

Discussions to date with industry representatives have revealed that there has been no research undertaken, and hence no published data available, on flood damages to banana plantations. However, sufficient information was obtained from Dennis Dillon, Dave McCarthy and Cameron Mackay, farmers with crops in the floodplain, to allow a preliminary analysis. There were some small differences in advice, but generally their comments were consistent.

A summary of the flood damage mechanisms is given below.

1. Scouring – plant will recover if stool not damaged.

2. Inundation

    a. Damage will occur if inundation > 24 hours

    b. Stool can be killed leading to up to 18 month loss in production depending on how quickly replanting occurs, ie, current crop and part of next year

    c. Sediment in heart if flood water sufficiently deep will also kill tree leading to a two year loss in production

    d. If follower killed but crop not lost, production from next year is lost.

3. Wind Damage – not within the scope of this study, but crops are lost when top is snapped off.

In the period January to April there is considerable variation across the floodplain in the maturity of the trees and hence the depth of inundation that would cause damage. It was concluded that a reasonable average assumption would be that death of the plant would occur if the depth exceeded 2.5 m; it was assumed that the inundation period was greater than 24 hours if depth exceeded 2.5 m. Damage of this nature would result in a loss of production up to 18 months.

Dennis Dillon provided the typical costs summarised in Table 6-3 for input into the economic analysis of flood damage. Cameron was not specific with costs and margins, but his general comments were consistent with the figures provided by Dennis. The typical yield is 2500cartons/ hectare and at any time there is about 20% of land fallow. Assuming trees are killed, the producer would incur losses associated with the growing costs and margin, ie, $8.50/carton, and then loss of margin for a further 6 months, ie, $2.00/carton/year or $1.00/carton/6 months. In total, the damages incurred following the death of the trees is assumed to be $9.50/carton or $24,000/hectare (rounded) over an eighteen month period. A 20% reduction in losses to allow for fallow land is then applied to produce the final damages. The calculation of damages presented in Section 6.3.1.5 show that damage to banana crops is the predominant contributor to the rural damages. This indicates that further investigation of the assumptions is required, if it is considered important to the outcomes of the study, or perhaps that, in the long-term, the viability of banana plantations on the floodplain should be investigated.

Table 6-3 Banana Cost Breakdown

Item	Rate
Typical Sell Price at Market	$14.50/carton
Average Freight Cost	$2.00/carton
Carton	$2.00/carton
Harvesting & Packing	$2.00/carton
Growing Costs	$6.50/carton
Margin	$2.00/carton

6.3.1.4 Beef Grazing

WBM (2001) and PBP (1995) provides a description of the effects of flooding on beef grazing in the Mid-Richmond valley in Northern NSW and the steps used to develop damage estimates per hectare of farmland. This methodology has been applied to this analysis.

The data determined from interviews in the Mid-Richmond was the most variable and uncertain of all of the data gathered for that investigation. Nonetheless, there was a consensus that major floods caused significant damage and resulted in financial loss.

The major flood damage associated with flooding of beef grazing pastures is the cost of moving cattle before a flood, the cost of agistment, and the loss incurred in selling animals prematurely.

The results of the investigation showed that the flood damage per hectare varied between $17 and $360 for specific floods. The estimated flood damage for a typical major flood was $210/ha. Since this estimate and the average flood damage for one recent flood in the Mid-Richmond (ie. April 1989) are similar, and less likely to be influenced by variations in grazier's memories and CPI fluctuations, a flood damage of $200/ha ($1994) was assumed for this analysis and converted to $240 ($2002).

6.3.1.5 Summary - Rural Damages

Using the inundation, land use and unit damages for each landuse described above, total damages for each ARI event were determined by summing the predicted damages for each individual landuse. The damages for each flood event are presented in Table 6-4 and as a probability-damages curve in Figure 6-1. It was assumed that zero damages occur in a 1 year ARI event. To calculate an average annual damage (AAD), the full range of floods up to the probable maximum flood (PMF) is required. The largest flood considered in this study is the 100 year ARI event and so a "correct" AAD can't be calculated. With this limitation, an AAD for rural land use of $1.6 million was estimated by calculating the area under the probability-damages curve. These results are presented in Table 6-5.

The data in Table 6-4 shows that the total damage is dominated by damages to banana crop. As was noted in Section 6.3.1.3, the analysis of damages to the banana crops is based on preliminary data and can be considered preliminary only. Further investigation would be required to improve the estimate of flood damages to bananas.

Table 6-4 Rural Flood Damages per Land Use

Flood Event (years ARI)	Damages per Landuse ($2002)			
	Sugar Cane ($)	Banana ($)	Beef ($)	Total ($)
100	1,003,000	7,790,000	350,000	9,143,000
50	819,000	6,175,000	334,000	7,327,000
20	581,000	4,028,000	314,000	4,924,000
10	418,000	2,812,000	290,000	3,520,000
5	305,000	1,843,000	262,000	2,409,000
2	167,000	513,000	202,000	882,000

Table 6-5 Rural Flood Damage - Total

Flood Event (years ARI)	Annual Exceedence Probability	Existing Case ($2002)	
		Total Damages	Incremental Area Under Probability-Damage Graph
100	1%	$9,143,000	
50	2%	$7,327,000	$82,000
20	5%	$4,924,000	$184,000
10	10%	$3,520,000	$211,000
5	20%	$2,409,000	$296,000
2	50%	$882,000	$494,000
1	99%	$0	$216,000
Average Annual Damage (excl. floods > 100 year ARI)			**$1,483,000**

6.3.2 Urban Damages

Urban damages in the Johnstone Rivers system are concentrated in the Innisfail region and Mourilyan. However, this analysis also includes damage to residential properties outside of these townships such as smaller communities and farm houses.

The damage to urban areas is principally to property and can be categorised into residential, commercial and industrial sectors. The derivation of urban damages has utilised stage-damage relationships developed over the last 20 years from other floodplain management studies and research (Smith, 1994 and EM, 1999)

A basic procedure for these calculations used is provided below.

- Identify the areas inundated and the depth of inundation for the range of design flood events (2, 5, 10, 20, 100 year ARI) modelled using the TUFLOW hydraulic model.
- Determine the damages due to a particular flood event using the assumed floor levels of dwellings which are potentially flood-affected.
- Calculate the depth of flooding within each dwelling for each ARI events.
- Prepare stage-damage relationships for residential and commercial properties. These relationships will account for such factors as the relative degree of flood preparedness of the community.
- Produce total flood damages for the range of flood events for both residential and commercial/industrial properties.
- Sum damages for all dwellings for each ARI event and present the results in a probability-damage graph.
- Determine the AAD.

6.3.2.1 Floor Levels

In order to determine damages due to flooding, it is necessary to firstly determine at what level floodwaters are able to enter buildings. Floor level survey data is not directly available. Therefore, the floor levels were assumed to be the ground level at the house plus 0.5 m. A GIS layer was created that identified each habitable property, as could best be determined from the aerial photography, and a ground level that was obtained from the DEM at the house.

During the course of the study, the Steering Committee considered that a house raising analysis should be investigated further using an improved estimate of floor levels. Therefore, an estimate of the floor level of all urban residential properties previously assumed to be inundated in a 100 year ARI flood was undertaken by estimating the height of the floor above the ground level; the estimate was a visual assessment from the road corridor.

Floor levels were used in conjunction with the predicted flood levels for each ARI event to determine whether floodwaters enter the building and, if so, to what depth.

6.3.2.2 Stage-Damage Relationship

Stage-damage relationships (or "curves") are used to determine the flood damage sustained by a particular property based on the depth of flooding ("stage" is another way of referring to depth). For example, if floodwaters entered a house to a depth of say 1m, the stage-damage curves would be used to determine the average damage in dollars that water of depth 1m would cause. Similarly, if floodwaters entered a shop to a depth of say 0.5m, stage-damage curves would be used to calculate the average damage in dollars that 0.5m of water in a shop would cause.

As explained in Section 6.1, total damages consist of direct and indirect costs. Direct damages include damage to the actual building and damage to contents (such as carpets and televisions in the case of a residential property or stock in the case of a commercial property). Indirect costs include loss of business due to time for floodwaters to subside and for cleaning up to be completed. In this investigation, total damages have been used.

Stage-damage curves are **critical** in the calculation of damages and benefit-cost ratios. The derivation of these curves is a complex and time-consuming process. It requires surveys to be undertaken of houses, businesses and contents in the region to determine the relationship between depth of flooding and potential damage. Surveys of this type allow the development of **potential** stage-damage curves. Potential curves represent the maximum damage that would occur if there was no action by residents to move material items out of reach of floodwaters. As residents usually do take some action in times of flood, **actual** damages are usually less than potential damages. The amount by which actual damages are less than potential is a function of warning time, flood preparedness and depth of flooding. For example, with no warning time a resident would be unable to move many belongings to a higher area but the number of belongings moved to a safe position would increase with the increase in warning time. Alternatively, a resident who is unaware and thus unprepared for flooding may not move any belongings regardless of warning time as they do not realise that they are threatened. Smith (1994) has developed a graph showing the relationship between these factors and the ratio between actual damages and potential damages. This graph is reproduced in Figure 6-2.

Stage-damage relationships for this study were adopted from curves contained within Working Paper 8 of the Lismore Levee Scheme EIS investigation (EM, 1999) and WBM (2002). However, it is important to note that EM (1999) questions the accuracy of the Lismore stage-damage curves. EM (1999) states *"...the original stage-damage curves for Lismore were developed in 1979, and there has been no check on their accuracy. Therefore, the results that arise from these stage-damage curves may be inaccurate and not reflect the true situation. In particular, the commercial stage-damage curves have always been suspect because of the high level of preparedness of the...businesses in Lismore, the stage-damage curves could be high by a factor of at least 3."*

A comparison of several potential commercial stage-damage curves from varying sources with those from Lismore revealed that the Lismore curves are considerable higher than others. In WBM (2002) the Lismore commercial damage curves were modified to account for the increased warning time in the Mid-Richmond region resulting in a reduction in commercial damages. These curves from WBM (2002) were adopted for the Johnstone River. However, the warning provided by EM (1999) is still applicable and until such time as more accurate curves are developed, results should be treated with caution.

Residential Curves

For the purposes of this investigation, the residential stage-damage curves are assumed to be the same as those used by EM (1999). As these curves do not account for damages below flood level, damages sustained to gardens, garden equipment and storages below floor level are assumed to increase linearly from zero at ground level to $1000 at floor level. This is the same assumption used by EM (1999).

Different damages curves are available for one and two storey houses, the condition of the house (poor, fair and medium) and the preparedness of residents. Detailed data of the houses is not available, so all houses are assumed to be one storey and in fair condition. Unprepared curves are used for the 50 and 100 year ARI and PMF flood events. Although a PMF event has not be analysed for the study, for the purposes of calculating the annual average damages, a PMF flood level of the 100 year ARI flood level plus 2 m was adopted. The basis for this assumption was estimated flood levels in Innisfail for historical floods. For the 20, 10, 5 and 2 year ARI events, it is assumed that residents will be prepared and hence the prepared curves apply.

Commercial/Industrial Curves

As in EM (1999), the same set of curves has been used for commercial and industrial properties. The curves in EM (1999) were derived from two sources:

- Unprepared curves were derived from Sydney curves originally developed by Smith et. al. (1990).
- Prepared curves were derived from the Lismore curves originally developed by Smith et. al. (1979).

These curves were revised for the Johnstone River Flood Study using the same procedure that was used for the Mid-Richmond Floodplain Risk Management Study (WBM Oceanics, 2002).

To assess commercial damages these stage-damage curves are used in a similar manner to the residential curves in that it is assumed that residents will be unprepared for design events equal to or larger than the 50 year design flood. Thus, unprepared curves are used for the 100 year and PMF flood events. For the 20, 10, 5 and 2 year events, it is assumed that residents will be prepared, and hence, the prepared curves apply.

The commercial damages curves categorise buildings into small, medium and large and also require the number of storeys and condition of the building. Commercial properties with a floor area > 1000 m^2 were assumed to be large. The floor area of larger commercial properties was assumed to be similar to the roof area which was measured from the aerial photography. All other commercial properties were assumed to be of medium size. All buildings were assumed to one storey and in fair condition for this analysis due to a lack of data to allow definition into various types.

6.3.2.3 Damages

The peak depth of flooding was determined at each dwelling for the 2, 5, 10 20, 50 and 100 year ARI and PMF event and the associated cost extracted from the stage-damage relationships. Total damages for each flood event were determined by summing the predicted damages for each individual dwelling. If floodwaters did not enter a particular dwelling but inundated a portion of the property, damages to the grounds of the property was assumed to increase linearly from zero at ground level to $1000 at floor level as explained previously. This is a nominal amount representing costs due to damage of gardens, sheds and other items. EM (1999) also used this amount in Lismore Levee investigations.

As was noted in Section 6.3.2.1, an improved estimate of urban residential floor levels was undertaken during the course of the study. The base damages were recalculated using the revised floor level estimates and used in all economic analyses after that time. Economic analyses undertaken prior to obtaining the improved data were not recalculated. Therefore, both sets of base case damages data are presented here.

The residential/commercial probability-damages curves for both floor level assumptions are presented in Figure 6-3. Average annual damages were then determined by calculating the area under this curve. The total damages for each flood event and the calculation of the AAD are presented in Table 6-6 and Table 6-7. The revised floor level estimate reduced the AAD from about $3.6 million to $3.0 million. The breakdown of the damages into residential and commercial given in Table 6-8 shows that the damage to commercial properties is significantly higher than to residential properties.

It should still be recognised that the urban residential floor levels are still an estimate and that the damages calculations for rural residential and commercial properties are still based on the assumption that the floor level is 0.5 m above the level in the DEM.

Table 6-6 Flood Damages Using Preliminary Floor Level Estimate (Resid/Comm)

Flood Event (years ARI)	Annual Exceedance Probability	Existing Case ($2002)	
		Total Damages	Incremental Area Under Probability-Damage Graph
PMF*	0%	$227,000,000	
100	1%	$97,100,000	$1,620,000
50	2%	$53,930,000	$755,000
20	5%	$6,320,000	$904,000
10	10%	$1,700,000	$201,000
5	20%	$400,000	$105,000
2	50%	$11,400	$61,700
1	99%	$0	$2,900
Average Annual Damage			**$3,650,000**

* A PMF (probable maximum flood) was not modelled. The total damages estimate for the PMF was calculated assuming a flood level 2 m higher than the 100 year ARI flood level. Neither the damages estimate nor the flood level assumption should be quoted.

Table 6-7 Flood Damages Using Revised Floor Level Estimate (Resid/Comm)

Flood Event (years ARI)	Annual Exceedance Probability	Existing Case ($2002)	
		Total Damages	Incremental Area Under Probability-Damage Graph
PMF*	0%	$210,000,000	
100	1%	$83,100,000	$1,460,000
50	2%	$44,800,000	$640,000
20	5%	$3,800,000	$728,000
10	10%	$860,000	$116,000
5	20%	$200,000	$53,300
2	50%	$6,000	$31,200
1	99%	0	$1,500
Average Annual Damage			**$3,030,000**

* A PMF (probable maximum flood) was not modelled. The total damages estimate for the PMF was calculated assuming a flood level 2 m higher than the 100 year ARI flood level. Neither the damages estimate nor the flood level assumption should be quoted.

Table 6-8 Flood Damages per Property Type

Flood Event (years ARI)	Annual Exceedance Probability	Existing Case ($2002)	
		Commercial	Residential
PMF*	0%	$150,000,000	$60,000,000
100	1%	$65,000,000	$18,000,000
50	2%	$35,000,000	$10,000,000
20	5%	$2,400,000	$1,400,000
10	10%	$585,000	$275,000
5	20%	$133,000	$67,000
2	50%	$4,500	$1,500
1	99%	0	$0

* A PMF (probable maximum flood) was not modelled. The total damages estimate for the PMF was calculated assuming a flood level 2 m higher than the 100 year ARI flood level. Neither the damages estimate nor the flood level assumption should be quoted.

6.4 Infrastructure Damage

Infrastructure damages includes damages to telephone, electricity, roads, rail, flood structures and other public utilities. This study has reviewed historical flood damages costs for public utilities through liaison with JSC, JSRIT, Bundaberg Sugar, Telstra, Ergon and the DMR to obtain an indication of the magnitude of infrastructure damages incurred during flooding.

6.4.1 JSC and JSRIT

The JSC and JSRIT advised that the following damages were incurred for the February 1999 and March 2000 flood events:

- JSC Roads/Bridges/Parks - $1,633,261
- River Trust Assets - $513,271

6.4.2 Bundaberg Sugar

Bundaberg Sugar infrastructure on the floodplain includes the rail network for the trams. Numerous attempts to contact representatives from Bundaberg Sugar during the course of the study were unsuccessful, and hence details of damages to their network were not obtained.

6.4.3 Telstra

Telstra is responsible for landline and some mobile phone communications assets through the Johnstone River study area. Discussions were held with Telstra personnel (Anthony Pezutto) to assess flood damages data and flooding issues. The main assets that are affected by flooding are underground cables. The cables are waterproofed, but lightening strikes can create pinholes that only become evident during flooding resulting in some disruption to services. The Innisfail exchange in

Rankin Street has all electrical equipment on the second floor to reduce the risk of flood damage. The main problem during flooding is access, although Telstra receives good support from the CDC and SES. Infrastructure flood damages were not available specifically for the study area.

6.4.4 Ergon

Ergon is responsible for the assets required for provision of electricity through the Johnstone River Study area. Mr Lex Boothby from Ergon was contacted to discuss the impact of flooding on Ergon's infrastructure. Ergon does not record the monetary cost of flood damages to its infrastructure. However, the following points were noted.

- The major difficulty during a flood is gaining access to the infrastructure. On occasions this can be expensive with access using helicopters required.
- During some large flood events there has been damage, probably from debris, to aerial conductors that cross the river.
- Damage to meter boxes is sustained during some floods.
- The major control rooms are at Pigeon Hill and in Innisfail CBD. The Pigeon Hill control room is too high to be affected by flooding. In Innisfail, Ergon advised that important infrastructure is built above flood level, but could not provide that level.
- Poles are not lost through washout.

It was concluded from the discussion with Mr Boothby that historically the damages to Ergon infrastructure has not been significant.

6.4.5 DMR

Maintenance of DMR roads within Johnstone Shire Council is undertaken by DMR. The data supplied by DMR on flood damages to their road network within the study area is given in Table 6-9. Damages to the Palmerston Highway are most likely to be from local catchment flooding. The northern section of the Innisfail-Japoon Road is within the study area, but the proportion of the damages given in the table that fall within the study area is not known. In the context of other damages within the study area, the damages to the DMR road network is not substantial. For example, the damages in the February 1999 flood were probably in the range $50,000 to $100,000. DMR advised that they have recognised the impact of flooding on their road network and are implementing over time, measures to reduce flood damages.

Table 6-9 Historical Flood Damages to DMR Infrastructure

Road	Submergence Damage	Saturation Damage
February 1999		
Palmerston Highway	$ 15,197	
Innisfail – Japoon Road	$ 118,493	
March 1999		
Palmerston Highway	$ 20,539	
Innisfail – Japoon Road	$ 58,897	
February 2000 & March 2000		
Palmerston Highway	$ 92,515	$ 99,952
Innisfail – Japoon Road	$ 104,333	
South Johnstone Road	$ 2,397	
November 2000 and February 2001		
Palmerston Highway	$ 4,447	
Innisfail – Japoon Road	$ 34,268	

6.5 Intangible Damages

There are a number of intangible costs of flooding to the community including the following:

- loss of life and limb;
- preparedness (cost of flood warning, planning, community education);
- inconvenience;
- isolation/evacuation;
- stress and anxiety;
- disruption;
- health issues.

These intangible damages are not easily quantifiable and have not been included in the monetary assessment of flood damages. However, they are discussed in relation to each management measure assessed within this study.

6.6 Total Damages

Total flood damage is found by summing the calculated damages for each of the damage types reviewed in the previous sections. Infrastructure and intangible damages are not included in this calculation.

As noted earlier, the full range of floods up to the probable maximum flood (PMF) is required to calculate the AAD. The largest flood considered in this study is the 100 year ARI event and so a "correct" AAD cannot be calculated. However, an approximation was made for the residential/commercial damages in a PMF and included in the calculation of the average annual damages for residential and commercial properties (refer in Section 6.3.2.3). The rural damages in a PMF were not calculated. The total damages excluding the PMF damages are presented in Table 6-10 and including the PMF residential and commercial damages are presented in Table 6-11. The AAD excluding floods greater than the 100 year ARI is about $3 million and with the inclusion of an approximation to the residential and commercial PMF damages, the AAD is about $4.5 million. To properly calculate the AAD, the calculation of the PMF flood extent and height needs to be improved and the rural damages included. From the calculations done within the limitations of this study, it could be reasonably assumed that the AAD for the study area is > $4.5 million, excluding damages to public infrastructure and intangible damages.

Apart from the lack of modelled flood data for events >100 year ARI, the other significant uncertainties in the calculation of the AAD are the banana damages and the floor level assumptions. Importantly, these uncertainties will have no significant influence on the findings of the study because the AAD is primarily used for the economic assessment of flood management measures. In these assessments, the change in AAD as a result of the implementation of the measure is of interest, not the absolute AAD.

Table 6-10 Total Flood Damage (excl floods >100 Year ARI)

Flood Event (years ARI)	Annual Exceedence Probability	Existing Case ($2002)	
		Total Damages+	Incremental Area Under Probability-Damage Graph
100	1%	$92,243,000	
50	2%	$52,127,000	$721,850
20	5%	$8,724,000	$912,765
10	10%	$4,380,000	$327,600
5	20%	$2,609,000	$349,450
2	50%	$888,000	$524,550
1	99%	$0	$222,000
Average Annual Damage (excl. floods > 100 year ARI)			**$3M**

+ Excluding infrastructure and intangible damages

Table 6-11 Total Flood Damage (PMF estimated)

Flood Event (years ARI)	Annual Exceedence Probability	Existing Case ($2002)	
		Total Damages[+]	Incremental Area Under Probability-Damage Graph
PMF	0%	$210,000,000	
100	1%	$92,243,000	$1,511,000
50	2%	$52,127,000	$721,850
20	5%	$8,724,000	$912,765
10	10%	$4,380,000	$327,600
5	20%	$2,609,000	$349,450
2	50%	$888,000	$524,550
1	99%	$0	$222,000
Average Annual Damage			**$4.5M**

[+] Excluding infrastructure and intangible damages

[*] A PMF (probable maximum flood) was not modelled. The total damages estimate for the PMF was calculated assuming a flood level 2 m higher than the 100 year ARI flood level. Neither the damages estimate nor the flood level assumption should be quoted. The total damages figure for the PMF does not include rural damages.

7 HISTORICAL FLOODPLAIN WORKS ASSESSMENT

As on most floodplains, there have been changes, both natural and man-made, to the Johnstone River floodplain. The community is concerned that some of these changes may be altering the flooding characteristics of the floodplain. A desktop review of concerns raised by both the community and the Steering Committee was undertaken. Following this review, the Steering Committee selected a number of works to be investigated using the flood model. The desktop review is first presented followed by the detailed assessment using the flood model.

7.1 Desktop Review

The community and the Steering Committee raised concerns relating to the following floodplain works.

- Construction of Carello's levee
- Construction and operation of the floodgates on Saltwater Creek, Sweeneys Creek and minor drains
- Filling of the town swamp
- Realignment of Ninds Creek and other issues
- Raising of Coquette Point Road
- Vegetation of Saltwater Creek
- Sediment build up in Gladys Inlet
- Local drainage issues
- Raising of Bruce Highway at Mourilyan

7.1.1 Carello's Levee

Carrllo's Levee is located on the left bank of the Johnstone River downstream of the confluence of the North and South Johnstone Rivers. It is approximately 1.6 km long and abuts agricultural properties predominantly used for sugarcane farming. The construction of this levee has been a topic of discussion in Innisfail over many years. The discussion has predominantly related to whether or not the levee increases flood levels in the low lying residential areas of East Innisfail and Webb.

The Committee's decision was to investigate the impact of Carello's levee on flood levels.

7.1.2 Floodgates

Floodgates in the Innisfail area mainly consist of one-way culverts or gates and an embankment. The culverts are fitted with floodgates to allow discharge of the local stormwater runoff from the upstream side and to prevent encroachment of tidal and floodwater into the local area behind the floodgates.

The floodgates reduce the impact of flooding in Innisfail, especially in smaller flood events, but they might increase flood levels on other parts of the floodplain.

The Committee's decision was to investigate the impact of removing the floodgates and levees to demonstrate the benefits and disbenefits of the structures.

7.1.3 Filling of the Town Swamp

The filling of the town swamp has reduced the flood storage in the town area. A reduction in flood storage without compensatory works may increase flood levels. The loss of storage is not significant in the context of the total storage in the floodplain. However, this is an issue of concern in the community and would remain a concern if an assessment using the model was not undertaken.

The Committee's decision was to investigate the impact of the filling of the town swamp on flood levels.

7.1.4 Ninds Creek Realignment

The study brief states that there have been "*drastic changes which have occurred to both the alignment and cross-section of Ninds Creek in the Etty Bay Road Area*". During Johnstone River flooding, water flows back up Ninds Creek or in larger floods it breaks across at Mourilyan and into Ninds Creek. In either case, the Ninds Creek catchment is effectively a large flood storage. Therefore, the realignment of the creek will have no significant impact on flood levels in a Johnstone River flood.

However, the realignment and cross-section changes may impact on flood levels and velocities during a local catchment flood that is not influenced by backwater from the Johnstone River. If the realignment is a straightening of the creek, the creek will become more efficient hydraulically, unless energy dissipation measures are incorporated into the realignment, resulting in a decrease in flood levels upstream of and through the realignment and possibly an increase in velocity. There may be an increase in flood levels downstream of the realignment. The flood levels associated with a local catchment flood are not likely to be significant when compared to flooding from the Johnstone River. Therefore, an increase in flood levels during a local catchment flood as a result of the changes to the channel are unlikely to be significant in the context of impacts on property. Separate from the flooding issues, channel realignment can result in increased scouring of the banks and beds if velocities are increased.

During the resident survey, concerns over vegetation removal and draining of swamps in the Ninds Creek catchment were raised, the main concerns being related to environmental issues that are beyond the scope of this study. Removal of vegetation can impact on flood levels during local catchment events in a similar manner to the creek realignment as discussed previously, but will not impact significantly on Johnstone River flooding. If swamps are drained and then land filled, there may be some impact on both local catchment and Johnstone River floods due to the lost storage, although the impacts would only be minor unless the loss of storage is significant in the context of the Ninds Creek catchment.

The Committee's decision was to not investigate the impact on flooding of realignment, vegetation removal and land filling in Ninds Creek.

7.1.5 Raising of Coquette Point Road

Concern has been raised that the Coquette Point Road has been raised in the vicinity of Ninds Creek without a compensatory increase in through drainage. If raising of Coquette Point Road has reduced the interchange of flow between the Johnstone River and the Ninds Creek catchment, it may have had some impact on flood levels. In Johnstone River flooding there would possibly be small decreases in flood level in Ninds Creek, mainly during the rising flood, and small increases in the Johnstone River. During local catchment flooding, raising the road may increase flood levels in Ninds Creek upstream of the road, although as commented previously, an increase in flood levels during a local catchment flood are unlikely to be significant in the context of impact on property.

The Committee's decision was to not investigate the impact on flooding of raising Coquette Point Road.

7.1.6 Saltwater Creek

Concern has been raised that the vegetation in Saltwater Creek is overgrown, and as a result, additional water is being pushed south to the area of the old town swamp rather than flowing out through Saltwater Creek into the Johnstone River. During Johnstone River flooding, the Saltwater Creek floodgates will be shut thereby preventing any flow from Saltwater Creek into the Johnstone River. Therefore, under these flooding conditions, clearing of the vegetation in the creek is unlikely to significantly reduce the flow to the old town swamp.

During a significant local catchment event during which the level of the Johnstone River does not prevent the opening of the floodgates, clearing of the vegetation may reduce flows to the town swamp if a local catchment event can produce a large enough flow to cause a breakout to the old town swamp.

The local catchment scenario could be modelled although the results will be indicative only because of the level of detail of Saltwater Creek in the model.

The Committee's decision was to not investigate the impact on flooding of vegetation removal in Saltwater Creek.

7.1.7 Sediment Build-up in the Johnstone Rivers and Gladys Inlet

Anecdotal evidence indicates that sediment is aggregating in parts of the Johnstone River and Gladys Inlet resulting in a rise of the bed level of the river. The resultant reduction in the flow carrying capacity of the main channel of the rivers can worsen overbank flooding. The commercial boating industry has also expressed concerns that the build up of sediment may, in the long-term, have an impact on the viability of the industry through a loss of a navigable channel.

The Committee's decision was to investigate the impact on flooding of channel dredging.

Although sediment aggregation is an historical change to the river system, this investigation will be reported in Section 8, (Flood Modification Measures), as the assessment will be a channel dredging option rather than an investigation of the impact of a known build-up of sediment.

7.1.8 Local Drainage Issues

There have been concerns raised by the community relating to the drainage system in local storm events. The flood model has not been set up with sufficient detail to accurately model the local drainage system during local rainfall events. Therefore, any assessment would be indicative only.

The Committee's decision was to not investigate the impact on local flooding of changes to the local drainage system.

7.1.9 Raising of Bruce Highway at Mourilyan

In larger floods the South Johnstone River can break its banks and flow across the Bruce Highway near Mourilyan. Local residents, the Callidonis', expressed concerns that the Bruce Highway was raised at this location without sufficient drainage structures to compensate for the loss of waterway area over the highway. As a result, flood levels on the Johnstone River side of the highway might have increased.

If the highway has been raised without the addition of sufficient culverts, there will be an increase in flood levels on the Johnstone River side up to and just over the point of overtopping of the highway. On the downstream side, the flood levels will typically be lower and the flow can be more concentrated, sometimes resulting in scouring if sufficient outlet protection is not provided.

The Committee's decision was to investigate the impact on flooding of raising the Highway.

7.1.10 Summary

A summary of the Steering Committee's decision relating to the modelling of past development works is given in Table 7-1.

Table 7-1 Summary of Steering Committee Decisions on Assessment of Past Works

Issue	Steering Committee's Decision
Carello's Levee	Yes
Floodgates	Yes
Filling of Town Swamp	Yes
Ninds Creek Realignment and other issues	No
Raising of Coquette Point Road	No
Saltwater Creek Vegetation	No
Sediment Aggregation	Yes – as a management option
Local Drainage Issues	No
Bruce Highway Raising at Mourilyan	Yes

7.2 Detailed Model Assessment

7.2.1 Carello's Levee

The impact of the removal of the levee was assessed using the February 1999 flood and the 100 year ARI design flood. The bank levels were lowered as shown in Figure 7-1. After the analysis was completed, it was found that the photogrammetry levels of the ground to the north of the levee at the eastern end were high. As this potentially influenced this analysis, the levels were corrected and the analysis was re-run using only the 100 year ARI flood. It was found that the incorrect ground levels did not significantly alter the analysis, but only the 100 year analysis is presented.

The impact of removing the levee on flood levels was assessed at the flood peak and while the flood was rising. The latter was done to give an indication of the impacts during smaller floods.

The changes in flood level are presented in Figure 7-2 and Figure 7-3. Overall, removing the levee did not significantly reduce flood levels. For example, the peak 100 year ARI flood level in Webb would be reduced by about 10 mm if the levee was removed. At 19 hours into the 100 year ARI flood, ie, as the flood is rising, the flood level in Webb would be reduced by about 20 mm and in Innisfail by about 10 mm if the levee was removed. This gives an indication of the size of reductions that would be expected in smaller floods. In all cases there are increases in water level downstream of the levee as result of its removal. This is expected given the increase in flow across this area.

7.2.2 Floodgates

The six floodgates included in the model are shown on Figure 7-4; Saltwater Creek (F2); Sweeneys Creek (F4); trunk town drain (F3); drain south of Forrest Island (F1); drain north of Mundoo (F5); and drain at northern end of Carello's property (F6). The culverts and associated embankments of the four major floodgates (F1, F2, F4 and F6) were removed from the hydraulic model to assess their impact on flooding. The impact of the removal of the floodgates was assessed using the February 1999 flood and the 5 year and 50 year ARI design floods.

Figure 7-4 shows that flooding in the Innisfail township area would have been worse in February 1999 flood without the floodgates with the peak flood level about 170 mm higher. The maximum impact behind the floodgate near Forrest Island (F1) is approximately 30 mm.

In a 5 year ARI event, the peak 5 year ARI flood level in the Innisfail township area would be approximately 380 mm higher (Figure 7-5) without the floodgates in place. The 50 year ARI peak flood levels would be about 10 mm higher as shown in Figure 7-6. The floodgates provide little benefit at the peak of larger flood events because the floodgate embankments are significantly overtopped. However, they would still provide benefit during the rise of these larger floods thereby effectively giving residents additional preparation time. Figure 7-7 shows that at 19 hours into the 50 year ARI event, the removal of the floodgates would increase the flood level by 460 mm in Innisfail area, or alternatively, having the floodgates in place would reduce the flood level by 460 mm at 19 hours.

7.2.3 Removal of Fill in Town Swamp

The town swamp used to be a low lying area located at the west of Innisfail township as shown in Figure 7-8. This area has been filled resulting in a reduction in the flood storage. A reduction in flood storage without compensatory works has potential to increase flood levels in the area adjacent to the filled area.

The ground level prior to the filling of swamp was not readily available. After discussions with JSC, Will Higgins from JSRIT advised that the existing surface level of RL 2.6 m AHD at the boundary of this area approximately represents the original levels of the town swamp. Based on this assumption, the impact of removing fill in the town swamp was assessed using the February 1999 historical flood and the 50 year ARI design event.

Figure 7-8 shows that the February 1999 peak flood levels would have been about 60 mm lower around the Innisfail township area if the town swamp had not been filled. In a 50 year ARI flood, the peak flood level would be approximately a 50 mm lower in the area south of the town swamp (Figure 7-9). The impact is similar during the rising of the flood as shown at 33 hours into the 50 year ARI event in Figure 7-10. However, at this time there is also a minor increase in the flood level in the area north of the town swamp, possibly because lowering the swamp as increased the flow to the north.

7.2.4 Impact of Raising Bruce Hwy at Mourilyan

The Department of Main Roads upgraded a section of Bruce Highway at Mourilyan sometime in the last 10 to 20 years. As part of the upgrade, the road grade of the Highway was raised 300 mm to 500 mm on the advice of Alan Dunne from JSRIT. There are a number of houses and a sugar mill located on the floodplain between the Highway and the South Johnstone River. Residents from this area are concerned that the upgrade of the Highway has worsened flooding in this area.

Hydraulic modelling was undertaken to assess the impact of the Bruce Highway upgrade. The configuration of the old Highway was approximated in the model by making the following changes:

- Highway levels were reduced by 0.3m to 0.5m as shown in Figure 7-11; and
- Culverts across the Highway in this section were excluded.

Impacts of the above mentioned changes to Bruce Highway were assessed using the 100 year ARI flood. There is no significant change in peak 100 year ARI flood levels, as shown in Figure 7-12, because the highway is substantially submerged. The greatest impact was found to be at 28 hours into the 100 year ARI flood (refer Figure 7-13), which is approximately equivalent to the peak in a 50 year ARI flood in the South Johnstone River at Mourilyan. With the highway lowered, the flood level at this time is typically 100 mm to 300 mm lower on the river side of the highway and up to about 250 mm higher on the eastern side of the highway. These impacts are generally localised around the highway.

8 FLOOD MODIFICATION MEASURES ASSESSMENT

8.1 Assessment Process

Flood modification measures are designed to alter the behaviour of the flood itself by reducing flood levels and/or velocities, or by excluding floodwaters from areas at risk. They are also referred to as structural measures. The identification, analysis and recommendation of structural measures followed a 7 stage process:

1. Compilation of possible structural measures using input from the Steering Committee, Community and WBM;
2. Desktop review of measures by WBM;
3. Selection by the Steering Committee of measures for preliminary flood impact analysis;
4. Preliminary flood impact analysis by WBM;
5. Review by Steering Committee of preliminary flood impact analysis and short listing of measures for detailed assessment;
6. Detailed assessment of short listed measures by WBM;
7. Review by Steering Committee of detailed analysis and selection of measures for inclusion in the Floodplain Management Plan.

The preliminary flood impact analysis was undertaken using the TUFLOW hydraulic model and either the February 1999 historical flood or one of the design floods. The detailed analysis was also undertaken using the hydraulic model, but the impacts were assessed using all six design floods.

Table 8-1 is a summary of all the flood modification measures identified and the Steering Committee's decisions on the level of analysis and recommendations. Further details on both the preliminary analysis and the detailed analysis are provided in the remainder of this chapter.

Table 8-1 Flood Modification Measures & Steering Committee Decisions

Measure	Preliminary Analysis?	Detailed Analysis?	Recommended by SC?
Realignment of Carello's Levee	Yes	No	No
Channel at Carello's Levee (a) constructed channel (b) scoured channel;	 Yes No	 Yes Yes	 No No
Raise existing Saltwater and Sweeneys Creek floodgate levees	Yes	Yes	Yes
Webb Levee - along river bank from Corinda Street downstream	Yes	Yes	No
Levee around Webb as proposed in Cameron McNamara (1985)	No	No	No
River bank levee near Innisfail East State School	Yes	No	No
River bank levee near TAFE	No	No	No
Increase size of culverts at Crocodile Farm	No	No	No
Dredging options	Yes	Yes	No
Saltwater Creek devegetation	No	No	No
Increased cross-drainage along Coquette Point Road at Ninds Creek	No	No	No
Increase drainage capacity under Bruce Highway near Mourilyan	No	No	No
River levee bank to reduce/prevent breakout of river across to Mourilyan	No	No	No
Levee scheme as proposed by Cameron McNamara in 1985	Yes	No	No
Floodgate on Gracey Creek	No	No	No
Tabone diversion channel	Yes	No	No
Overflow channel into Ninds Creek and then to Mourilyan Harbour	No	No	No
Dam on North Johnstone	No	No	No

8.2 Desktop Review

Ideas for flood modification measures were sourced from the following:

- the Steering Committee via a brainstorming session undertaken at the meeting on 3 July 2002;
- the community via the open sessions held in Innisfail;
- WBM.

WBM undertook a review of these measures giving consideration to the tangible and intangible benefits. These comments along with WBM's recommendations for further analysis are summarised in Table 8-2.

Table 8-2 Summary of Desktop review

Management Measure	Comment
Modification to flow paths at Carello's Levee	The construction of a substantial channel across the bend is likely to reduce flood levels in the river upstream of the new flow path and increase flood levels in the river downstream. A reduction in flood level in the Webb area is expected, although it is not likely to be substantial. It is not likely to have any significant impact on flood levels in Innisfail given the current flood gradient from the bend to Innisfail. If this is the case, the benefit-cost analysis may indicate that the measure is not viable. It is likely to reduce the pressure on the bank on the outside of the bend, but may increase pressure on the bank further downstream. May reduce sediment build up locally through the provision of a more efficient channel, but may lead to an increase in sedimentation further upstream and downstream. ***Although it may transpire that the benefits do not outweigh the dis-benefits, it is recommended that modelling be undertaken given the strong community interest.***
Raise existing Saltwater and Sweeneys Creek levees at floodgates	These floodgates and levees currently reduce the flow into Innisfail during floods. Increasing the levee height will further reduce the flow resulting in a reduction in flood level. The cost for this measure would not be substantial. The measure is not likely to have significant impact on flood levels elsewhere on the floodplain. Given that the proposal is to increase the height of existing levees rather than the introduction of new floodgates, and that it only impacts at higher flows, it is not likely to have any significant environmental impact. ***It is recommended that this measure be modelled.***

8.3 Preliminary Analysis

The analyses of those measures for which only a preliminary analysis was undertaken are presented in this Section. Those measures for which a detailed analysis was undertaken are presented in Section 8.4. The Steering Committee's recommendation is given at the end of the discussion of each measure.

8.3.1 River Bank Levee near Innisfail East State School

The South Johnstone River overtops the river bank near the Innisfail East State School in about a 20 year ARI flood event flooding the lower areas of East and South Innisfail. A 100 year ARI levee was trialled along the river bank as shown in Figure 8-1, which also shows the impact of the levee on peak 100 year ARI flood levels. Some of the area protected by the levee still floods because of backwater from the Ninds creek area, but the flood levels are still reduced. The levee causes small increases of about 40 mm across the river and up into Innisfail.

The reductions in flood levels in East Innisfail are unlikely to provide any significant reduction in flood damages because in the low lying areas where the reductions are evident, the existing houses are mostly high set.

The Committee's decision is that the Innisfail East Levee should not be included in a Floodplain Management Scheme.

8.3.2 Carello's Levee realignment

Assessment of the removal of Carello's Levee was presented in Section 7.2.1. This section reports the assessment of modified configuration of the levee as shown in Figure 8-2. The modification included the removal of the eastern half of the levee and a realigned levee as shown in the inset of Figure 8-2. Lowering of a portion of land to the east of the realigned levee to RL 2 m AHD was also part of this measure. The overall objective was to "re-open" an overbank flow path of the Johnstone River.

The impact of this measure was assessed for the February 1999 flood. There were minimal impacts for both the peak level (Figure 8-2) and during rising of the flood (Figure 8-3).

The Committee's decision is that the Carello's Levee Realignment should not be included in a Floodplain Management Scheme.

8.3.3 Cameron McNamara (1985) Levee Scheme

Cameron McNamara (1985) proposed a levee system for the urban areas of Innisfail as shown in Figure 8-4. The impact of this levee scheme was assessed using the currently derived 100 year ARI design flood event. Figure 8-4 shows the impact of this levee scheme on the 100 year ARI peak flood levels. There would be a reduction in peak flood level of up to 450 mm in Innisfail township and approximately a 250 mm reduction in East Innisfail and the Ninds Creek catchment.

However, the proposed 1985 scheme would increase flood levels outside the area protected by the levee. The maximum increase in the peak 100 year ARI flood level to the north of Innisfail would be approximately 300 mm and to the south of Innisfail there would be increases of up to approximately 450 mm.

The Committee's decision is that the 1985 Levee should not be included in a Floodplain Management Scheme.

8.3.4 Tabone Diversion Channel

The intention of this channel was to reduce flood levels in Innisfail, Innisfail Estate and Webb by diverting flows from the North Johnstone River upstream of Innisfail to approximately Barney's Point as shown in Figure 8-5. It was recognised that there might be increases in flood levels in the river around the channel outlet and possibly in the floodplain through which it passes.

The impact of the diversion channel on peak 100 year ARI flood level is shown in Figure 8-5.

There would be a number of environmental issues that would require consideration including impacts on flora and fauna along the route of the channel and the local water table.

The diversion of flow out of a river system can impact on the sediment transport regime. It is likely that there would be a change in the pattern of aggregation and erosion in the North Johnstone downstream of the inlet into the new channel.

The Committee's decision is that the Tabone Diversion Channel should not be included in a Floodplain Management Scheme.

8.3.5 River Dredging – Scheme 1

A dredge area approximately 7 km long and 100 m wide from the Johnstone River mouth along the North Johnstone River was considered in this assessment. Dredging of an approximately 1 km of the South Johnstone River was also considered in this analysis. It was assumed that the river bed level would be lowered to RL –4.0 m AHD throughout the dredging area. Figure 8-6 shows the extent of dredging assumed in the analysis.

The impact of this assumed dredging on the February 1999 peak flood level is an overall reduction in flood levels as shown in Figure 8-6. The maximum decrease in the peak 1999 flood level is approximately 320 mm.

The Committee's decision is that a detailed analysis should be undertaken of a reduced river dredging scheme.

8.4 Detailed Analysis

The Steering Committee selected the following measures for detailed analysis:

- Constructed Carello's Channel
- Scoured Carello's Channel
- Raised Sweeneys Creek and Saltwater Creek Floodgate Levees
- Webb Levee
- River Dredging Scheme 2

The detailed analysis required that the measures be tested using all design floods. The investigation into the impact of the measures was not limited to change in flood height, but also included an assessment of the change in velocities that will occur if the measure is implemented and consideration of environmental matters. An economic analysis was undertaken to determine the Benefit-Cost Ratio (BCR) of the measures. The schemes were modelled independently so that the impacts of the measure could be determined without interference from another measure.

The flood impact analysis of each of the measures is presented and followed by the economic analysis along with environmental considerations.

8.5 Hydraulic Analysis

The impact of the flood modification measures on flood height and velocity were investigated using the 2, 5, 10, 20, 50 and 100 year ARI design floods.

8.5.1 Constructed Carello's Channel

This measure investigated the flood mitigation benefits of the construction of a 140 m wide channel with an invert of RL –2.0 m AHD across the corner of the bend in the river at the eastern end of Carello's property as shown in Figure 8-7. The hydraulic losses at a bend such as this are large and the construction of an additional waterway should improve the efficiency of the bend and reduce upstream flood levels.

The impact of the channel on peak flood levels for all design floods is shown in Figure 8-7 to Figure 8-12. The analysis indicates that the channel would result in widespread reductions in flood level extending up both rivers, into Innisfail and up through the Ninds Creek floodplain. The reductions in flood level are in the range 30 mm to 100 mm. The analysis indicates that there would be increases in flood level on Carello's property. The magnitude of the increases is largest in the 5 year ARI flood with the increases of more than 500 mm predicted in some parts of the property. In larger floods the magnitude is less, but the impacts extend over a larger area. However, these impacts could be alleviated through the construction of a levee along the western edge of the proposed channel.

The change in velocity for the 2 year, 20 year and 100 year ARI floods is shown in Figure 8-13 to Figure 8-15. Along the proposed channel there are increases in velocity as would be expected because this area is currently an overflow rather than a main channel. Around the existing bend there are reductions in velocity, also expected because the flow through this section of the river is reduced. There are localised increases in velocity of the order of 0.3 m/s along the banks immediately upstream and downstream of the proposed channel. Bank protection may be required to mitigate against these increases, especially upstream of the channel where the existing bank experiences erosion during flooding.

Some adjustment to the sediment regime in the existing channel would be expected as a result of the changes in velocity. Changes to tidal velocities would also be expected and these may also impact on the local sediment regime.

8.5.2 Scoured Carello's Channel

An alternative to constructing the Constructed Carello's Channel (refer Section 8.5.1) is to allow it to form through natural scouring rather than excavation resulting in construction cost savings. Nature would be assisted by removing the eastern end of Carello's levee and by excavating a "nick" at the bank down to RL 0.0 m AHD. Removal of the levee would require the construction of a new levee across what would become the new property boundary. Details of the proposal are shown in Figure 8-16. A separate hydraulic analysis of this proposal was not undertaken as was assumed for the purpose of the economic analysis that ultimately the channel will be of a similar size to the Constructed Carello's Channel.

8.5.3 Raised Sweeneys Creek and Saltwater Creek Floodgate Levees

During flooding, Innisfail acts as a storage basin with floodwater initially backing up through Saltwater and Sweeneys Creeks before there is widespread overtopping of the banks. These two creeks are currently floodgated, although the floodgates are overtopped in relatively small flood events. A review of the base case design floods indicated that the levees above the floodgates could be raised to a 20 year ARI flood height without a significant extension of the levee system.

The preliminary analysis using a 100 year ARI flood indicated that concept improved the flood protection of Innisfail without any significant negative impacts, except for increases of flood level by about 120 mm north of Innisfail in the floodplain to the east of Sundown Hill. There are some low lying houses in this area so a levee was included along a short section of Frith Road for the detailed analysis.

The impact of the proposed levees on the peak 5 year, 10 year, 20 year and 50 year ARI floods is shown in Figure 8-17 to Figure 8-20. The 2 year and 100 year ARI flood events are not presented because there is no significant change as a result of this measure on flood levels in these floods. Reductions in the peak flood level in Innisfail are evident in the 5 year, 10 year and 20 year ARI events, with the largest reductions of about 510 mm in the 20 year ARI event as would be expected. In the 10 year event there are increases in flood level in the range 100 mm to 300 mm to the south of the proposed levee at Frith Road because water is being pushed to the south whereas it previously flowed to the north. However, increases of this size are not considered significant in this area as it is agricultural land and the increases would be for a relatively short time.

To the south of Innisfail there are increases in flood level of about 30 mm in the 20 year ARI flood event over a broad area. In the southern section of the CBD to the north of Bamboo Creek, the increases are in the range 30 mm to 60 mm. In the 50 year ARI event, the proposed levees have no significant impact on flood levels in these areas.

The impact of the raising of the levees on the velocity at the peak of the 10 year, 20 year and 50 year ARI floods is shown in Figure 8-21 to Figure 8-23 respectively. In these figures there are isolated areas in the CBD where an increase in velocity is indicated. This occurs because the comparison is done at the peak of the flood when the flood level in town and in the existing case is higher than with the raised levees. Hence, the comparison is not for a similar flooding condition. It is unlikely that the peak velocity at these locations will have changed substantially. More importantly, along the banks of the North Johnstone River and Bamboo Creek there are no significant increases in velocity. Although not shown in these figures, it is likely that the local velocity at the levee will be higher than the existing case when the levees are overtopped in a flood event larger than a 20 year ARI event. This may result in some local scouring depending on the level of water behind the levee and duration of the increased velocities.

To reduce the impacts to the south of Innisfail, especially in the Jones Street area, a levee behind the houses on the western side of Jones Street was modelled to reduce the flow from the west. However, the levee increased the flood level in Jones Street at the times during the flood when the flow was from east to west. A levee parallel to and on the northern side of Scullen Avenue was then tried. The levee also included flap gated pipes to allow flow to the north. The impact of the scheme on peak 20 year ARI flood levels is shown in Figure 8-24. When compared to Figure 8-19, impacts in the 20 year ARI event without the Scullen Avenue levee, the levee appears to be reasonably successful in reducing the impact in the Bamboo Creek and Jones Street areas. However, the impacts have only reduced from about 30 mm to 27 mm, but this resulted in most this area being categorised in the ± 0.03 m band and hence shaded yellow on the figure. The Scullen Avenue levee is included in the scheme, although in Section 8.6.5 the need for the levee is questioned on economic grounds.

8.5.4 Webb Levee

This measure investigated the construction of a levee along the bank of the Johnstone River at Webb as shown in Figure 8-25. The crest level was set at approximately the 10 year ARI flood level following the preliminary investigation using the 100 year ARI flood that indicated a levee higher than the 10 year would most likely adversely impact in other areas of the floodplain.

The impact of the proposed levee on the peak 5 year, 10 year, 20 year and 50 year ARI floods is shown in Figure 8-25 to Figure 8-28. The 2 year and 100 year ARI flood events are not presented because there is no significant change as a result of this measure on peak flood levels in these floods. The largest impacts occur in the 10 year ARI flood. The levee does not prevent inundation of Webb in a 10 year ARI event because floodwaters can still flow around the western end of the proposed levee and also back-up from Ninds Creek, but there are some areas that would be fully protected in a 10 year ARI event. In those areas where water would still enter, the decrease in peak flood level is typically in the range 200 mm to 300 mm. In the 10 year ARI event there are small increases in peak flood level in the river of up to about 40 mm. In the Ninds Creek catchment to the south, the decrease in flood level is typically about 60 mm. In the 20 year and 50 year ARI floods the levee also reduces the peak flood level in Webb by 30 mm to 100 mm.

The impact of the raising of the levees on the velocity at the peak of the 5 year, 10 year and 20 year ARI floods is shown in Figure 8-29 to Figure 8-31 respectively. In each of the floods the velocity on the river bank downstream of the levee has increased. In areas where there are established mangroves, the increased velocity is not likely to have a significant impact on the stability of these banks. However, along sections of the bank that are not well vegetated, increases in velocity are likely to increase scouring. In the 10 year ARI event (Figure 8-30) the velocity also increases along the bank on the opposite side of the river at the bend. The increases are about 0.3 m/s. This section of the river bank is already under pressure, and an increase such as this may further destabilise the bank at this location.

8.5.5 River Dredging – Scheme 2

The preliminary analysis of a dredging scenario presented in Section 8.3.5 showed that dredging of the river reduced flood levels in the floodplain over a wide area. The 1999 flood was used for the preliminary assessment. Therefore, the steering committee elected to undertake a detailed assessment of dredging.

For the detailed analysis, the width of dredging was reduced from 100 m to 40 m, the channel was assumed to be dredged to RL –4.5 m AHD rather than RL –4.0 m AHD as was previously used, and no dredging was assumed in the South Johnstone River. The extent of dredging is shown in Figure 8-32. Some minor variation of the positioning of the dredge channel within the river cross-section from that shown in Figure 8-32 would not significantly alter the results presented in this report.

The impact of the proposed dredging scheme on the 2 year, 5 year, 10 year, 20 year, 50 year and 100 year ARI floods is shown in Figure 8-32 to Figure 8-37. There are widespread reductions in flood level in all events. The reductions are typically in the range 30 mm to 70 mm, although in the 10 year and 20 year ARI floods, there are reductions of up to about 120 mm in Innisfail. The flood height in town is dependent on the duration of flooding in the river as well as the peak height in the river. More specifically, the longer the duration of the flood in the river, the more time there is for flood water to overflow into Innisfail. Therefore, the benefit gained in Innisfail will vary from flood to flood, even if the same peak river height occurs.

Figure 8-38 to Figure 8-40 show that the dredging scenario will not significantly alter velocities.

8.6 Economic and Environmental Considerations

8.6.1 Background

Economic Considerations

In general, the benefits of the construction of flood modification measures are as follows:

- increased flood immunity of properties protected by the measure leading to ;
- increased flood immunity of roads protected by the measure and thus improved mobility of the community during flooding;
- decreased cost of flood damage to properties protected by the measure;
- decreased potential for loss of life during a flood event within the area protected by the measure;
- decreased emotional, social and psychological trauma experienced by residents in times of flooding.

It is important to note that flood modification measures can have the effect of increasing flood levels in other areas, thereby resulting in increased flood damages to properties elsewhere.

Of the factors listed above, the change in flood damages is the only one that can be easily quantified in monetary terms. In Section 6, the flood damages for the existing floodplain were calculated. The reductions (or increases) in these damages have been calculated to quantify the monetary benefit of each measure.

The overall financial viability of an option is initially assessed by calculating the monetary benefit-cost ratio (BCR). These ratios are used to evaluate the economic potential for the option to be undertaken. A monetary benefit-cost ratio of 1.0 indicates that the monetary benefits are equal to the monetary costs. A ratio greater than 1.0 indicates that the benefits are greater than the costs while a ratio less than 1.0 indicates that the costs are greater than the benefits. The change in infrastructure damage as result of implementing the measure is not included in the benefit-cost analysis.

In floodplain management, a BCR substantially less than 1.0 may still be considered viable because the economic analysis does not include the intangible benefits of a measure. For example, funding for flood mitigation works in Queensland is available through the Regional Flood Mitigation Programme administered by the Department of Natural Resources and Mines. Although the BCR is one of the criteria used for assessing applications to this fund, there is no minimum BCR requirement and funding has gone to projects with a BCR less than 0.5.

In order to calculate the BCR, the annual financial benefits (the change in average annual damages) of a measure needs to be converted to a total benefit over a period of time. This is due to the difficulty in comparing a "lump sum" cost with an "annual" benefit.

A financial project life of 50 years was chosen for this study. **This does not imply that the projected structural life of the scheme is only 50 years.** In fact, some measures should be effective in reducing the frequency of flooding for centuries to come.

It is **not** correct to simply multiply a long term average annual benefit by the financial project life of 50 years to derive a total worth of the benefits. To do so would ignore the important point that the benefits from this scheme (ie. reduced flood damages) will occur over time and in the future.

For example, a benefit of $2.3 million to be gained in 10 years time is not worth $2.3 million now but only $1.2 million now. This is because $1.2 million could be invested now and appreciate at say 7 % p.a. over and above inflation for 10 years. This would then be equivalent to $2.3 million in 10 years time. This is called the **Present Worth** of the benefit. It is a universally accepted economic theory and used in all major project economic analyses. The adopted rate of 7 % is called the discount rate and is the middle of the range 6 to 8 % recommended by the Queensland Government for assessing public works.

As an example, Table 8-3 shows the present worth of the annual benefit realised at different times over a 50 year period.

Table 8-3 Present Worth of Annual Benefits

Year	Annual Average Benefit ($ million)	Present Worth ($ million)
0	2.3	2.3
1	2.3	2.2
10	2.3	1.2
25	2.3	0.4
50	2.3	0.1

If the present worth benefits for each year are totalled for the 50 years, the total present worth (or total benefit) of the benefits is $ 31.7 million. The calculation of the total benefit can be simplified through the use of a Present Worth Factor. Rather than calculating the present worth for each year and summing to calculate the total benefit, a Present Worth Factor can be used when the annual average benefit is identical in each year. The Present Worth Factor is calculated using equation (1).

The Present Worth Factor is multiplied by the annual average benefit to calculate the total benefit. The Present Worth Factor is 13.8 for a 50 year period and a discount rate of 7%.

It is interesting to note that if a longer financial project life of say, 100 years was chosen then the total present worth of the benefits is only $1.1 million more at $32.8 million. This is due to the fact that the present worth of the benefits to be accrued in the second 50 year period is low because of the length of time until the benefits are realised.

$$\frac{\left[1-\left(\frac{1}{(1+i)^n}\right)\right]}{i} \tag{1}$$

where
n is the number of years
i is the discount rate(%)

The procedure for calculating benefit-cost ratios is outlined below:

1. Calculate the **average annual benefit** associated with the option (i.e. the reduction in annual average damages) using the method described in Section 6.3,

2. Convert the **average annual benefit** to a **total benefit** by multiplying by the **present worth factor***;

3. Calculate the **total cost** of the option.

4. Calculate the monetary **benefit-cost** ratio:

$$Benefit-Cost\ Ratio = \frac{Total\ Benefit}{Total\ Cost}$$

It is important to recognise that the monetary benefit-cost ratios represent only one of the issues that must be considered in respect to viability of an option. Other issues such as social and psychological impacts, although difficult to quantify, must be included in the complete assessment.

Benefit-cost ratios may be sensitive to variations and/or inaccuracies in the following:

- existing ground levels along a proposed levee route;
- proportion of a levee that would need to be a concrete wall levee and not an earth levee;
- proportion of the length of an earth levee that would require road reconstruction;
- construction, maintenance and operation costs;
- dredging quantities.

Environmental Considerations

There is a range of social and environmental issues associated with flood modification measures that may need to be addressed in an environmental impact study, should an option be implemented. These issues include:

- *Impacts on flood response and evacuations*

- *Impacts on riverbank stability*;
- *Public utility impacts* – for example, sewer routes may need to be revised.
- *Visual impacts and blockage of views* – Levees can have a detrimental impact on the visual aesthetics of an area. They can do this by blocking views or by visually spoiling a formerly attractive area.
- *Heritage and archaeology impacts*;
- *Impacts to traffic routes*; and
- *Impacts to fauna passage and flora*.

In evaluating the overall viability of an option, these issues need to be considered in conjunction with the benefit-cost ratios.

8.6.2 Unit Rate Costs of Works

These unit rate costs are from WBM (2002), SKM (1999) and industry sources. The rates are considered applicable in 2002.

Table 8-4 Unit Rates for Levee Construction Costs

Item		Unit	Rate
Earth Levee Construction			
- earthworks construction with the material obtained from within a 10km radius		m^3	$ 17.00
Concrete Levee Construction			
- concrete masonry blockwork up to 1m high		m	$ 293.00
- concrete wall			
	Height		
	1.0	m	$ 434.00
	1.5	m	$ 675.00
	2.0	m	$ 985.00
	2.5	m	$ 1,463.00
	3.0	m	$ 1,935.00
	3.5	m	$ 2,509.00
	4.0	m	$ 3,173.00
	4.5	m	$ 3,848.00
	5.0	m	$ 4,523.00
- concrete levee with piers		m	$ 3,500.00
Landscaping			
- provision of mounds and mulched gardens including trees and shrubs		m^2	$ 28.00
- provision of general architectural treatment to the surface of a concrete levee		m^2	$ 28.00
- provision of a rock wall finish to the surface of a concrete levee		m^2	$ 150.00
- provision of a 50mm layer of loam sown with couch grass seeds and maintained for 6 months		m^2	$ 6.00

Table 8-5 Unit Rates for Levee Maintenance Costs

Item	Unit	Annual Rate
Mowing		
- mowing of earth batters	m	$ 4.00
Community Education		
- in relation to levee		$ 4,000

Table 8-6 Unit Rates for Levee Operation Costs

Item	Unit	Annual Rate
Levee Monitoring		
- ensuring levee remains intact and regular surveying of levee (every 5yrs or so)	m	$ 2.50

Table 8-7 Unit Rates for River Dredging Operations (Industry Sources)

Item	Unit	Annual Rate
Dredging to Stockpile (up to 3 km)	m^3	$ 4.50
Washing Sand	m^3	$ 2.50
Transport		
- Innisfail to Cairns	m^3	$ 20.00
Selling Rates		
- from stockpile	m^3	$ 1.00
- to market	m^3	$ 15.00

Table 8-8 Unit Rates for Carello's Channel (Industry Sources)

Item	Unit	Annual Rate
Excavation	m^3	$ 10.00
Rock Protection		
-rock wall at 1V:1.5H	m^2	$ 230.00

8.6.3 Constructed Carello's Channel

The proposed channel would be subject to sufficiently high velocities during flooding to scour its banks. Therefore, it is assumed that the banks will be protected with rock and be sloped at 1V:1.5H. The channel would require excavation of approximately 250 000 m^3 of material. The adopted rate of $10/$m^3$ assumes that it is a cut and fill operation and hence does not allow for cartage. No allowance has been made in the costings for the disposal of acid sulphate soils or on-going maintenance of the channel. The cost to treat acid sulphate soils on-site is about $30/$m^3$.

A summary of the BCR calculations for the raising of the levees is given in Table 8-9. The benefits are based on the assumption that floor levels are the ground level plus 0.5 m, which may result in an over-estimate of the benefits if the floor levels are on average under-estimated. The benefits were not recalculated using the revised estimate of floor level obtained for the house raising analysis. A cost breakdown is given in Appendix A. .

If the Sweeneys and Saltwater Creek floodgate levees are raised (refer Section 8.6.5), the benefits of in the town area of constructing the channel will be reduced and hence the BCR would be lower.

Table 8-9 BCR Analysis of Constructed Carello's Channel

Average Annual Benefit	Total Benefit over 50 Years	Construction Costs	On-Going Costs over 50 Years	Total Cost	BCR
$113,000	$1,560,000	$5,316,000	None allowed	$5,316,000	0.29

The proposed channel would require the clearing of an area of mangroves. Preliminary assessment indicates that the following approvals/permits could be required for the works:

- for works in tidal waters, approved plans under Section 86 of the Harbours Act 1955;
- for dredging, Environmentally Referable Activity (ERA) 19a approval under the Environment Protection Act 1994;
- if any material is to be placed on land, a Marine Land Dredging By-law 1987 permit;
- if any marine plants are disturbed, approval under Section 51 of the Fisheries Act 1994.

Other approvals may also be necessary. The requirement for several approvals may trigger the Integrated Planning Act 1997.

Based on experience elsewhere, the environmental impacts of the excavation on a wide range of environmental, social and cultural issues would need to be carefully assessed prior to approval being granted. The benefits of the channel would also need to be fully justified.

8.6.4 Scoured Carello's Channel

The total construction cost is estimated to be $360,700; a cost breakdown is given in Appendix A. Included is a fee for a geotechnical and hydraulic investigation which should be undertaken before design works commence to determine the likelihood of such a channel forming to RL –2.0 m. Rock protection may be required along the side of the new channel adjacent the Carello's property boundary. The cost of rock protection is not included in the calculation of the BCRs. The cost to construct a rock wall down to RL –2.0 along a length of about 235 m would be approximately $350,000.

It is not known when scouring of the channel would be completed and hence when the benefits would be realised. Therefore, a sensitivity analysis of the BCR was undertaken by assuming that benefits begin to accrue over a range of times from immediately to 40 years.

Benefits were determined by assuming that the Scoured Carello's Channel will reduce flood levels to the same extent as the Constructed Carello's Channel (refer Section 8.6.3). However, the average annual damages were recalculated using the revised estimate of floor levels obtained for the house raising analysis (refer Section 6.3.2.1). Therefore, a direct comparison with the BCR in Table 8-9 is not possible.

If the Sweeneys and Saltwater Creek floodgate levees are raised (refer Section 8.6.5), the benefits in the town areas due to the scoured channel will be reduced and hence the BCR would be lower.

If consideration is given to shifting the location of the channel, a revised hydraulic analysis and hence benefit analysis would be required because a channel in a different location may not provide the same reduction in flood levels.

The BCR are presented in Table 8-10. The column "Commencement of Benefits" refers to the period of time after the initial works are undertaken that the channel is fully scoured to approximate the Constructed Carello's Channel. The analysis indicates that an acceptable return on the initial outlay would be achieved if the channel scoured within 20 to 30 years and no rock protection is required. Without having undertaken specific geotechnical or hydraulic investigations into the likelihood of a similar channel forming naturally, it is difficult to make a definitive statement on the viability of such a proposal. However, based on anecdotal evidence of scouring both prior to and after the construction of Carello's levee, it is considered unlikely that a channel of this size would form naturally over a 20 to 30 year period.

The environmental issues would be similar to those identified for the Constructed Channel (section 8.6.3). An additional environmental consideration would be the deposition of the scoured material into the river system.

Table 8-10 BCR Analysis of Scoured Carello's Channel

Commencement of Benefits	Total Benefit	BCR
Immediately	$1,017,000	2.8
In 10 years	$536,000	1.5
In 20 years	$254,000	0.7
In 30 years	$110,000	0.3
In 40 years	$37,000	0.1

8.6.5 Raised Sweeneys Creek and Saltwater Creek Floodgate Levees

A summary of the BCR calculations for the raising of the levees is given in Table 8-11. The costs include the construction of a levee to RL 4.5 m at Frith Road and the construction of a levee adjacent to Scullen Avenue to reduce the impact in Jones Street. On-going costs in this analysis include annual monitoring and survey of levees, mowing and community education. The damages calculations used the revised estimate of floor levels obtained for the house raising analysis (refer Section 6.3.2.1). A cost breakdown is given in Appendix A.

The addition of the Scullen Ave levee adds considerably to the cost of the measure with only small additional benefits in Jones Street. Further investigation is considered warranted into the significance of about a 30 mm increase in flood level in Jones Street in a 20 year ARI flood, remembering that the impacts are not evident in a 50 year ARI event.

Table 8-11 BCR Analysis of Raised Floodgate Levees

Levee	Average Annual Benefit	Total Benefit over 50 Years	Construction Costs	On-Going Costs over 50 Years	Total Cost	BCR
Sweeney	$59,600	$822,000	$28,000	$62,000*	$318,000	2.6
Saltwater			$23,000	$22,000		
Frith Road			$28,000	$12,000		
Scullen Ave			$116,000	$27,000		

Side slope of levee is 1m vertical to 4 m horizontal; top width of levee is 2m

* Includes community education over 50 year period for all three levees

The raising of the levees is not expected to have any environmental impacts given that the floodgates are already in place. The increase in floodgate levee height would bring a positive social benefit by increasing the warning time to residents and business protected by the levees. However, it is likely that the fall of the floodwaters in the town area will be slightly retarded by the increased height on the levees.

8.6.6 Webb Levee

It was assumed that the levee would be concrete in front of waterfront properties and earth in other areas. The earth levee was assumed to have a top width of 2 m and side slopes of 1:4. A summary of the BCR calculations for Webb levee is given in Table 8-12. On-going costs in this analysis include annual monitoring and survey of levees, mowing and community education. The benefits are based on the assumption that floor levels are the ground level plus 0.5 m, which results in an over-estimate of the benefits. The benefits were not recalculated using the revised estimate of floor level obtained for the house raising analysis.

Table 8-12 BCR Analysis of Webb Levee

Levee	Average Annual Benefit	Total Benefit over 50 Years	Construction Costs	On-Going Costs over 50 Years	Total Cost	BCR
10 Year ARI	$11,700	$161,500	$584,000	$90,000	$674,000	0.24

A levee along the river bank would reduce the visual amenity, particularly where it is constructed in front of waterfront properties. Because of this, the levee may not receive the support of the affected members of the community.

The levee is unlikely to have any other detrimental environmental impacts other than the bank stability issues discussed in Section 8.5.4.

8.6.7 River Dredging – Scheme 2

The BCR calculations considered three scenarios:

1. dredge to stockpile and sell in Cairns;
2. dredge to stockpile and sell locally;

3. dredge to spoil

A dredge quantity of 380,000 m^3 with a 10% silt content was assumed for the analysis. The unit rates for the calculations are given in Table 8-13. The analysis does not allow for maintenance dredging or the costs associated with undertaking an Environmental Impact Study. An estimate of maintenance dredging costs cannot be calculated without a sediment processes study being undertaken to ascertain the average annual sediment load. The results of the analysis are presented in Table 8-13.

These BCR are based on the assumption that the floor level is the ground level plus 0.5 m. The BCR is strongly dependent on the floor level as indicated in the results of a sensitivity analysis presented in Table 8-13. The benefits included in the calculation of the BCR are only those relating to a reduction in flood damages. The benefits are based on the assumption that floor levels are the ground level plus 0.5 m, which results in an over-estimate of the benefits. The benefits were not recalculated using the revised estimate of floor level obtained for the house raising analysis.

Table 8-13 BCR Analysis of River Dredging – Scheme 2

Dredge Scenario	Total Benefit over 50 Years	Upfront Funding Required Assuming all Sand Sold	BCR*
Dredge to stockpile and sell to Cairns	$2,280,000	$4,370,000	0.52
Dredge to stockpile and sell locally	$2,280,000	$1,330,000	1.7
Dredge to spoil	$2,280,000	$1,710,000	1.3

*Note: Excludes cost of maintenance dredging

Table 8-14 Sensitivity of River Dredging BCR to Floor Level Assumption

Floor Level Scenario	BCR*	
	Dredge to stockpile and sell locally	Dredge to stockpile and sell to Cairns
Ground Level plus 1.0 m	1.08	0.33
Ground Level plus 1.5 m	0.68	0.21
Ground Level plus 2.0 m	0.49	0.15

*Note: Excludes cost of maintenance dredging

As was noted above, these costings do not include the costs associated with maintenance dredging which would reduce the BCR. Without undertaking a study of the sediment regime in the Johnstone River it is not possible to quantify the maintenance dredging requirements. However, anecdotal evidence obtained during discussions with long-term observers of the river would suggest that the sediment load in the river is relatively high. If this is the case, then there may be significant costs in maintaining the dredged river profile. It is possible that in a larger flood, the dredged channel may be filled which would then require that the full dredging be undertaken to maintain the flood benefits.

Therefore, it is important to note that, if dredging is implemented as a floodplain management strategy, maintenance dredging is vital.

8.6.7.1 Impacts of Dredging on Estuarine Ecology

Potential impacts of channel dredging on the estuarine ecology of the Johnstone River can be considered from the perspective of both primary/direct and secondary/flow-on effects. Key issues arising from a marine ecology perspective include:

- Impacts of dredging;
- Impacts of spoil disposal to offshore areas.

Impacts of Dredging

The effect of dredging on ecological communities, including species of recreational/commercial importance, is difficult to quantify. However, the likely effects of dredging activities can be summarised as follows:

- removal of benthos (bottom living organisms);
- alteration of habitats through the modification of bed profiles;
- short-term water quality impacts associated with the creation of turbid plumes.

Removal of benthos

Dredging would result in the physical removal or disturbance of benthic macroinvertebrate communities within the channel. The longer-term impacts of this removal/disturbance on invertebrate community structure will depend on the characteristics of the disturbed community and the extent of dredging.

Areas that are frequently dredged or are disturbed on a regular basis, such as navigation channels, are likely to support opportunistic (early successional) communities comprised of species that are capable of rapid re-colonisation. The loss of these communities is typically short-term, with recovery times measured in months.

Dredging in areas that are infrequently disturbed may, however, remove diverse benthic communities that would be significantly different from those which would recolonise the dredged area.

Dredging will temporarily remove benthic communities used as a food resource by fish and crustaceans. However, the recovery process is anticipated to be rapid for the more common, opportunistic invertebrate species. There are no data available to assess this potential effect on fish, although the high mobility exhibited by most of the common species may result in fish temporarily moving elsewhere if food is in short supply.

In the long-term, it is expected that moderately rich and abundant benthic communities will colonise the dredged channel.

Habitat Modifications

Dredging will increase the depth of the channel. The recolonising communities will differ from those existing prior to dredging because of changes to the types of habitats available for benthic organisms. Dredging may influence current velocities within the river, potentially resulting in changes to benthic communities.

Unless it is appropriately designed, dredging has the potential to cause hydrodynamic impacts in the river, possibly including changes to water levels and/or currents. Such possible changes could flow on to ecological communities through changes to habitats. Hydrodynamic changes could impact on mangrove communities if water levels change or if erosion/accretion occurs.

Water Quality Impacts

Dredging is likely to generate a turbid plume derived from re-suspended sediments in the works area. Hydrological modelling would be required to predict the extent of any plume.

The impacts of the turbid plume will depend on the tolerance of flora and fauna to elevated turbidity, reduced light penetration and sedimentation. The mouth of the Johnstone River may already experience high concentrations of suspended solids, particularly during ebbing tides, floods and under certain wind/wave conditions. If this is the case, it would be expected that most of the species that occur within this area are adapted to short-term increases in turbidity, and that patterns in community structure reflect the influence of turbid waters.

Any longer-term increases in suspended solid concentrations would be expected to impact on marine flora and, to a far lesser extent, fauna. Seagrasses are sensitive to reduced light, increased sedimentation (through smothering) and suspended solid concentrations, with the lower distribution limit of seagrasses determined by light availability. Persistent, elevated suspended solid concentrations would therefore be expected to result in adverse impacts on seagrass, with a potential reduction in the lower depth limit.

The extent of seagrasses in the lower Johnstone River is unknown. However, any seagrasses present are likely to already experience periods of elevated turbidity in response to wind/wave action and flood-related turbid river plumes.

8.6.7.2 Impacts of Material Placement at the Offshore Spoil Ground

Generation of turbidity plumes

Dredging and dredged material placement at sea is likely to result in elevated suspended sediment concentrations (turbidity plumes). As described above, hydrological modelling would be required to predict the extent of any plume.

Turbidity plume impacts at the spoil ground would be expected to be similar to those associated with the dredging.

Burial of Biota

Dredged material placement at the spoil ground will result in the burial of aquatic organisms that have colonised the spoil ground. Some buried organisms may be able to migrate through appreciable depths of placed material (Herbich 1992) but other organisms are likely to be lost.

Depending on the amount of material deposited, recolonisation of dredged areas and the spoil ground may likely occur within a short time of dredging being completed. Previous studies elsewhere (Burnett River; WBM Oceanics Australia 2000) indicate that there was no detectable long-term impact (ie. after one year) of past spoil disposal activities on benthic communities.

Further investigation would be required to assess the impacts of spoil disposal activities on benthic communities.

The loss of benthic macroinvertebrates from the spoil ground could represent a reduction in available food resources for fish. Most fish species that inhabit the area are mobile, and would therefore forage in other parts of the study region.

8.6.7.3 Approval Requirements

A preliminary assessment indicates that the approvals/permits required for Carello's channel (Section 8.6.3) could be required for the dredging works:

Other approvals may also be necessary. The requirement for several approvals may trigger the Integrated Planning Act 1998.

Based on experience elsewhere, the environmental impacts of dredging on a wide range of environmental, social and cultural issues would need to be carefully assessed prior to approval being granted. The benefits of dredging would also need to be fully justified.

8.7 Summary

Key issues relating to each of the four measures investigated in detail are summarised Table 8-15.

Table 8-15 Summary Table – Flood Modification

Measure	BCR	Key Issues	Committee Decision
Constructed Carello's Channel	0.29	Provides widespread minor reductions in flood level. BCR does not allow for maintenance, acid sulphate soils or cartage and does not use revised floor level data. High capital cost. Environmental considerations relating to the clearing of mangroves, excavation and disposal of spoil.	The Committee's recommendation is that this measure should not be included in a Floodplain Management Scheme.
Scoured Carello's Channel	0.1 to 2.8	BCR strongly dependent on period taken for channel to scour. BCR does not allow for rock protection. BCR used revised floor levels. BCR will be lower if Sweeneys and Saltwater Creek floodgate levees are raised. Environmental considerations relating to the clearing of mangroves, excavation and disposal of spoil and deposition of scoured material in river system. Further hydraulic and geotechnical investigation recommended to assess the likelihood of success.	The Committee's recommendation is that this measure should not be included in a Floodplain Management Scheme.
Raising of Sweeneys & Saltwater Creek Floodgates	2.6	Significant benefits in areas protected by levees. Some minor increases in flood level "outside" of the levee. BCR uses revised floor level data. Additional levee may be required at Scullen Avenue, although it is a high capital cost item with minimal benefit. Further investigation into the significance of a 30 mm increase in the peak 20 year ARI flood levels in the Jones Street area is recommended before including this levee in the scheme. No environmental issues.	The Committee's recommendation is that this measures should be included in a Floodplain Management Scheme.

Measure	BCR	Key Issues	Committee Decision
Webb Levee	0.24	Reduces flood levels behind levee, although does not completely stop inundation in all areas behind levee. Significant intangible benefits. BCR does not use revised floor level data. High capital cost with benefit to minor part of community. No significant increase in flood levels elsewhere on the floodplain. No significant environmental considerations other than localised bank stability issues and visual amenity.	The Committee's recommendation is that this measure should not be included in a Floodplain Management Scheme.
Dredging – Scheme 2	0.5 to 1.7	Provides widespread minor reductions in flood level. BCR strongly dependent on floor level assumptions and did not allow for maintenance dredging. BCR does not use revised floor level data. May still be viable option if benefits to other industries are considered. Significant environmental considerations.	The Committee's recommendation is that this measure should not be included in a Floodplain Management Scheme, but that the findings of this report could be used as supporting information in other dredging applications.

9 PROPERTY MODIFICATION MEASURES

The aim of property modification measures is to reduce the number of buildings that are inundated in a particular design flood event. This can be achieved by: (i) purchasing flood-prone buildings and re-locating or removing them; (ii) raising the floor level of existing buildings; and/or (iii) imposition of controls on property and infrastructure development. The following property modification measures were investigated:

- **Voluntary House Purchase**
  Purchasing houses that are located within a High Hazard - Floodway area.
- **Voluntary House Raising**
  Raising the floor level of individual houses to a specified level thereby reducing the number of houses that are inundated during flooding events. Criteria are defined (e.g. buildings that are inundated in the 50 year design flood) for selecting those buildings to be considered for house raising.
- **Development Control Planning**
  The imposition of controls on property and infrastructure development. For example, setting the minimum habitable floor level for new houses based on the design flood levels.

Before these measures can be developed, it is necessary to define the flood hazard on the floodplain.

9.1 Hazard Assessment

Integral to the development of a Floodplain Management Plan is the definition of flood hazard over the floodplains. This section discusses the different approaches available for defining flood hazard. The Queensland Department of Natural Resources and Mines is currently writing a floodplain management manual for Queensland, but a publishing date has not yet been determined, so the discussion in this section relies on the Australian and NSW guidelines and floodplain management plans prepared for catchments in NSW.

9.1.1 Description

Flood hazard is the term used to describe the potential risk to life and limb and potential damage to property resulting from flooding. The degree of flood hazard varies both in time and place across the floodplain. Floodwaters are deep and fast flowing in some areas, whilst at other locations they are shallow and slow moving. It is important to determine and understand the variation of degree of hazard and flood behaviour across the floodplain over the full range of potential floods.

9.1.2 Flood Hazard Categorisation

A review of the methodology in CSIRO (2000), DLWC (2001) and previous floodplain management studies for the categorisation of flood hazard is undertaken and a methodology is recommended for the Johnstone River Floodplain.

9.1.2.1 CSIRO (2000)

It is necessary to divide the floodplain into flood hazard categories that reflect the flood behaviour across the floodplain. CSIRO (2000) refers to the degree of flood hazard as being a function of:

- the size (magnitude) of flooding;
- depth and velocity (speed of flowing water);
- rate of floodwater rise;
- duration of flooding;
- evacuation problems;
- effective flood access;
- size of population at risk;
- land use;
- flood awareness/readiness;
- effective flood warning time.

CSIRO (2000) suggests four degrees of hazard; low, medium, high and extreme. The categorisation of the floodplain is largely qualitative using the above factors. For example, medium hazard is where adults could wade safely, but children and elderly may have difficulty, evacuation is possible by a sedan, there is ample time for flood warning and evacuation and evacuation routes remain trafficable for at least twice as long for the required evacuation time.

A key factor in the ease of evacuation from an area is the water depth and the velocity along the evacuation route, ie, the stability of pedestrians wading through flood waters or vehicles driving along flooded roads. CSIRO (2000) notes that there are estimation procedures available for stability estimation, but considers that further research is required across a broader range of conditions and so does not recommend a procedure for hazard categorisation on this basis.

9.1.2.2 DLWC (2001)

DLWC (2001) identifies similar contributing factors to flood hazard as identified in CSIRO (2000). However, in recognition of the need to incorporate floodplain risk management into statutory planning instruments, DLWC (2001) recommends that land-use categorisation in flood prone areas be based on two categories, ‘hydraulic’ and ‘hazard’. Hydraulic categories “*reflect the impact of development activity on flood behaviour”*, and hazard categories reflect “*the impact of flooding on development and people.*” Three hydraulic categories are identified – fringe flooding, flood storage and floodway – and two hazard categories – high and low resulting in the following categories:

1. Low Hazard – Flood Fringe
2. Low Hazard – Flood Storage
3. Low Hazard – Floodway
4. High Hazard – Flood Fringe

5. High Hazard – Flood Storage

6. High Hazard - Floodway

A definition of the hydraulic and hazard categories is given in Table 9-1.

DLWC (2001) recommends that the definition of hazard initially be undertaken using relationships between depth (D) and velocity (V) of floodwater, ie, using hydraulic principles, and then the categorisation should be refined using the other contributing factors to hazard noted in Section 9.1.2.1.

The consideration of depth and velocity is based on curves presented in the DLWC (2001) and shown in Figure 9-1 and Figure 9-2. In basic terms, the first of these curves shows high hazard for:

- depths greater than 1m;
- velocities greater than 2 m/s;
- D + 0.3 x V > 1.0 (where D=Depth, V=Velocity).

Table 9-1 Definition of Hydraulic and Hazard Categories (DLWC, 2001)

Category	Definition
Hydraulic	
Flood Fringe	The remaining area of flood prone land after floodway and flood storage have been defined. Development in this area would not have any significant effect on the pattern of flood flow and/or flood levels
Flood Storage	Those parts of the floodplain that are important for the temporary storage of floodwater during the passage of a flood. A substantial reduction of the capacity of the flood storage would increase nearby flood levels, re-distribute flows and increase flows downstream.
Floodway	Those areas where a significant volume of water flows during floods and are often associated with natural channels. If they are even only partially blocked, there will be a significant increase in flood levels and possibly a re-distribution of flows resulting in impacts elsewhere.
Hazard	
Low	People and possessions could be evacuated by trucks and/or wading. The risk to life is considered to be low.
High	Evacuation by trucks would be difficult, able-bodied adults would have difficulty wading to safety, possible danger to personal safety and structural damage buildings is possible.

9.1.3 Recommended Approach

In considering the application of these issues to the specific flood characteristics of the lower Johnstone River floodplain, it is noted that:

- duration of flooding is universally long (in the order of days) across the floodplain;
- warning times can be short (~6 hrs);
- rates of floodwater rise are reasonably fast; and
- flood awareness is generally high and does not vary significantly across the floodplain.

The above four parameters are not significantly variable across the floodplain to warrant specific treatment and are therefore not used to define variations in the flood hazard, but should be included in development control measures. The flood hazard is therefore defined on the remaining, varying characteristics of:

- the size of the flood;
- depth and velocity of floodwaters; and
- evacuation and access.

On this basis it recommended that the hazard categories in Table 9-2 be adopted for the Johnstone River floodplain and that they be defined in accordance with the criteria in Figure 9-3 which combines Figure 9-1 and Figure 9-2.

Table 9-2 Flood Hazard Categories for Johnstone Floodplain

Hazard Category	Base Flood Event	Characteristics
Low	100 yr	➢ Areas that are inundated in a 100yr flood, but the floodwaters are relatively shallow (typically less than 1m deep) and are not flowing with high velocity ➢ Adult can wade
High – Wading Unsafe	100 yr	➢ The depth and/or velocity are sufficiently high that wading is not possible - risk of drowning
High – Depth	100 yr	➢ Areas where the floodwaters are deep (> 1m), but are not flowing with high velocity. ➢ Damage only to building contents, large trucks able to evacuate
High – Floodway	100 yr	➢ Typically areas where there is deep water flowing with a high velocity ➢ Truck evacuation not possible, structural damage to light framed houses, high risk to life
Extreme	100 yr	➢ Typically areas where the velocity is > 2 m/s ➢ All buildings likely to be destroyed, high probability of death

The High Hazard – Wading Unsafe category is included as it is considered that it may be of benefit to the State Emergency Service in their planning response. It is not a category in the preliminary Development Control Plan (DCP).

9.1.4 Flood Hazard Maps

Using the Flood Hazard categorisation described in the previous section, flood hazard has been determined for the entire floodplain using the 100 year ARI design flood and is presented in Figure 9-4.

9.2 Voluntary House Purchase

9.2.1 Description

House purchase is primarily aimed at reducing risk to life-and-limb by purchasing houses that are in High Hazard Floodway areas, but purchasing these houses can also have a secondary benefit of reducing flood damage. Such measures can only be undertaken on a voluntary basis with the property owner.

There are no residential buildings located within the High Hazard Floodway zones shown in Figure 9-4 and so voluntary house purchase is not considered further.

The Committee's recommendation is that this measure should not be included in a Floodplain Management Scheme.

9.3 Voluntary House Raising

9.3.1 Description

House raising is aimed at reducing the flood damage to houses by raising the floor level of individual buildings to a specified level. Thus, the number of houses that are inundated during flooding events may be reduced. Such measures can only be undertaken on a voluntary basis.

Assessments undertaken in relation to the raising of buildings have been limited to urban areas, although rural buildings may be considered on a case-by-case basis. The reduction in damages achieved by raising a building is determined using the stage-damage relationships as discussed in Section 6.3.2.2.

A preliminary analysis undertaken using an assumption that the house floor level was 0.5 m above the ground level was presented to the SC on 19 September 2002. On reviewing this analysis, the SC considered that the measure should be investigated further using a better estimate of the floor level. An estimate of the floor level of all urban residential properties previously assumed to be inundated in a 100 year ARI flood was undertaken by estimating the height of the floor above the ground level; the estimate was a visual assessment from the road corridor. As part of this survey, it was also noted whether or not the house could be raised. Alan Dunne from the JSRIT undertook this survey. No other building type data was collected as part of this survey. The analysis using this later data is presented.

A basic procedure for calculating reduction in flood damage is as follows:

- re-calculate the existing average annual damages using the revised estimate of floor level;

- define a criteria for selecting those buildings to be considered for house raising - as outlined in Section 9.3.2, three different criteria were defined;
- calculate the average annual damages after raising those houses that satisfy the defined criteria;
- estimate the cost of raising the houses; and
- determine a monetary benefit-cost ratio for each scenario.

9.3.2 Criteria

The best return for investment would be achieved by raising only those houses inundated in smaller events because they are inundated more frequently, but this would not be equitable across the community. Therefore, raising of all eligible houses to the 100 year ARI flood level (plus a freeboard) is considered. To demonstrate the benefits of raising lower houses, two scenarios involving raising only lower houses were also investigated. The three scenarios analysed are presented in Table 9-3.

Table 9-3 Description of Voluntary House Raising Options

Option	A	B	C
Description	Raising of houses currently inundated by a 20 year flood event	Raising of houses currently inundated by a 50 year flood event	Raising of houses currently inundated by a 100 year flood event

For each option, the floor levels of buildings inundated by the specified flood event were considered to be raised to above the 100 year flood level, the height above the 100 year ARI flood level is normally the freeboard specified in the DCP. Option B includes all houses identified for raising in Option A, and Option C includes all houses identified for raising in Options A and B.

Commercial properties and rural residential properties were not considered in this assessment, although if voluntary house raising is included in the final scheme it should be made available to residential properties in rural areas.

9.3.3 Benefits

The monetary benefits of house raising arise from the reduction in the level of flood damage incurred by the community. In addition, there are a number of health, social, and psychological benefits as people are spared the trauma associated with having their homes and/or businesses inundated by flood waters. These are not easily quantifiable in monetary terms, and are not included in the monetary benefit-cost calculations.

9.3.4 Costs and Impacts

The average cost of raising a house in Cairns as provided by a Cairns-based house-raising companies is \$22,000. This price includes jacking, restumping, extending stairs, electrical and plumbing. Contingencies of 25% were added to the average prices. This is higher than prices quoted by two Townsville companies for a similar analysis in Ingham where a typical price of \$16,000 was quoted. An average price of \$20,000 plus 25% for contingencies was adopted for this analysis.

One impact of house raising could be the reduction in the proportion of low cost rental properties in the market. This can be construed as a negative impact because low income families may be forced to relocate. However, if there is an abundance of low cost rental properties in the market, this impact may not be significant.

9.3.5 Monetary Benefit-Cost Ratios

Table 9-4 presents the costs and benefits of each of the house-raising options analysed. The BCR decreases as houses with higher existing floor levels, and hence less frequent above floor flooding, are included. This is expected because the cost to raise a house is the same in each option, but the benefits are only realised in larger floods and hence the benefits are realised less frequently than for lower houses that are raised.

The analysis indicates that option A is worth consideration, but in only raising houses that have existing floor levels of 20 year ARI or less creates an inequity in spending of public money. This inequity would be particular obvious in a situation where neighbouring houses have very similar floor levels, but one house is identified for raising using public money and the other is not. These issues can be at least partially managed by making the scheme available to all with houses inundated in a 100 year ARI event, but by applying funding arrangements as discussed in Section 9.3.6. The analysis has only considered houses in Urban areas, but for the purposes of equity, consideration should be given to making the scheme available to the rural community.

Funding for this scheme could be obtained through the Regional Flood Mitigation Program which is administered by the Department of Natural Resources and Mines. This program would contribute 2/3 of the funds. Allocation of funding is based on four criteria, one of which is BCR. Given the low BCR of Option C and the fund's annual budget for all of Queensland of about $6M, the application may struggle to obtain funding under the scheme depending on the other applications in that year. Given the limited funds available each year in this fund and the practicalities of raising houses, it is recommended that if an application is made, that it be on the basis of raising approximately 20 houses per year over about a ten year period assuming there would be about a 50% take up rate.

Table 9-4 BCR - House Raising

	Option A	Option B	Option C
Total Benefit	$0.4 M	$1.0 M	$1.1 M
Number of Houses Raised	48	194	338
Total Cost	$1.2 M	$4.9 M	$8.5 M
BCR	0.33	0.20	0.13

9.3.6 Funding Arrangement Options

As raising of a house is likely to result in an increase in property value, it is reasonable that the owner contribute a portion of the cost required to raise the house. The proportion that the owner is asked to contribute should be chosen carefully so as not to discourage the owner from raising their house. In recent floodplain management plans developed by WBM, the owner is required to contribute at least one-sixth (1/6) of the overall cost.

Two possible funding arrangements are the "Sliding Rule" and "Band Rule". An example of the "Band Rule" funding arrangement is given in Table 9-5 and an example of the sliding rule in Figure 9-5.

Table 9-5 Example of Funding Arrangement used in NSW for House Raising

Band	First $10,000 of House Raising Cost			Remainder of House Raising Cost		
	Council	State	Owner	Council	State	Owner
Band A[1]	1/6	4/6	1/6	1/6	4/6	1/6
Band B[2]	1/3	2/3	-	-	-	full amount
Band C[3]	-	2/3	1/3	-	-	full amount

[1] Band A – Houses inundated above floor level in the 20 year event

[2] Band B – Houses inundated above floor level in the 50 year event (but not the 20 year event)

[3] Band C – Houses inundated above floor level in the 100 year event (but not in the 50 year event)

9.3.7 Summary

The Committee's recommendation is that House Raising Option A should be included in a Floodplain Management Scheme with the local contribution fully funded by the property owner.

9.4 Development Control

9.4.1 Background

In recent years, floodplain management has placed increasing emphasis on non-structural solutions. In particular, the use of town planning controls, which relate to a number of different non-structural floodplain management measures including floor level controls, flood warning and evacuation, building design, voluntary house purchase, distribution of appropriate landuses etc.

Traditional floodplain planning has relied almost entirely on the definition of a single flood standard, which has usually been based on the 100 year ARI flood event. Overall, this approach has worked satisfactorily. However, it is now viewed as simplistic and inappropriate in certain situations. In particular, it has failed to comprehensively consider the varying land uses and flood risks on the floodplain.

A number of new approaches have emerged from Floodplain Management Studies completed in regions of NSW which provide a transitional level of control based on flood hazard and the sensitivity of the possible range of landuses to the flood risk. As noted earlier, DLWC (2001) reflects this new approach to floodplain planning.

This section reviews the planning tools available to town planners in floodplain management, the traditional approach to floodplain management, new planning approaches that have emerged and recommends an appropriate approach for the Johnstone River area.

The 'Traditional Approach' to planning, which has been widely adopted by councils, involves:

- consideration of a range of events to select a 'Flood Standard', typically the 1 in 100 year ARI event or a known historical flood, irrespective of landuse;
- adoption of the 'Flood Standard' to define flood liable land, above which flood planning is not considered and below which development control occurs.

The Traditional Approach to floodplain planning results in restricted development on a merit basis below the Flood Standard and most development above the flood standard. This also reinforced the community belief that there is no flood hazard above the standard.

In general, this approach has worked well, but has led to a number of problems including (Bewsher and Grech, 1997):

- creation of a 'hard edge' to development at the Flood Planning Level (FPL);
- distribution of development within the floodplain in a manner which does not recognise the risks to life or the economic costs of flood damage;
- unnecessary restriction of some land uses from occurring below the FPL, while allowing other inappropriate land uses to occur immediately above the FPL;
- polarisation of the floodplain into perceived 'flood prone' and 'flood free' areas;
- lack of recognition of the significant flood hazard that may exist above the FPL (and as a result, there are very few measures in place to manage the consequences of flooding above the FPL);
- creation of a political climate where the redefinition of the FPL (due to the availability of more accurate flood behaviour data, or for other reason) is fiercely opposed by some parts of the community, due to concern about significant impacts on land values ie. land which was previously perceived to be 'flood free' will now be made 'flood prone' (despite the likelihood that such impacts may only be short term).

Therefore a number of councils in NSW have considered it inappropriate to adopt a single Flood Standard.

A number of new planning approaches have emerged from Floodplain Management Studies completed in regions of NSW (Hunter, Hawkesbury, and Paterson) which provide a transitional level of control based on flood hazard and the sensitivity of the possible range of landuses to the flood risk. This approach is incorporated into CSIRO (2000) and DLWC (2001). In DLWC (2001) the following changes have been implemented.

- The term Flood Liable Land is replaced by the term Flood Prone Land and is to be defined as land inundated by the Probable Maximum Flood (PMF).
- The focus on the PMF changes from considering "if" it happens to "when" it happens. That is, the probability of a PMF is extremely small but real and therefore requires consideration in the Floodplain Management process (this has been driven by the recent occurrence of floods exceeding the 100 year event).
- It reinforces the need to manage the floodplain through assessment of a range of design floods rather than a selected standard flood.

- The Flood Standard is to be replaced by Flood Planning Levels (FPL's), which indicates that a range of planning levels may be used. This is one of the most crucial changes in that it reinforces an approach of matching FPL's with different land-uses and using the FPL's as planning control mechanisms. Many different factors are to be considered in the selection of appropriate FPL's.
- The adoption of the varying FPL's is promoted in the available planning tools.
- There is reinforcement of the links required between the Floodplain Management Plan and the emergency management.
- Other issues are also introduced or further reinforced such as Ecologically Sustainable Development, Total Catchment Management, Community Consultation, climate change and riverine environment enhancement.

Figure 9-6 illustrates the general approach to planning promoted in DLWC (2001). The approach promotes the definition of varying flood hazard across the floodplain and defines appropriate landuses with the hazard zones, and when required, provides adequate development controls for the relevant landuse and hazard.

9.4.2 Current Approach in Johnstone Shire

9.4.2.1 Flood Policy

Currently, the Johnstone Shire Council has a Planning Approach to floodplain management which is based on the Traditional Approach. A single Flood Planning Level (FPL) of the 50 year ARI flood is used, although this is currently being revised. All development control within flood prone land is related to this FPL. No formal consideration is given to floods larger than the 50 year ARI flood or the associated hazards. JSC is currently revising its planning scheme and it is recommended that this opportunity be taken to incorporate floodplain risk management into the scheme.

9.4.3 Review of Approaches

The following sections provide a summary of the various approaches with a recommended approach outlined in Section 9.4.4. Issues that must be considered in the development of a DCP are listed below.

- Landuse categories
- Floodplain planning controls will be developed through the Floodplain Management Study
- Flood hazard categorisation must be complete by the commencement of the process using methods approved by the SC
- Johnstone Rivers System Characteristics:
    - extent and depth of flooding and hazard can be mapped reasonably accurately as a result of the modelling undertaken as part of this study;
    - the majority of flooding on the floodplains is deep slow moving floodwater;
    - a major proportion of the flood prone land is rural landuse;
    - a major concern is the management of flooding in the urban centres;

- there are a large number of residential properties that would be inundated in a 100 year ARI flood event

- Community tolerance and acceptance of the level of flood inconvenience

9.4.3.1 Traditional Approach

The Traditional Approach to floodplain planning has been described and reviewed in Section 9.4.1. The approach has been adopted by many councils throughout QLD, but has been found to be inadequate in areas that have experienced flooding larger than the FPL. Also this approach is not in line with current developments in floodplain management such as detailed in CSIRO (2000) and the NSW Floodplain Management Manual (DLWC, 2001). WBM Oceanics Australia recommends that this approach is not adopted for planning in Johnstone Shire.

9.4.3.2 Planning MATRIX

An approach initially developed for the Blacktown Floodplain Management Study in NSW by Bewsher Consulting, and adopted by a number of other councils, is the Planning MATRIX Approach. The approach distributes landuses within the floodplain and controls development to minimise the flood damages as illustrated in Figure 9-6. Using this approach, a matrix of development controls, based on the flood hazard and land use, can be developed which is illustrated and explained in Figure 9-7 (Bewsher and Grech, 1997). A number of plans showing flood hazard, landuse and flood level information accompany the MATRIX, the total of which constitutes a DCP.

Steps involved in developing a Planning MATRIX follow:

- Categorising the Floodplain - divide the floodplain into areas of differing hazard.
- Prioritising Land Uses - review all landuses used by council and divide into discreet categories of land uses with similar levels of sensitivity to the flood hazard. The categories are then listed under each hazard band in the planning matrix in priority of land use.
- List Planning Controls (Building and Community Response) - assign different planning controls to modify building form and the ability of the community to respond in times of flooding, depending on type of land use and location. A number of these controls will be non-structural controls identified in the Floodplain Management Study.

The developed DCP can be adopted by Council as a new DCP to cover development applications.

9.4.3.3 Lismore Floodplain Management Study 1999 (PBP)

The Lismore Floodplain Management Study undertaken by Patterson Britton and Partners (PBP) involved:

- the review and comparison of an existing floodplain DCP and the Lismore LEP;
- provision of recommendations to amend both tools for compatibility purposes;
- highlighted areas in the current DCP and LEP which are deficient particularly in regard to control mechanisms for various landuses (particularly, the consideration of major events between the 100 year ARI and PMF which have not been considered in the current planning tools); and

- outlined development control issues which require committee resolution.

Based on preliminary discussions with PBP, the general Planning Assessment Approach they have adopted for this comparison process in Lismore and also other study areas, in which no floodplain planning tools exist (eg. Hunter River), has been very similar to the Planning MATRIX approach outlined above. However, the final deliverable product, being the planning tables and the relevant landuse and hazard maps, presents the approach in a different format.

9.4.4 Recommended Approach

JSC are currently preparing a new planning scheme under the Integrated Planning Act 1998 (IPA, 1998). The new scheme is at a draft stage. The new scheme should account for landuse, flood hazard and recommend appropriate control measures or solutions. A possible approach to include these floodplain management principles is given below, although it is understood, as discussed below, that there may be some difficulties in this approach.

- For each Landuse Category, develop a Flood Planning Matrix. When development applications are being processed, Council staff will source the appropriate matrix to specify any control measures related to flooding.
- Identification of the appropriate flood hazard category(ies) applicable to a property will be made through a flood hazard map.
- The system proposed has been designed to be performed using hardcopy plans or interactively carried out on a computer using Council's GIS.

It is understood from preliminary discussions with Bob Devine and Darryl Jones from the JSC, that some adjustment to the format and content of the draft scheme would be required to incorporate such an approach. Further, IPA (1998) may not allow land to be categorised such that development on it is prohibited as is done in the example matrices in High Hazard Floodway areas. Another consideration is the use of the word "controls". These may need to be called "possible outcomes" as controls indicates some type of a restriction. If it is concluded that it is not possible to incorporate a flood planning matrix into the new scheme, then it is recommended that the principles of floodplain management that are incorporated into the example planning matrices presented in Section 9.4.5 be incorporated into the planning scheme.

9.4.5 Development of JSC Planning Matrices

Example planning matrices are contained within Figure 9-8, Figure 9-9 and Figure 9-10 for each of the discrete land use categories. These matrices would need to be adapted to the particular requirements of JSC.

9.4.6 Use of JSC Planning Matrix

It is intended that the planning matrix be utilised by those Council officers assessing or advising on development applications. The procedure used by officers follows these steps:

- identify the land use of the site under consideration;
- identify the flood hazard category applicable to the site under consideration again by either visual inspection of hardcopy plans or by interrogation of a GIS layer;

- use the matrices to determine the controls relating to the site based on land use and flood hazard category.

There is the potential for a significant advantage in being able to access the land use and flood hazard category from a GIS database as both items are able to be provided with one on-screen query. The data has been developed with this in mind.

9.4.7 Recommendation

It is considered that the adoption of floodplain management principles into the Planning Scheme is fundamental and should occur. The planing matrix presented incorporates these principles and could be adopted, but it is understood there may be some difficulty in incorporating such a document into new planning schemes that are being developed under IPA (1998). An alternative may be to incorporate the recommendations in the matrix into the new scheme but in a compatible format.

The Committee's recommendation is that floodplain management principles should be included in the Town Planning Scheme.

10 RESPONSE MODIFICATION MEASURES

Response modification measures are aimed at increasing the ability of people to respond appropriately in times of flood and/or enhancing the flood warning and evacuation procedures in an area. The following response modification measures have been investigated:

- **Flood Warning & Emergency Planning**
  An effective flood warning system, in combination with a high level of community awareness, is invaluable in minimising the flood damages and trauma associated with flooding. An accurate, prompt warning system ensures that residents are given the best opportunity to move their possessions out of the danger of floodwaters. Comprehensive emergency planning ensures that no time is wasted in the event of a flood and response measures are implemented efficiently.

- **Raising Community Awareness**
  As the community becomes more aware of the potential for flooding, it is less likely that people will experience health and psychological trauma following a flood. Also, the community will be more likely to respond effectively to flood warnings and to remove possessions and themselves from the dangers of floodwaters.

To assess the status quo in each of these areas, the study team met with the Manager, Technical Services who is a member of the Counter Disaster Committee (CDC). In the following sections, further background information is provided along with an assessment of the status quo and decisions of the Steering Committee.

10.1 Flood Warning & Emergency Planning

10.1.1 Description

The primary responsibility for flood warning and emergency response in the Johnstone Shire is given to the CDC. There are many factors which determine the success or otherwise of the flood warnings and assistance that the CDC are able to provide. These factors may be divided into the four main groupings of:

1. Community ***awareness***.
2. Quality of flood information ***received*** by the CDC from other sources.
3. Ability of the CDC to ***assess*** this information.
4. Ability of the CDC to ***respond*** to their assessment by providing advice and assistance to the community.

Each of these key areas is discussed in detail in the following sections.

10.1.2 Community Awareness

Community awareness and preparedness is an important factor in determining the success of flood warnings and response. A flood aware community is able to understand flood warnings, how they relate to their particular situation and to respond appropriately. It is important to note that community awareness and flood warning are strongly linked.

10.1.2.1 Status Quo

The following assessment of the current level of community awareness is based on information obtained from discussions with Will Higgins (former Manager Technical Services, JSC). Ideally this information would be obtained from members of the community not directly involved in emergency management, but this was not within the scope of the study.

- The community has a high flood awareness because of the regular flooding that has historically occurred in the floodplain. However, the experience is limited to small to medium size floods in the range 1 to 30 year ARI, the 30 year ARI being about the return period of the 1967 flood in the Innisfail CBD.
- Will Higgins from JSRIT was not sure of the level of understanding in the community as to what flood information is available, how it can be accessed during a flood and how it can be interpreted, but thought that there may be some confusion given the questions asked on the flood information line during floods.
- The JSC produces a Cyclone Advice Booklet. The booklet focuses primarily on cyclones, but does include some information on what to do during flooding and also explains the Bureau of Meteorology flood classifications. This booklet is not directly distributed to the community, but it can be collected from Council offices. Therefore, the booklet may not be widely distributed.
- An advertisement relating to flood awareness is placed in the newspaper each year. The advertisement may not be widely read because it is placed in the regular JSC advertising block and would not be very large.

10.1.2.2 Recommendations

Public Education

It is recommended that the public education program be expanded to raise community awareness and explain new initiatives. Recommendations are given in Section 10.2.

Flood Totems

Molino and Rogers (1999) outlined the potential confusion associated with attempting to convey flood information in the form of either flood frequencies or flood heights. Also, assigning categories to bands of flooding (i.e. “minor”, “moderate” and “major”) can also be misleading because the names of the categories mean different things to different people, depending on their experience of flooding. A strategy adopted by Molino and Rogers (1999) involved colour coding the flood categories and simply using the names of the colours for each of the flood categories.

The concept of coloured flood totems has been presented to the SC. The Bureau of Meteorology (BoM) informally expressed some reservations when this concept was discussed on the Herbert River study because they believe it may cause confusion with their current flood classification system (minor, moderate and major) which also is colour coded. However, the BoM colour classification system is not well recognised nor actively promoted.

If implemented, the flood totem will form a significant focus for both community preparedness and flood warning. Introduction of these to the community will serve to provide education as to the flood potential within their area. They will also allow the effective communication of the expected peak level of a flood in a way that all can understand and readily apply to their own situation. This is particular important in this catchment where the warning time can be relatively short.

The flood totem is discussed in further detail in Section 10.1.5.1.2.

10.1.3 Quality of Flood Information Received by the CDC

10.1.3.1 Status Quo

An extensive ALERT flood warning system is installed in the catchment. It provides reliable instantaneous rainfall and river height information. The CDC directly accesses this information during a flood: this information is also available to the public via the internet, but this may not be widely known. The CDC also receives peak flood height predictions at many river gauges throughout the catchment from the BoM. The BoM predicts the peak flood heights using their URBS hydrologic model. Real-time data for the URBS model is obtained from the ALERT system. The URBS hydrologic model has been well calibrated to many floods and provides reliable peak flood height predictions in the lower Johnstone River system. The BoM will continue to refine the URBS model following future flood events.

Under current procedures, the Counter Disaster Committee (CDC) is activated once the nominated representative receives a flood warning from the Bureau of Meterology. Although this system generally works well, the response time available on the Johnstone River can be as as short as about 6 hours. Any additonal warning time will allow the CDC and the community to better respond to the threat and thereby potentially reduce the risk to life and property.

It is proposed to install river height and rainfall alarms at the Nerada and Corsi alert stations. The alarm would be triggered at a predetermined river height or rainfall scenario. The alarm would be sent to either a pager or mobile telephone of a nominated member of the CDC who then activates the CDC if required. This proposal will potentially increase the warning time, and hence preparation time, in the order of 2 hours.

Additonal warning time could be achieved by installing alarm systems on rainfall gauges further up the range than Nerada and Corsi. It is recommended that discussions be held with the Bureau of Meterology to determine the most appropriate gauges. Possibilities include Sutties Creek, Greenhaven, Milla Milla, Bartle View, Topaz and Crawfords Lookout. The alarm would be triggered at a predetermined rainfall scenario. The alarm would be sent to either a pager or mobile telephone of a nominated member of the CDC who then activates the CDC if required.

10.1.3.2 Recommendations

The Committee recommended that alarms be included on the Nerada and Corsi Alert Stations and that the inclusion of alarms at other strategic locations be investigated.

10.1.4 Assessment of Flood Information

10.1.4.1 Status Quo

At present, the main method of interpretation used by the CDC is local knowledge and experience. This is without doubt a strong basis for assessment. However, this experience is limited to floods in the 20 to 50 year ARI range.

The Innisfail Wharf gauge is the main gauge used by the CDC to assess the magnitude of flooding and the level of response required.

10.1.4.2 Recommendations

Information able to be offered to the CDC from this study will aim to enhance the local knowledge base especially for floods larger than those experienced in living memory. Outputs from the study could include flood inundation extent and depth maps and flood velocity maps. These emergency management maps can be related to a gauge height or the colour classification system described below.

An opportunity exists to establish a flood classification system that provides a link between community education and flood warnings. For example, a flood warning from the CDC to the public would refer to the flood as a particular colour in addition to the current warnings which provide the predicted peak flood height at the river gauges. The methodology for assigning a colour code to a flood is explained in Section 10.1.5. Coloured flood totems would be positioned at various locations around the urban and commercial areas and possibly along main roads in rural areas thereby giving residents an appreciation of the implications of, for example, a "red" flood in their local area.

The Committee's recommendation is that colour classification of floods and a review by the CDC of the benefits of emergency maps are included in a Floodplain Management Scheme.

10.1.5 CDC Response

10.1.5.1 Warnings

10.1.5.1.1 Status Quo

Once the CDC has assessed the data received, it disseminates flood warnings to the community in the following ways:

- Flood information telephone lines;
- Radio broadcasts.

The information available to the community is usually rainfall, the current river height, the predicted peak river height and the expected time that the peak will occur. The community can also access this information through the BoM web site.

The shortcomings of this system include a reliance on peoples' prior experiences of flooding to put a predicted river height peak into their local context and a reliance on people to seek information during floods rather than being directly warned. The latter problem is exaggerated by the short warning times in this catchment.

10.1.5.1.2 Recommendations

It is recommended that the CDC improve its current procedures for disseminating information to the public. Improvements could include a local flood warden system, faxing warnings to local business and colour banded flood totems.

The establishment of a local flood warden system would help to make people aware of flood warnings. The CDC would notify the local flood wardens who would then have the responsibility to disseminate this information to their local area. This may not be practical in the Johnstone catchment where the storm/cyclone event can still be over the town when the flood warnings start, thereby making it dangerous for a warden to be contacting residents. To further help the dissemination of flood warnings, local businesses could be faxed warnings or key members of the business community could be contacted and asked to notify their business neighbours.

The implementation of colour classification of floods and flood totems would improve the community's awareness of flooding and allow them to quickly assess the implications of the flood in their local area. The colour of the flood is determined using a correlation between the flood height at the gauge and a colour system and the peak flood height issued by the BoM for that gauge. Using Webb as an example, the system would work as follows:

1. The CDC receives a predicted peak flood height at the Innisfail Wharf gauge;
2. Using the flood classification chart in Figure 11-1, the CDC will determine the colour classification of the flood;
3. CDC then advise the Flood Information Service of this classification and broadcast to the community (it is recommended that rainfall, river height and predicted river heights still be issued);
4. The SES uses both their prior knowledge of flooding and the new flood inundation extent and depth maps to assess their response (the new flood maps would be tied into the colour classification system, ie, their would be different flood maps for each colour flood);
5. Residents walk to the nearest flood totem to assess the implications of the flood warning in their local area (an example of a flood totem is given in Figure 11-2 - note that the actual totem would only show colours, not flood levels or flood ARI).

The colour scheme and band widths presented here are intended to be indicative only. The final colour scheme should be determined in consultation with the SES at both a local and state level. Although the band widths in this example are based on flood ARI, these should not be considered obligatory break points between colour bands. In fact, it may be more sensible to base the break points on flooding conditions in the floodplain, eg, when an area becomes inundated or when there will be substantial above floor flooding.

As shown in Figure 11-2, the flood totems should show floods larger than the 100 year ARI. This should be done to ensure that the flood warning system is able to cope with such floods and that there is an understanding in the community that larger floods do occur. The largest flood analysed for this study is the 100 year ARI event. Consideration should be given to analysing larger floods.

There are some areas on the floodplain where this approach may not be suitable without some further refinement of the system. Innisfail itself is one such example. Innisfail is effectively an off-river storage during floods, ie, it fills up once the river overtops its banks, initially at Sweeneys and Saltwater Creeks. Therefore, the peak flood height in Innisfail is dependent on both the river height and the duration that floodwaters are flowing into Innisfail. In its simplest form, the totem system does not account for the duration of flooding, but it may be possible to develop a system that incorporates duration using the BoM predicted hydrograph.

In addition to the current telephone flood line, an automated telephone warning system as has been proposed for flood warning elsewhere in Australia could be considered. This automated system is not available as a service from Telstra. It appears that the system is a commercially available package that supports a variety of functions. One function is the ability to automatically dial a list of pre-recorded telephone numbers. This list could be divided into different areas in the catchment. When the call is answered a recorded message is played. For example, if the call is made from Innisfail, the recorded message may state something similar to:

"*Hello. This is a CDC Flood Warning Recorded Message. A blue flood is expected to peak in Innisfail at 3am tomorrow morning. Repeating... A blue flood is expected to peak in Innisfail at 3am tomorrow morning. For more information please tune to Radio Station on 4KZ on frequency xxx or call the flood information line on 4776 xxxx.*"

Following completion of all calls, the package appears to be able to wait a designated period before dialling the unanswered numbers again. It appears that the automated telephone package is also able to answer incoming calls as a flood information line and provide further recorded details on the expected flooding.

The Committee's recommendation is that flood totems and a review of the public warning system be included in a Floodplain Management Scheme.

10.1.5.2 Community Support During Floods

10.1.5.2.1 Status Quo

The CDC and SES have good procedures in place for responding to community needs during floods based on many years of responding to flooding. However, as noted previously, the procedures are based on experience of floods in the range 20 to 40 years ARI.

10.1.5.2.2 Recommendations

It is recommended that the CDC review the data that will be available from this study, especially for larger floods, to ensure that the planning is adequate for flood events larger than previously experienced. For example, it is recommended that the SES review evacuation triggers and safe evacuation areas, especially in larger floods. These could be related to the colour classification system. For example, areas or buildings would be identified that could be used for evacuation centres. These areas or buildings would also be linked to the colour classification system. An example from another floodplain in NSW is given in Figure 11-3.

Although it has not been determined as part of this study, consideration should be given to an assessment of the probable maximum flood (PMF). This is an extreme event, but the CDC should incorporate this size event into their counter disaster plan. The focus of disaster management during a flood of this magnitude should be on saving lives rather than property.

The Committee's recommendation is that the Floodplain Management Scheme should require that the CDC undertake a review of the data from the study to ensure that planning is adequate for larger flood events and that flood totems be implemented.

10.1.6 Summary of Response Modification Recommendations

The Steering Committee has recommended that:

- alarms be included on the Nerada and Corsi Alert Stations and that the inclusion of alarms at other strategic locations be investigated;
- colour classification of floods and flood totems be included in a Floodplain Management Scheme;
- a review by the CDC of the benefits of emergency maps be included in a Floodplain Management Scheme;
- the CDC improve its current procedures for disseminating information to the public.
- the CDC undertake a review of the data from the study to ensure that planning is adequate for larger flood events.

10.2 Raising Community Awareness

10.2.1 Description

The aim of raising community awareness of flooding is to minimise the psychological and monetary damage caused by flooding by increasing the level of preparedness of the community. If people are aware that they reside in a flood prone area and that it is possible that their homes and/or businesses may be inundated by a major flood, they are likely to react appropriately if a flood occurs. Conversely, if people are not aware of the seriousness of flooding in the area, they are unlikely to take flood warnings seriously, thus placing themselves and their property at risk. Furthermore, they may even place others at risk by hampering SES flood response efforts.

It is important to ensure that people are aware that they live in a flood prone area and that floods can cause serious damage to property and can endanger the lives of people and animals. Informed residents are less likely to be caught unaware if a flood occurs and will be more likely to make their own flood response plans (e.g. organising for furniture to be moved to a safe location).

In the Johnstone region, many people come from families that have resided in the area for several generations. In most cases, these people have either experienced a flood or have heard first hand accounts of floods from family members or friends. Therefore, they are likely to have a high level of flood awareness. However, these people may not be aware that there may be larger floods than those events that they have experienced or heard of. In addition, there are a significant number of new rural and urban residents in the region who may not have the same level of flood awareness. In some instances, these people:

- have not experienced a flood in the area;
- have not heard first hand accounts of previous floods;
- live in houses that are not near the river, but are actually in the floodplain and are subject to flooding; and/or
- are not likely to take flood warnings seriously.

Both groups of people, those who have a low level of flood awareness and those who may not believe that there will be a larger flood than the biggest historical flood, that should be the target of a flood information campaign.

10.2.2 Flood Awareness Campaign

An integrated flood awareness campaign should be initiated with the aim of increasing the public's knowledge of flooding in the region. Such a campaign is most likely to be a success if it conveys simple messages that can be reinforced and reiterated by all facets of the public relations exercise. It would be important to use clear language and explain terminology that may confuse people. For example, it would be necessary to explain that a 100 year flood event is a flood event that has a 1% chance of occurring in a given year.

The flood awareness campaign could utilise two different categories of messages:

- **General Messages** - messages that relate to the whole community and could be conveyed via public media (e.g. newspapers); and
- **Specific Messages** - messages that address the susceptibility of individual households to flooding and could be conveyed via private media (e.g. individual household packages).

10.2.3 General Messages

The general messages that are relevant to the entire Johnstone community could include:

- many areas of the Johnstone region are flood prone;
- floods can cause serious damage to property and can endanger the lives of people and animals;
- there are different categories of floods and the impacts of these different types of floods vary;
- a Floodplain Management Plan has been developed to help reduce the damage caused by floods;

- the Plan will only be effective if community members are willing to cooperate and act; and
- more detailed information about flooding in the Johnstone region is available from the SES.

A number of different methods and media could be utilised to help convey these messages. If the colour classification of floods is adopted, the colour scheme could be used in all aspects of floodplain management in the region, thereby ensuring that there was consistency in the message being conveyed. This would allow residents to become familiar with the terminology being used to describe the magnitude of floods.

Some of the initiatives that could be utilised to convey the general messages are listed below.

- **Slogan -** a simple slogan that could appear on signs, booklets, stickers etc.
- **Flood Signs -** showing the colour-coded flood bands and the heights of previous floods. These could be erected along the riverbank (e.g. next to bridges) and could include photographs of previous floods at that location.
- **Historical Displays** – Similar to the flood signs but perhaps more regional in focus. An obvious place in Innisfail would be at the wharf (the display would need to be flood proof or replaced after floods!).
- **Totem Poles -** showing the colour-coded flood bands (refer to 10.1.5). In order to encourage community acceptance, it is recommended that flood totem poles do not include any information on historical floods nor any signage indicating their relation to flooding to reduce vandalism. This will help to reduce the community feel that they being publicly labelled as "flood prone". This places a greater reliance on leaflets and advertising to ensure understanding of flood totems significance.
- **Flood Awareness Leaflets -** containing general flooding information, including an explanation of the colour-coded flood bands. This could be a separate leaflet or an expansion (and renaming) of the current Cyclone Advice Booklet. A leaflet could be sent to homes on a regular basis (e.g. sent out with the rates notice every few months), or the booklet distributed say once a year to every home.
- **Flood Awareness Week -** a week of the year (preferably at the start of Summer) devoted to promoting flood awareness. Features on flooding, including dramatic photographs of previous floods, could be run in the local newspapers. Local radio stations could hold competitions with a flood theme etc. Flood awareness workshops could be held with flood wardens during this week. Guided tours could be run showing historical flood marks, mitigation systems and flood warning systems.
- **Flood Education in Schools -** provide schools with information kits and activities that are designed to increase flood awareness. This could be coordinated with the Flood Awareness Week.
- **Web Site** – The JSC web-site should include flood awareness and flood warning information.

The contact details of the SES would be provided on the flood signs and the leaflets for people who wanted to find out more information about flooding in the Johnstone River region.

10.2.4 Specific Messages

The aim of the specific messages would be to inform people of whether their house and/or business is located in a flood prone area and answer questions such as: "Is my home really at risk of being inundated by a flood?" Diagrams could be generated, similar to Figure 11-4, which use floor level, ground level and flood level data to generate flooding information that is specific to individual buildings; this would require floor level survey. If people can see that the 1974 flood would have resulted in their house being inundated, they are likely to react seriously to flood warnings and follow the advice of the SES. Specific messages would only need to be conveyed to people who own buildings that are at risk of being inundated (i.e. within the Probable Maximum Flood (PMF) extent – not determined as part of this study).

These specific messages could be conveyed to residents by way of a household flood awareness package. Such a package could be hand delivered by SES personnel, as this would have the added advantage of giving the SES personnel a better understanding of the area so that door to door delivery of flood warning messages would be more efficient. A procedure that ensured that all new residents received a package would need to be instigated.

The flood awareness household package could contain the following information.

- **Flood Information Brochure -** including information about the history of flooding in the region, an explanation of why the household packages are being distributed, what the colour-coded flood bands represent and general information about what to do before, during and after a flood. Flood warning and emergency planning is covered in more detail in Option 12. The contact details of the SES would be provided for people who wanted to find out more information about flooding.
- **Household Flood Diagram -** a basic diagram, similar to that depicted in Figure 11-4, showing the floor level of the building in relation to the height of the colour-coded flood bands and the height of previous floods. The largest flood assessed for this study is the 100 year ARI event. It is recommended that the maximum level indicated on such a diagram be larger than this, for example, the 500 year flood level could be the maximum level indicated. This information would be specific to the location of the building.
- **Stickers/Fridge Magnets/Rulers -** a range of items stamped with the campaign slogan to promote flood awareness.

10.2.5 Recommendations

Raising community awareness is considered to be essential. A higher level of flood awareness can:

- reduce loss of life and injuries during major floods;
- reduce psychological trauma;
- reduce monetary damages;
- increase effectiveness of evacuations;

- increase effectiveness of SES operations.

The Committee's recommendation is to incorporate a Flood Awareness Campaign into a Floodplain Management Scheme. The campaign will convey general and specific messages and will utilise some of the tools described in this section.

11 SUMMARY OF FLOOD MANAGEMENT MEASURES

A detailed assessment of existing flood behaviour and flood management measures has been undertaken. The assessment included a review of existing Flood Modification, Property Modification and Response Modification measures. A range of measures was considered and the Steering Committee has recommended that a number of these measures be incorporated into a Floodplain Management Plan. These recommendations are summarised in the Floodplain Management Plan (WBM, 2003) along with costs and an implementation strategy for each measure.

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APPENDIX A: COST BREAKDOWN OF FLOOD MODIFICATION MEASURES

Town Levee Construction Costs

	Frith Rd		Sweeneys Ck		Saltwater Creek		Scullen Avenue	
Earth Levee Construction	$	9,570	$	8,143	$	9,962	$	45,691
Concrete Levee Construction	$	0	$	3,579	$	0	$	0
Treatment of Concrete Levee	$	0	$	614	$	0	$	0
Seal footpath	$	0	$	4,702	$	0	$	0
Pipes with Flapgates	$	5,000	$	0	$	0	$	20,000
Landscaping	$	5,163	$	2,782	$	6,137	$	17,137
Sub-Total 1	$	19,733	$	19,823	$	16,099	$	82,828
Contingencies (25%)	$	4,933	$	4,956	$	4,025	$	20,707
Sub-Total 2	$	24,677	$	24,779	$	20,124	$	103,536
Additional Items* (12%)	$	2,960	$	2,974	$	2,415	$	12,424
CONSTRUCTION TOTAL	**$**	**27,627**	**$**	**27,753**	**$**	**22,539**	**$**	**115,960**
Mowing and Gardening	$	7,522	$	1,857	$	13,310	$	16,401
Community Education	$	0	$	55,203	$	0	$	0
MAINTENANCE TOTAL	**$**	**7,522**	**$**	**57,060**	**$**	**13,310**	**$**	**16,401**
Levee Monitoring incl. Annual Survey	$	4,701	$	4,298	$	8,318	$	10,251
OPERATION TOTAL	**$**	**4,701**	**$**	**4,298**	**$**	**8,318**	**$**	**10,251**
TOTAL	**$**	**39,850**	**$**	**89,111**	**$**	**44,167**	**$**	**142,612**

*includes survey, design and construction admin = 7.5%, geotechnical investigation = 1.5%, administration on-cost = 3% - values sourced from SKM (1999)

Carello's Channel Construction Costs

Clearing & Grubbing	$ 12,000
Excavation	$ 2,500,000
Rock Walls	$ 1,771,000
Sub-Total 1	$ 4,283,000
Contingencies (10%)	$ 428,300
Sub-Total 2	$ 4,711,300
Resumptions	$ 12,000
Sub-Total 3	$ 4,746,300
Additional Items* (12%)	$ 569,556
CONSTRUCTION TOTAL	**$ 5,315,856**
MAINTENANCE TOTAL	**None allowed**
OPERATION TOTAL	**None Allowed**
TOTAL	**$ 5,315,856**

*includes survey, design and construction admin = 7.5%, geotechnical investigation = 1.5%, administration on-cost = 3% - values sourced from SKM (1999)

Bill of Quantities for Scoured Carello's Channel

Item	Unit	Rate	Quantity	Total
Resumption	Ha	$7500	4.2	$31,500
Removal of levee	m^3	$10	1200	$12,000
Construction of new levee	Lump			$70,000
Maintenance of new levee	Lump			$22,000
Excavation of nick	m^3	$10	9,000	$90,000
Sub-total 1				**$225,500**
Contingencies	%	25		$56,375
Sub-total 2				**$281,875**
On-costs (design, survey and admin)	%	12		$33,825
Geotechnical and Hydraulic Investigations	Lump			$45,000
Total				**$360,700**

Webb Levee Construction Costs

Earth Levee Construction	$	61,759
Concrete Levee Construction	$	284,638
Treatment of Concrete Levee	$	52,001
Pipes with Flapgates	$	5,000
Landscaping	$	18,477
Sub-Total 1	$	416,874
Contingencies (25%)	$	104,219
Sub-Total 2	$	521,093
Additional Items* (12%)	$	62,531
CONSTRUCTION TOTAL	**$**	**583,624**
Mowing and Gardening	$	16,617
Community Education	$	55,203
MAINTENANCE TOTAL	**$**	**71,820**
Levee Monitoring incl. Annual Survey	$	17,089
OPERATION TOTAL	**$**	**17,089**
TOTAL	**$**	**672,533**

*includes survey, design and construction admin = 7.5%, geotechnical investigation = 1.5%, administration on-cost = 3% - values sourced from SKM (1999)

Dredging Costs

Scenario 1 – Dredge to Stockpile and Sell to Market in Cairns	
Dredge to Stockpile	\$ 4.50/m^3
Wash	\$ 2.50/m^3
Transport to Cairns	\$20.00/m^3
Market Rate	\$15.00/m^3
Assumed silt content	10%
Scenario 2 – Dredge to Stockpile and Sell Locally	
Dredge to Stockpile	\$ 4.50/m^3
Market Rate	\$ 1.00/m^3
Scenario 3 – Dredge to Spoil	
Dredge to Stockpile	\$ 4.50/m^3