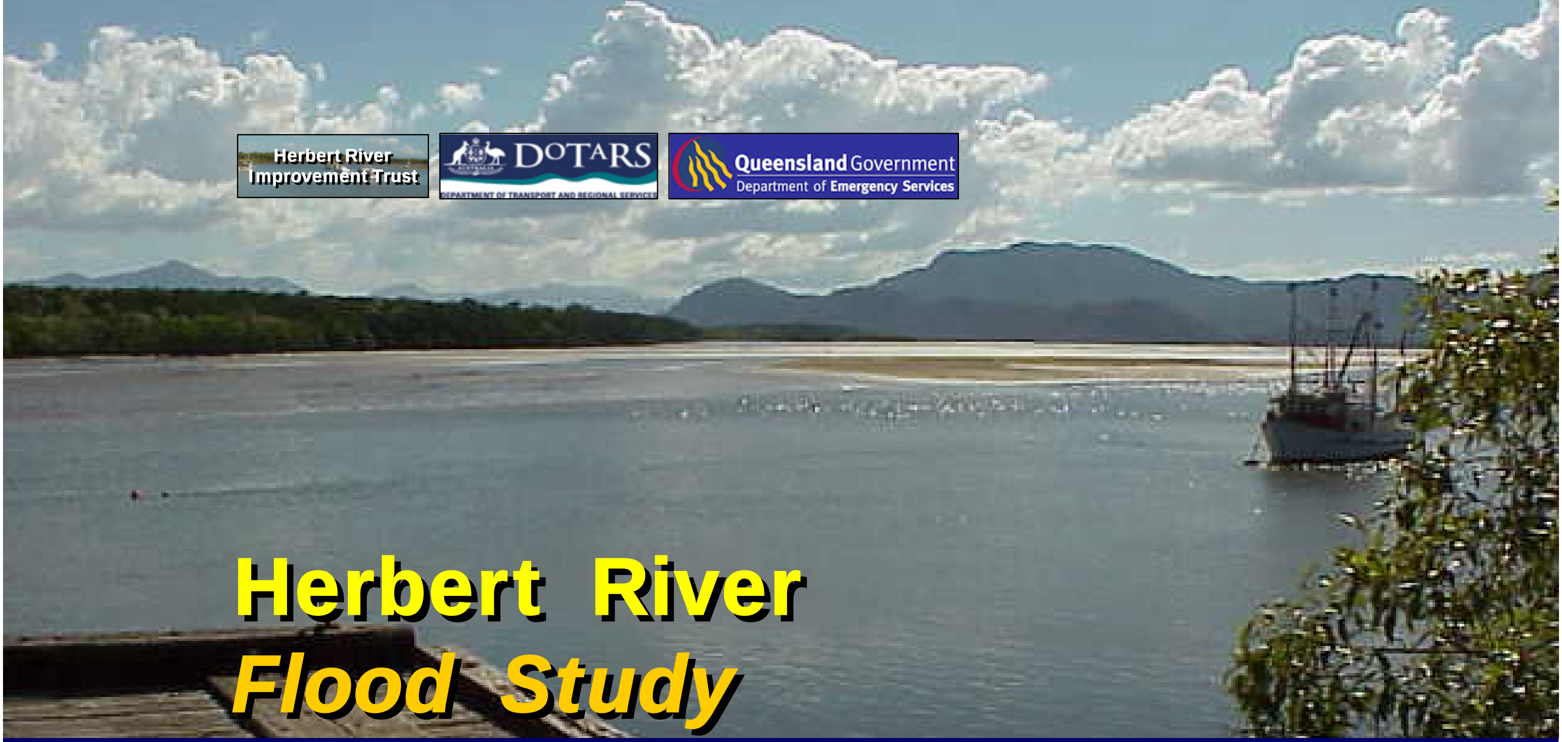
Herbert River Improvement Trust

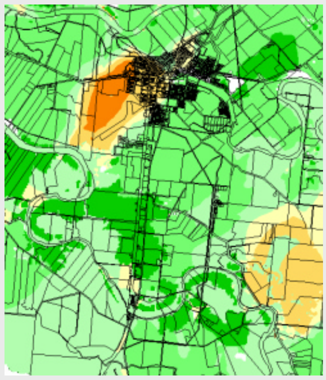
DOTARS Department of Transport and Regional Services

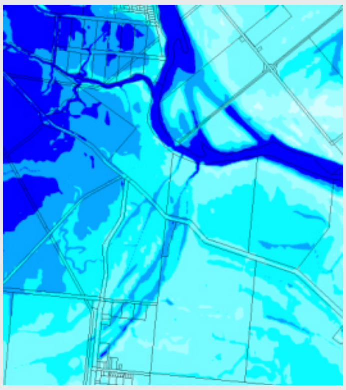
Queensland Government Department of Emergency Services



Herbert River *Flood Study*

Main Report - Volume 1 of 2





WBM OCEANICS AUSTRALIA

Herbert River Flood Study

Main Report

Volume 1 of 2

Prepared For:	Herbert River Improvement Trust
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	Title:	Herbert River Flood Study – Main Report - Volume 1 of 2
	Project Manager:	Mark Jempson
	Author:	Mark Jempson and Malcolm Jenkins
	Client:	Herbert River Improvement Trust
	Client Contact:	Ross Hogan
	Client Reference:	
	Synopsis:	This document details the Herbert River Flood and Floodplain Management Study. On the basis of this Study, the Floodplain Management Study Advisory Group recommends the inclusion of some of the options presented here for inclusion into the Floodplain Management Plan. The Plan exists as a separate document.

REVISION/CHECKING HISTORY

REVISION NUMBER	DATE	CHECKED BY		ISSUED BY	
0	05/12/02	W. Syme		M. Jempson	
1	06/06/02	C.Barton		M. Jempson	

DISTRIBUTION

DESTINATION	REVISION										
	0	1	2	3	4	5	6	7	8	9	10
Herbert River Improvement Trust	10	40									
BoM	1	0									
WBM File	1	1									
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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 Herbert River floodplain and funding has been obtained through this program to develop a Floodplain Management Plan.

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 Behaviour Definition	The nature and extent of the flood problem are determined.
2. Floodplain Management Measures Investigation	Management measures 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 Herbert River. It defines the existing flooding problem and assesses a range of measures and their ability to reduce the impact of flooding in the Herbert River area by controlling the flood risk and reducing flood damages. The impact on flooding of a number of past floodplain works are also assessed. Stage 3 is reported in a separate document (WBM, 2003), which summarises the preferred management measures identified in this report.

WBM Oceanics Australia was commissioned by the Herbert River Improvement Trust (HRIT) 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 annual average damage is the average damage 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 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).
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).
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	Digital Elevation 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 measures 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 measures aimed at reducing the flood risk. The plan is the principal means of managing the risks associated with the use of the floodplain. The plan usually contains 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 flood levels and a freeboard. 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. 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. 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.
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 flood level thus determing a flood planning level. Freeboard tends to compensate for factors such as wind/boart wave action, localised hydraulic effects and uncertainties in the design flood levels.

historical flood A flood that has actually occurred.

hydraulics 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 or water level 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 at a particular location.

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 and one 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 if a one-dimensional solution is used; and depth average if a two-dimensional solution is used..

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	Digital Elevation 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 Relevant Activity
FPL	Flood Planning Level
GIS	Geographic Information System
HRIC	Herbert Resource Information Centre
HRIT	Herbert River Improvement Trust
HSC	Hinchinbrook Shire Council
km	kilometre
m	metre
m^3/s	cubic metres per second (same as cumecs)
m AHD	Elevation in metres relative to the Australian Height Datum
PMF	Probable Maximum Flood
QGR	Queensland Government Rail
SES	QLD State Emergency Services

1 INTRODUCTION

1.1 Background

The Herbert River is a major river at the southern end of the Wet Tropics in North Queensland. Drawing 1-1 shows the locality of the river system and Drawing 1-2 shows key features of the Lower River. The catchment covers nearly 10,000 km^2 with its headwaters near Herberton on the Dividing Range. The River drains much of the western areas of the Atherton Tablelands west of the Wet Tropics. The Lower River traverses an alluvial distributary plain formed by frequent and major flooding and associated sediment deposition. There is a history of severe flooding on the floodplain with considerable damage to property, agriculture and public infrastructure.

In living memory, flooding in the Lower River has predominantly been the result of high rainfalls in the lower part of the catchment below Gleneagle. One exception is the 1967 flood which was a result of high rainfall in the upper catchment. This flood is well remembered in the community for its size and lack of rainfall in the lower catchment. Flooding in the Lower River can be influenced by the magnitude and timing of flows from the Stone River, a tributary with its confluence upstream of Trebonne. In large floods, the Stone River will break its banks upstream of Trebonne and flow into Trebonne Creek and Cattle Creek.

There are a number of townships and communities located on the floodplains in the Lower Herbert including Trebonne, Ingham and Halifax, Macknade, Bemerside, Cordelia, Blackrock and Toobanna. The floodplain is within the Hinchinbrook Shire, which is based in Ingham. The river geomorphology naturally distributes flood flows from the channel, it has less than 5 year ARI flow capacity downstream of Ingham, and has natural levees perched above the floodplain. As an example, by the time the flood peak reaches Ingham in a 100 year ARI flood, about 50% of the flow is distributed to the floodplain, by the time it is downstream of the Gairloch Washaway about 66% is distributed to the floodplain and at about 5 km downstream of the Halifax Bridge, about 90% of the flow is distributed to the floodplain.

Of the centres listed above, Halifax is most at risk from flooding. Halifax is located on the eastern bank of the Herbert River about 10 km upstream from the river mouth at Hinchinbrook Channel and is the most downstream of the towns on the Herbert River. Floodwaters regular inundate in and around Halifax. There are concerns within the Halifax community that, for the same size flood upstream, flooding is now worse in Halifax than previously.

During major flood events, water flows through the majority of Ingham and into a number of creek systems to the south with evacuations required in some parts. Initially floodwaters backup from the Herbert River through Palm Creek and Kingsbury Creek and then, in larger floods, there is significant overtopping of the river banks near Ingham. In larger floods, Trebonne Creek may also contribute to flooding in the southern parts of Ingham.

Flooding is not just an issue limited to the townships. Flooding from the lower Herbert River has played an important role throughout the development of Ingham and its surrounding region that supports a sugar cane industry with an economic value of approximately $250million. There are very few areas where sugar cane is grown that could be considered flood free. On the northern side of the river, the Ripple Creek and Seymour River catchments act as significant overflows during Herbert River flooding. To the south, Cattle, Lucy Palm and Victoria Creeks all act as outlets to the ocean during Herbert River flooding.

The community in this region is well aware of flooding issues and has strong views on flood management and mitigation issues. Flooding has also been a significant influence on local government politics and social issues in Hinchinbrook Shire. The community was given opportunities through a community consultation program to contribute to and understand the study process.

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 filling at the town dump, construction of Tyto Wetlands, the construction of floodgates on Ripple and Catherina Creeks and the construction of a private levee at the Halifax Washaway. This levee is known in the community as Mombelli's Levee. The flood model was used to quantify these impacts.

The study was not limited to hydraulic considerations. The river system supports substantial sediment loads during flood events. Sand is supplied from the upper catchment and deposited/ reworked in the lower estuary and adjacent beaches as part of the natural processes. Erosion and deposition of sediments have a significant influence on the stability of the bed and banks of the river and alter the flood hydraulic characteristics. A preliminary overview assessment of the geomorphological processes operating in the river was undertaken. The principal reason for this study component was to determine the need for a more detailed sediment transport study of the Herbert River, primarily related to major extraction proposals near Ingham and shoaling issues in the lower reaches near Halifax.

1.2 Study Area

The study area extends along the river and floodplain from Trebonne through to the ocean. On the floodplain to the south, the area of interest extends to Toobanna. It will be seen in Section 4 that the hydraulic model covers a larger area than defined here. This was done to ensure the reliability of the results in the area of interest.

1.3 Previous Studies

In 1980, Cameron McNamara completed a major flood management study for the Herbert River Improvement Trust (HRIT) and the Queensland Water Resources Commission (QWRC). However, in the 1985 Council elections, the sitting Council was removed from office, the key issue being the Council's implementation, without effective community consultation, of a 10 point Flood Management Plan developed from the recommendations of the report. As a result, the Flood Management Plan was only partially implemented.

Maunsell McIntyre (1999) completed an investigation into the extension of the Halifax Levee for the HRIT. Extending the levee would provide a similar level of flood protection to the southern part of Halifax to that currently afforded by the existing levee along the northern part of Halifax. The study was undertaken using a simple flood modelling approach over a localised area of the floodplain. Although the study concluded that a low-level, reasonably cost-effective levee would improve the flooding immunity for many residents with very few discernible disbenefits, the HRIT was advised at the completion of the investigation that, '*the approval in isolation of any particular levee without the context of a "whole-of-floodplain community" levee plan is not supported*'.

This was the impetus for the funding application to the Natural Disasters Risk Management Studies Program to undertake a new whole-of-floodplain Floodplain Management Study. While the 1980 study was advanced for its time, significant improvements have been made in computer modelling techniques since then. More flooding information is now available than in the past, due to intense flood height gathering during and after major events, by Hinchinbrook Shire Council's survey team and better data, including ground contour information, is now available.

1.4 Objectives

The objectives of the study were to:

- develop a state-of-the-art computer model of the Herbert River within the study area to determine flood risks, threats and impacts in the lower flood plain;
- develop strategies to negate these impacts, including a levee rationalisation strategy, zoning options and improved warnings and predictions;
- review current mitigation methods and make recommendations about future protection measures with consideration given to social, ecological and economic factors;
- carry out a preliminary investigation into sediment transport in the Herbert River, to enable an assessment to be made on the need to embark on a major sediment budget study for the total catchment of the Herbert River;
- prepare a report detailing the development of the model, the assessment of the effect of existing development and flood mitigation measures and detailing proposed flood mitigation measures;
- prepare a Floodplain Management Plan.

1.5 Study Advisory Group

The Herbert River Improvement Trust formed a Study Advisory Group (SAG) to oversee the Floodplain Management Study and to ensure that issues important to the Herbert River community have been addressed. The SAG comprised:

- community representatives;
- environmental group representatives;
- local Councillors;
- Council representatives;
- State Government representatives from the Department of Natural Resources and Mines and Department of Emergency Services.

The SAG has an important role in advising the Trust and Council 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 SAG and the community.

Throughout the study, regular meetings were held in Ingham with the SAG 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 SAG decided whether individual measures were to be incorporated into a Floodplain Management Scheme.

1.6 Community Consultation

An important aspect of the study was the Community Consultation program. It gave residents of the valley opportunities to provide personal experiences to WBM, to confirm flood patterns and heights, to suggest specific matters for investigation during the study and to provide input on management measures. It ensured that the community was involved in the study from the outset and developed a sense of ownership of the study.

2 STUDY APPROACH

There were seven key phases in this Flood Study and Floodplain Management Study.

A. Data collection

B. Flood Model Development and Calibration

C. Design Flood Analysis

D. Assessment of Past Floodplain Developments and Sediment Transport Assessment

E. Assessment of Management Measures

F. Finalise Management Measures

G. Reporting

The remainder of Section 2 outlines the adopted approach for each of these phases. A detailed description of each of the phases 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 topographic basis of the hydraulic model. Data for the DEM was obtained from existing photogrammetry, ground survey and river cross-sections. Extensive GIS data sets were obtained including cadastral data, aerial photography, land use 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 Hebert River floods and to identify local concerns within the region. The local knowledge of the flooding in the towns and the surrounding floodplains was found to be invaluable.

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 SAG 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 Hinchinbrook Shire Council and residents during the resident survey. Historical ocean tide data was required and was obtained from the Environmental Protection Agency (EPA).

2.2 Community Consultation

The consultation program was multi-faceted using community open sessions, a resident survey, media releases, a web page, posters and brochures and a telephone hot-line. Five series of community opens sessions were held at the 5 key milestones in the study listed below. Each series of open sessions comprised four sessions, two in Ingham and two in Halifax.

1. Study Commencement - Presentation of study team and flood behaviour & issues workshop.
2. Model Calibration - Presentation of preliminary calibration and historical works workshop.
3. Design Floods – Presentation of design floods and flood management workshop.
4. Flood Management Measures – Presentation of analysis of management measures & SAG decisions.
5. Study Closure - Presentation of Floodplain Management Plan.

2.3 Flood Model Development & Calibration

The flood model comprises a hydrologic 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. These hydrographs then become a key input into the hydraulic model.

The hydraulic model simulates the movement of floodwaters through waterway reaches, storage elements and hydraulic structures. The hydraulic model calculates flood levels and flow patterns, and 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 Herbert 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 and 1D dynamic hydraulic modelling system was adopted. In total, the hydraulic model covers more than 900 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 using the February 1992 and the March 1967 historical flood 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.

2.4 Design Floods

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 from flood events larger than the 100 year event.

Design flood levels, flows and velocities were determined for the 5, 10, 20, 50 and 100 year ARI floods and the Probable Maximum Flood (PMF); the PMF is an extreme flood deemed to be the maximum flood likely to occur. The approaches available to determine the inflow boundaries for the hydraulic model for each of these flood events were as follows:

1. The URBS hydrologic model - design rainfall events and temporal patterns are derived from AR&R (2001);

2. As for 1., but hydrographs factored to match peak flows derived from a flood frequency analysis;

3. Synthetic hydrographs developed from historical floods factored to match peak flows derived from a flood frequency analysis;

4. A combination of the above.

Option 4 was adopted for the 5 year through to 100 year ARI design floods and option 1 was adopted for the PMF.

2.5 Assessment of Past Floodplain Developments

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

- Mombelli's levee;
- Floodgates on Catherina & Ripple Creeks;
- Filling of the Town Dump;
- Tyto Wetlands;
- Road & Rail embankments.

2.6 Sediment Transport Assessment

A preliminary overview assessment of the geomorphological processes operating in the river was undertaken. The principal reason for this study component was to determine the need for a more detailed sediment transport study of the Herbert River, primarily related to major extraction proposals near Ingham and shoaling issues in the lower reaches near Halifax.

To gain an understanding of past trends of erosion and deposition a review of historical trends was undertaken by assessing historical aerial photography, surveyed river cross-sections and previous reports. This gives an insight into the relative nature and extent of changes and the degree of sediment transport.

A preliminary assessment of the transport potential of the river system was assessed through reviewing previous reports and undertaking preliminary calculations of sediment transport rates (bed material load) at various locations along the river.

The results from the historical review and the preliminary assessment of the transport potential provided a basis for comments on shoaling issues and impacts of sand extraction.

2.7 Evaluate Potential Floodplain Management Measures

Both structural and non-structural floodplain management measures were assessed. Structural measures are those measures that alter flood behaviour, eg. levees and diversion channels. Non-structural measures include development controls, voluntary house purchase, house raising and flood warning and emergency planning. Input for potential structural measures was sought from both the community and the SAG. The assessments considered hydraulic impacts, economic benefits, intangible benefits and environmental considerations.

2.8 Finalise Management Measures

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

2.9 Reporting

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

This Draft Flood Study Report will be presented to the SAG and feedback is sought before issuing the Final Flood Study Report. At this stage a Floodplain Management Plan will be issued. The Floodplain Management Plan will be a simple, easy to read and view document, using maps and plans to illustrate the preferred scheme. It will be designed for the lay-person and as a central tool for Council's day-to-day floodplain management activities.

3 DATA COLLECTION

3.1 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:

- Herbert Mapping Project – 1:20,000 photogrammetry flown in 1994
- Ingham Town Mapping – 1:4,000 photogrammetry flown in 1998
- Halifax Town Mapping – 1:4,000 photogrammetry flown in 1997
- 1992 river cross-section survey of the Herbert River and Anabranch undertaken as part of a long-term monitoring of the river – at some locations the latest survey available was before 1992;
- 1999 river cross-sections survey along the reach at Halifax undertaken for Maunsell McIntyre (1999);
- limited surveyed bed level data in lower tidal reaches of Herbert River and Victoria Creek – spot levels only, not full bathymetric survey – carried out in 2001 for this project;

The quoted vertical accuracy of the Herbert Mapping Project is 80% of points within ± 0.5 m. Peter Mowat, the HSC surveyor, advised in personal communications that checks undertaken by HSC have indicated a vertical accuracy of about ± 0.1 m on open surfaces. The photography was flown at the end of the crushing season when the majority of the floodplain was clear of any significant vegetation. The vertical accuracy of the Ingham and Halifax photogrammetry is ± 0.125 m.

The development of the hydraulic model required ground survey, undertaken in 2001, of the following features:

- road and rail drainage infrastructure data;
- road and rail top of embankment/rail level;
- Mombelli’s levee;
- Castorina’s Levee;
- town dump;
- Tyto Wetland;
- top of bank survey upstream of and near Ingham and at the Halifax Washaway.

Bridge plans were obtained from Department of Main Roads (DMR) road and Queensland Government Rail (QGR). Extensive top of bank survey in the lower reaches undertaken by the Department of Natural Resources and Mines (DNRM) in 1992 was also obtained.

3.2 Resident Survey and Site Inspection

In May 2000, a detailed site inspection of the Lower Herbert River system and floodplain was undertaken by Duncan Thomson from WBM and Max Fenoglio from the HRIT to identify key features that have a significant influence on flood behaviour. The site inspection included a four-day on-the-ground resident survey involving personal interviews with 23 residents.

Further input was obtained from the SAG and the community at the SAG meeting and community open sessions in May 2000. 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;
- 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;
- historical changes to topography (for example, raising of roads, building of floodgates) were discussed;
- an opportunity for residents to discuss issues of concern.

Through the course of the study additional site inspections were undertaken in conjunction with visits to attend SAG meetings to further investigate areas that became the focus of attention.

3.3 GIS Data Sets

An extensive GIS data set was obtained from the Herbert Resource Information Centre (HRIC) including:

- digital orthophotos;
- cadastral data;
- historical flood levels;
- landuse mapping;
- HSC, DMR, and QGR hydraulic structures;
- CSR rail assets.

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 (BoM), the Department of Natural Resources and Mines (DNRM), the HRIC, the HSC, the EPA and residents. The data collected is summarised in Table 3-1.

Table 3-1 Summary of Historical Flood Data Collected

Data	Source
Peak flood levels for the 1967, 1972, 1977, 1986, 1990, 1991, 1994, 1997, 1998, and 1999 floods. Location was referenced in MGA 1994 coordinate system.	HRIC and HSC
Historical rainfall data for floods in 1967, 1972, 1977, 1980, 1981, 1986, 1990, 1991, 1994, 1996, 1997, 1998, 1999, 2000 and 2001.	BoM - with the URBS hydrological model
Continuous river height and flow (at some locations) data for floods in 1967, 1972, 1977, 1980, 1981, 1986, 1990, 1991, 1994, 1996, 1997, 1998, 1999, 2000 and 2001. Main locations of interest included Abergowrie, Abergowrie Bridge, Ingham Pump Station and Gairloch. Data was supplied at other locations as well.	BoM - with the URBS hydrological model DNRM
Anecdotal flood behaviour	Residents and SAG
Historical ocean tide data	EPA

4 FLOOD MODEL DEVELOPMENT & CALIBRATION

4.1 Background

Computer models of the Lower Herbert River catchment and its floodplains were developed to define flood behaviour to provide a benchmark against which flood management measures were assessed, and to assess past floodplain works, flood hazards and flood damages.

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 which 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.

This Section describes the development and calibration of the models to known flood events and the assessment of design floods.

4.2 Hydrologic Model

The Bureau of Meteorology (BoM) has two established and calibrated URBS hydrologic models of the Herbert River catchment; one model represents the upper catchment and the second the lower catchment. The models were supplied to WBM for this study. The upper catchment model comprises 73 sub-areas and the lower catchment model has 58 sub-areas. The boundary between the two models is at Gleneagle Homestead. For the purposes of this study, WBM modified the lower catchment model by adding an additional 19 sub-areas covering the lower part of the floodplain. The majority of these sub-catchments do not drain into the Herbert River but were required to provide localised inflows over the floodplain. The full catchment and model discretisation is shown in Drawing 4-1.

The BoM has calibrated the models to about 20 flood events dating back to 1955, including all major flood events in the last few years. The models utilise recorded rainfall data from 10 rainfall stations in the lower catchment and 6 in the upper catchment, and river height data at 11 gauges in the lower catchment and 4 in the upper catchment.

A review of the URBS models, in consultation with BoM, revealed that the model is well calibrated to some flood events and poorly calibrated to others. The likely reason for the variation in the success of the calibration is the difference in the temporal distribution of the rainfall between flood events in relation to the location of the rainfall stations and the availability of rainfall data. The calibrations have typically improved in more recent years with the addition of more rainfall stations in the catchment.

For reasons discussed in Section 4.6, flows from the URBS model for the catchment above Abergowrie were not used for the calibration of the hydraulic model. The URBS model flows were used to provide local catchment inflow boundaries below Abergowrie.

4.3 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 2D / 1D modelling system was adopted: further information on TUFLOW modelling system is given in 4.4. In total, the model covers approximately 940 km^2 of the river and floodplain. The extent of the model and the 2D and 1D (quasi-2D) model areas are shown in Drawing 4-2.

The model is based on a 40 m square grid. Each square grid element contains information on ground topography sampled from the DEM at 20 m spacing (3 per side and one at the centroid of the grid), surface resistance to flow (Manning's n value) and initial water level. Details of the DEM are provided in Section 4.5. Eighteen areas of different land-use type, determined from aerial photography and site inspections, were identified for setting Manning's n values.

Following are the inflow boundaries adopted for the TUFLOW model.

1. Abergowrie (Herbert River)
2. Elphinstone Creek
3. Dalrymple Creek
4. Hawkins Creek
5. Ripple Creek
6. Arnot Creek
7. Lannercost Creek
8. Stone River
9. Cattle Creek
10. Frances Creek
11. direct rainfall on area covered by model

The location of the above boundaries (except for 11) are shown in Drawing 4-2.

Large 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. On the large bridges, the decks were modelled as dynamically nested 1D broad crested weirs to allow flow over the bridge. Smaller bridges and culverts were modelled as dynamically nested 1D structures.

4.4 TUFLOW Hydrodynamic Modelling System

TUFLOW solves the full 2D shallow water equations based on the scheme developed by Stelling (1984). The solution is based around the well-known ADI (alternating direction implicit) finite difference method. A square grid is used to define the discretisation of the computational domain.

Improvements to the Stelling (1984) scheme, including a robust wetting and drying algorithm and greater stability at oblique boundaries, and the ability to dynamically link a 1D model were developed by Syme (1991). Further improvements including the insertion of 1D elements or quasi-2D models inside a 2D model and the modelling of constrictions on flow such as bridges and large culverts, and automatic switching into and out of upstream controlled weir flow have been developed subsequently (WBM, 2000).

TUFLOW models have been successfully checked against rigorous test cases (Syme 1991, Syme et al 1998, Syme 2001), and calibrated and applied to a large range of real-world tidal and flooding applications. TUFLOW is a leading fully 2D hydrodynamic modelling system and has the unique ability to be dynamically linked to quasi-2D models and have quasi-2D models dynamically nested inside or through the fully 2D domain.

Hydraulic structure flows through large culverts and bridges are modelled in 2D and include the effects of bridge decks and submerged culvert flow. Flow over roads, levees, bunds, etc is modelled using the broad-crested weir formula when the flow is upstream controlled. For smaller hydraulic structures such as pipes or for weir flow over a bridge, ESTRY 1D models can be inserted at any points inside the 2D model area.

4.5 Establishing River & Floodplain Topography

The ground topography was based on a digital elevation model (DEM) developed by WBM using photogrammetry data (points on a 30 m grid and breaklines) provided by the Herbert Resource Information Centre (HRIC) and survey of the bed and top of bank supplied by the Hinchinbrook Shire Council and the Department of Natural Resources and Mines. As was detailed in Section 3.1, the photogrammetry data was originally obtained as part of the Herbert Mapping Project, the Ingham Town Mapping Project, and the Halifax Town Mapping Project. The DEM was developed using the software package 12D.

The bed level survey was supplied by the Hinchinbrook Shire Council as river cross-sections. 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 at least five or 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 river bed in the DEM 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.

4.6 Selection of Calibration / Verification Events

One flood event was required for calibration and one for verification. The selection criteria for the calibration and verification events were:

- the amount of 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.

There is historical peak flood level information available for the 1967, 1972, 1977, 1986, 1990, 1991, 1994, 1997, 1998, 1999. The flood level information for 1990 and 1994 onwards is concentrated around Ingham and Halifax, with little data on the floodplain. A large number of flood levels were surveyed following the 1967, 1977, 1986 and 1991 floods in the towns as well as the floodplain. Very little data was recorded following the 1972 flood. Therefore, it was concluded that the two events should be selected from the 1967, 1977, 1986 and 1991 floods.

A review of the hydrological model calibration was undertaken in conjunction with the Bureau of Meteorology. The BoM has developed stream rating curves at the stream height gauging stations: 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. In calibrating the URBS model, the hydrograph generated by the URBS model based on the catchment rainfall is compared to the hydrograph derived using the recorded height time series at the gauging stations and the stream rating. In undertaking the calibration, the hydrograph generated from the stream gauge is adopted as being reliable, although it is recognised that the rating can contain inaccuracies. A feature of the URBS hydrologic model is that it allows the user the option of “matching” the hydrographs at river gauging stations for which a stream rating is available. This feature was utilised at Gleneagle station to determine if it improved the calibration at Abergowrie or Abergowrie Bridge and the Ingham Pump Station; these are the stations of most interest in the study.

The calibration of the URBS model to the 1967, 1977, 1986 and 1991 was reviewed. Calibration results are provided in Figure 4-1 to Figure 4-8 for the 1991 and 1967 floods; these are the floods that were ultimately selected for calibration of the hydraulic model. Figure 4-1 and Figure 4-2 show the calibration at Abergowrie in the 1991 flood without and with matching at Gleneagle. The comparison between the recorded and model results indicate that the URBS model is replicating catchment timing reasonably well, but is significantly underestimating the discharge, even with the hydrograph matched at Gleneagle. This indicates that there was probably significant rainfall in the catchment between Gleneagle and Abergowrie that was not recorded. These differences between the recorded and modelled are reflected in the Ingham Pump Station calibration (Figure 4-3 and Figure 4-4).

A reasonably good calibration was obtained at Abergowrie Bridge for the 1967 flood and the calibration was further improved with matching at Gleneagle (Figure 4-5 and Figure 4-6); the

Abergowrie Bridge was used in this calibration as there was no height-time history available at Abergowrie. At the Pump Station, there was about a 15% difference between the model and recorded data without matching at Gleneagle (Figure 4-7), and with matching, the difference was about 10% (Figure 4-8).

In the 1977 flood the peak flow at Abergowrie was reasonably well matched but there was differences in the shape of the hydrographs. The peak of the 1986 flood was poorly matched.

The 1991 and 1967 floods were selected for the calibration and verification. The 1991 flood was selected because it is closest of the four floods to the time that the photogrammetric data was obtained, and it is still remembered by the community. The 1967 flood was selected for the verification because it is the "big" flood that the long-term residents still talk about. It is also different to the other floods in that the flooding in the Lower Herbert was predominantly a result of upper catchment rainfall, i.e., above the Gleneagle gauge. Despite the lack of lower catchment rainfall, it produced the highest flood level at the Ingham Pump Station since records commenced in 1917.

From this review it was concluded that the URBS calculated hydrographs at Abergowrie in the 1991 flood were not suitable as inflow boundaries for the calibration of the hydraulic model and that another strategy would be required for determining the inflow boundaries of the hydraulic model. This is explained in more detail in Section 4.7. It is noted that in events subsequent to 1991 and following the installation of more rainfall stations, that better agreement is achieved between the URBS calculated hydrographs and the derived hydrographs.

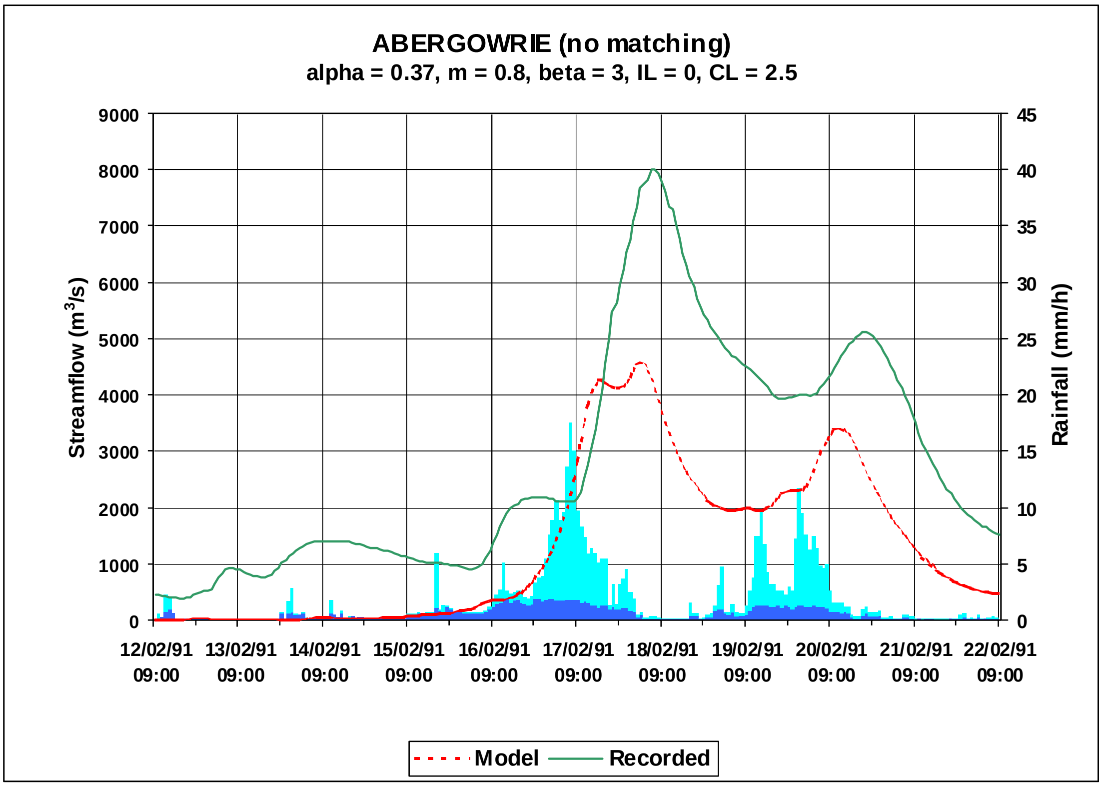


Figure 4-1 URBS 1991 Model Calibration at Abergowrie – no matching

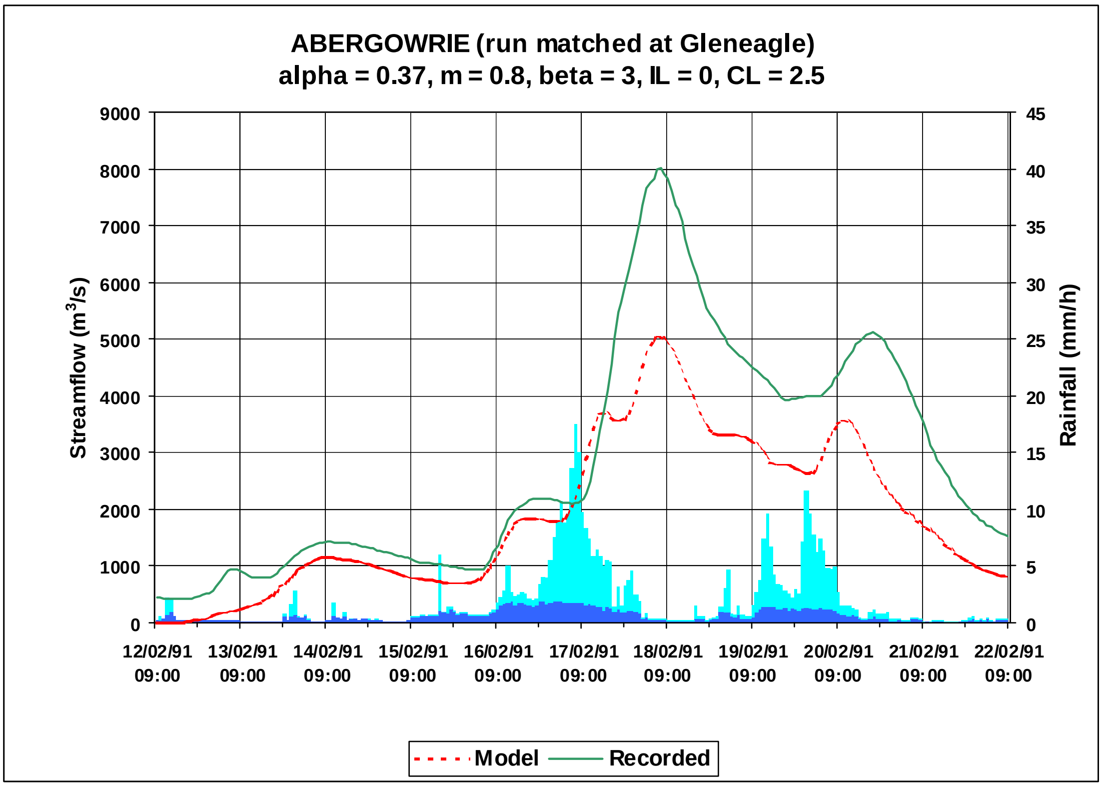


Figure 4-2 URBS 1991 Model Calibration at Abergowrie - matched at Gleneagle

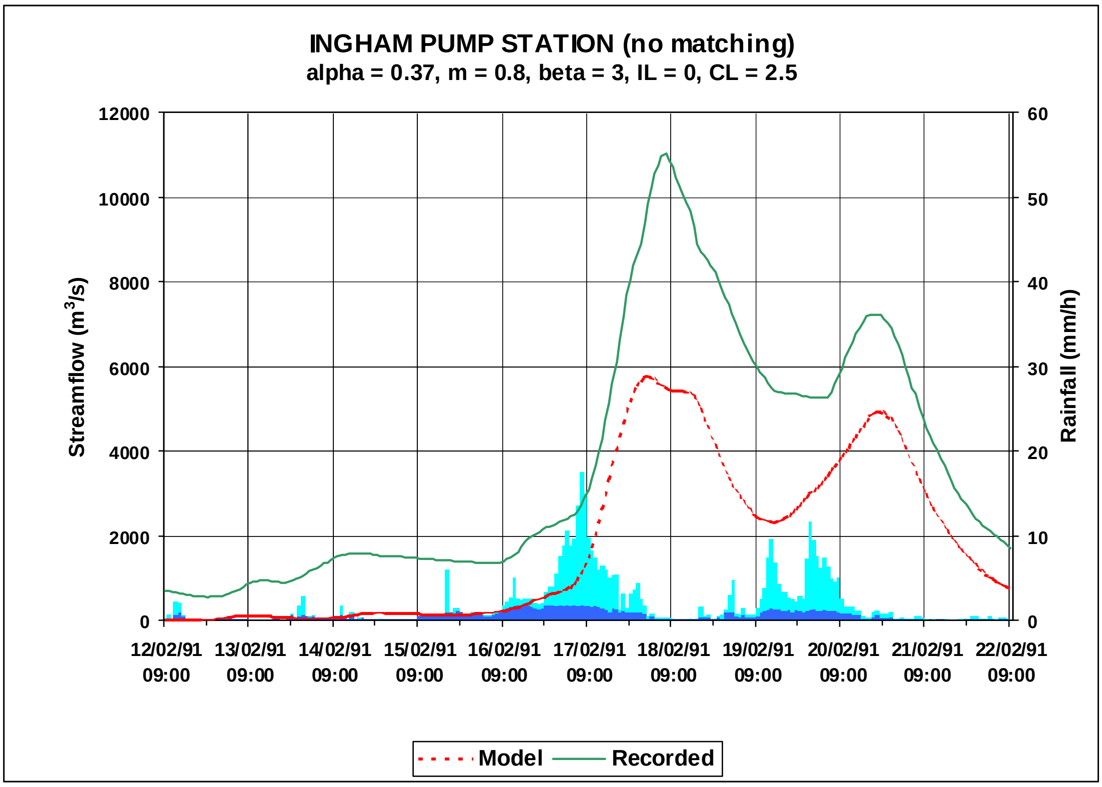


Figure 4-3 URBS 1991 Model Calibration at Ingham Pump Station – no matching

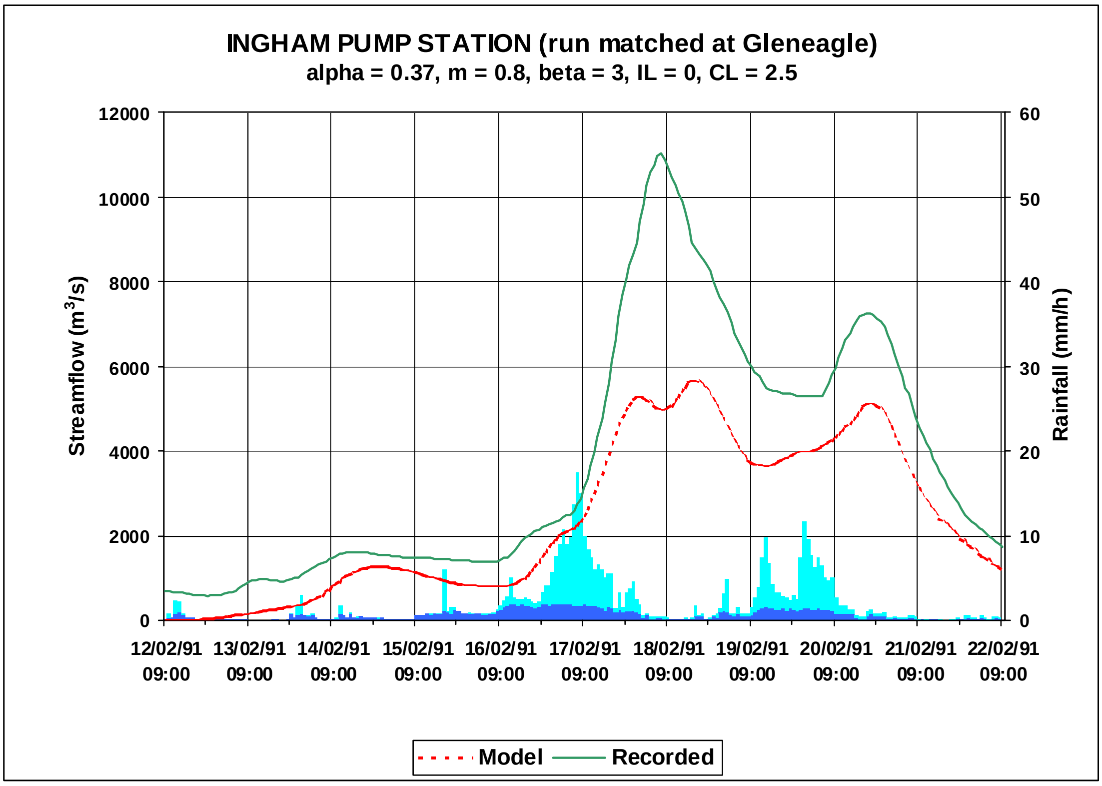


Figure 4-4 URBS 1991 Model Calibration at Ingham Pump Station – matched at Gleneagle

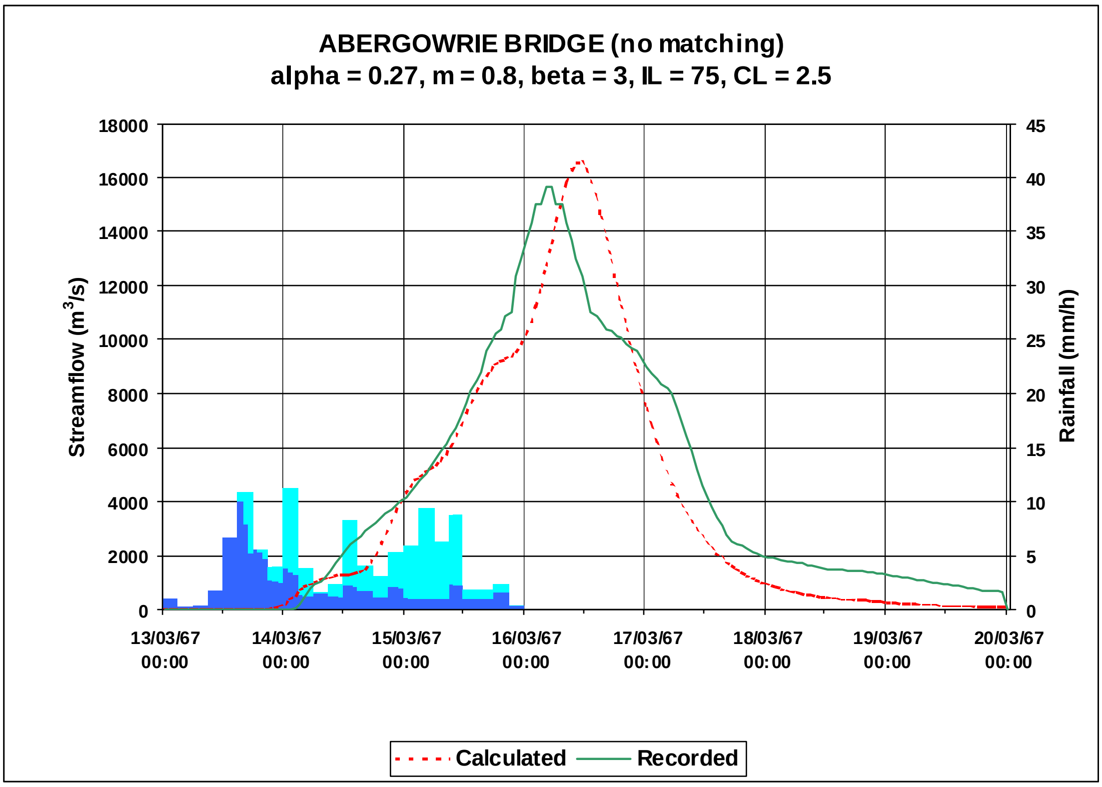


Figure 4-5 URBS 1967 Model Calibration at Abergowrie Bridge – no matching

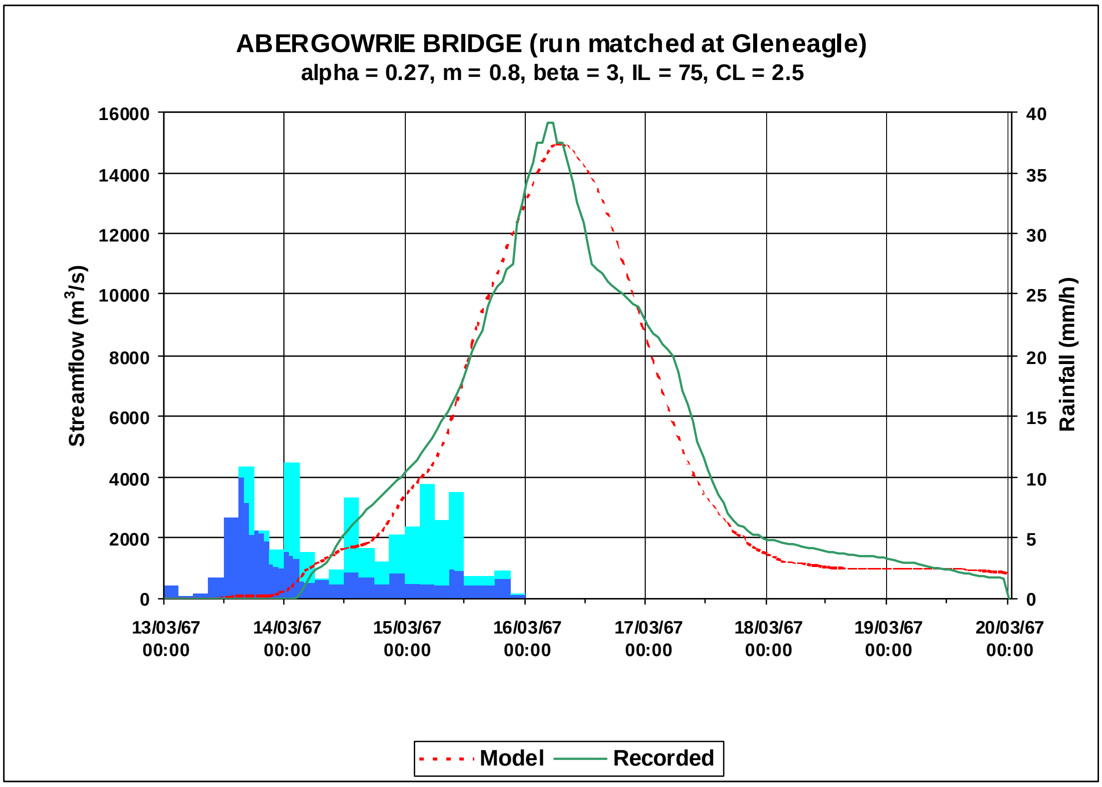


Figure 4-6 URBS 1967 Model Calibration at Abergowrie Bridge – Matched at Gleneagle

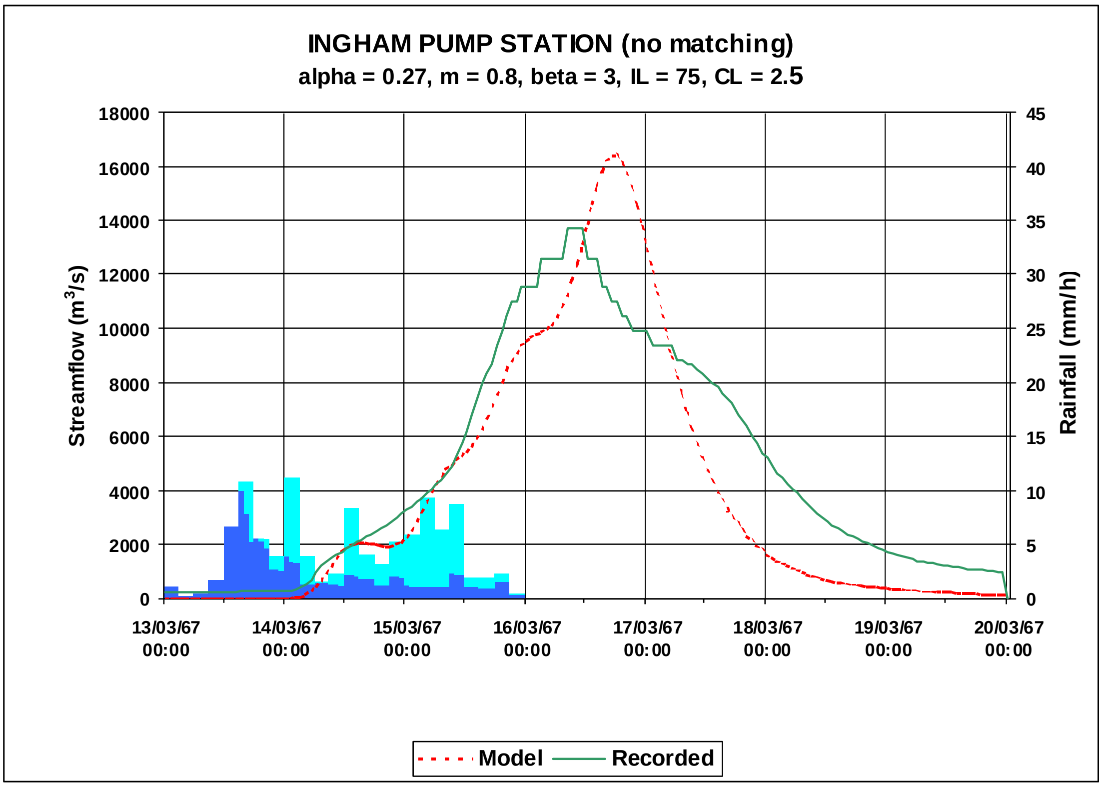


Figure 4-7 URBS 1967 Model Calibration at Ingham Pump Station – no matching

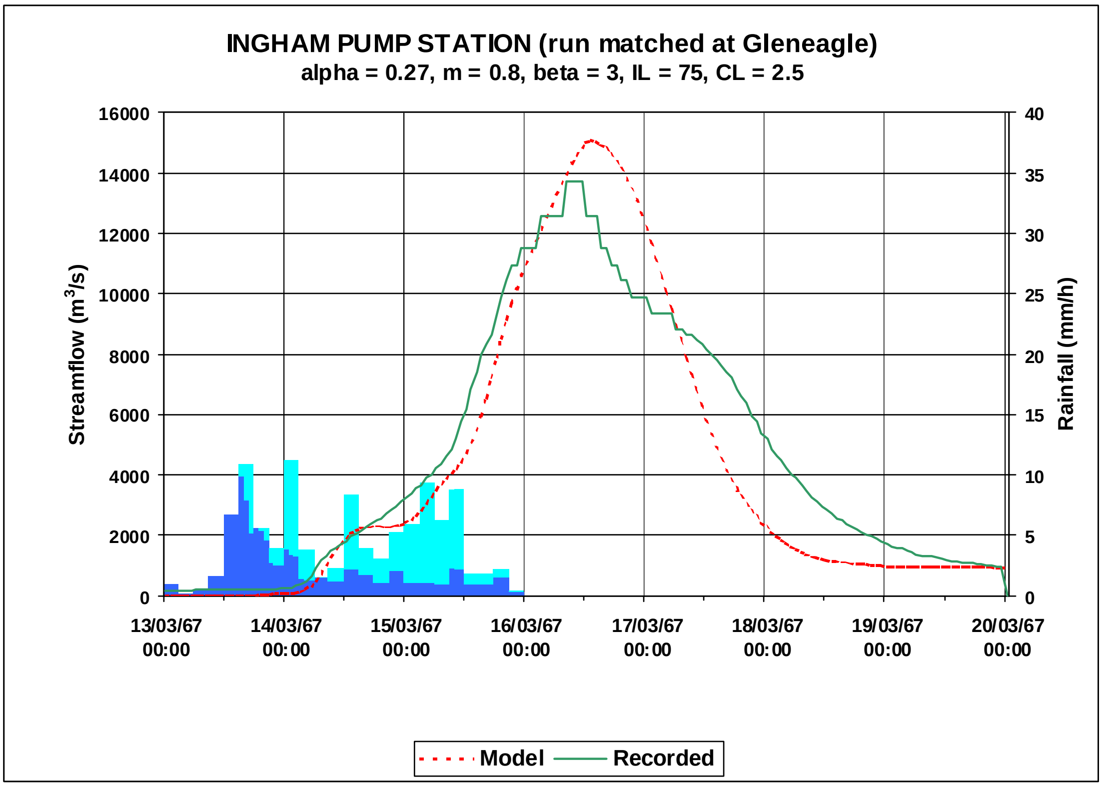


Figure 4-8 URBS 1967 Model Calibration at Ingham Pump Station – Matched at Gleneagle

4.7 Hydraulic Model Calibration

4.7.1 Calibration Procedure

Given that the URBS model representation of the flood events selected for calibration was not of sufficient accuracy for the purpose of this study, an alternative approach to the generation of the inflow boundary at Abergowrie was adopted. Details of this approach are given below for the calibration of the 1991 flood.

1. Initially the model was run with a height vs time (H-t) boundary at the boundary at Abergowrie (refer Drawing 4-2) based on the recordings there. Normally a flow vs time boundary (Q-t) as is adopted. For the tributary inflow boundaries downstream of Abergowrie, the calculated hydrographs from the URBS model were initially adopted.

2. The recorded ocean tidal cycle was applied at the downstream ocean boundaries.

3. The height-time data calculated by the TUFLOW model at Abergowrie Bridge (downstream of Abergowrie) was compared to the recorded data at the same location.

4. The Manning's n was adjusted in the river until good agreement was achieved at Abergowrie Bridge.

5. Once good agreement was obtained it can be assumed that the model was adequately predicting the inflow at Abergowrie. The H-t boundary at Abergowrie was then converted to a Q-t boundary based on the inflow determined by the model. This fixed the inflow so that any other changes to the model would not change the flow at the boundary, which is the case with a H-t boundary.

6. Factor the tributary inflows according to the ratio of the flow determined by the TUFLOW model at Abergowrie and that calculated by the URBS model.

7. Adjust the Manning's n through the remainder of the model to achieve calibration on the floodplain

4.7.2 1991 Calibration

The calibration of the model focussed on: the recorded height-time histories at Abergowrie Bridge, Ingham Pump Station and Gairloch Bridge; the surveyed peak flood levels along the banks of the river and across the floodplain; and general flooding patterns. The tolerance for calibration to recorded levels was adopted as ± 0.2m. Calibration parameters for both models were kept within conventional bounds, and parameter consistency across the model was maintained.

Figure 4-9 to Figure 4-11 provide a comparison between the recorded height-time histories at the three locations and the model results at these locations. At Abergowrie Bridge (Figure 4-9), good agreement is obtained between the recorded data and model results. There appears to be a small timing difference between the recorded and model data. This can be traced back to the recorded data used at the hydraulic model boundary at Abergowrie. Initially the timing differences were greater (the peak occurred at Abergowrie Bridge before Abergowrie), but the BoM advised that there may be an error in the recorded times at Abergowrie. It is believed that the recorded times at Abergowrie

Bridge are correct because it is a manually read gauge. Therefore, the timing of the boundary data was adjusted, but a small difference remained.

At Ingham Pump Station (Figure 4-10), the peak flood level calculated by the model is about 0.16 m less than that recorded by the gauge. There are some differences in the rising and falling limbs that may be related to the timing and size of the major tributary inflows such as Elphinstone Creek, Dalrymple Creek and the Stone River.

At the Gairloch Bridge (Figure 4-11), the model is over-predicting the peak flood level by about 0.15 m, and there are differences in the rising and falling limbs as is the case at the Ingham Pump Station.

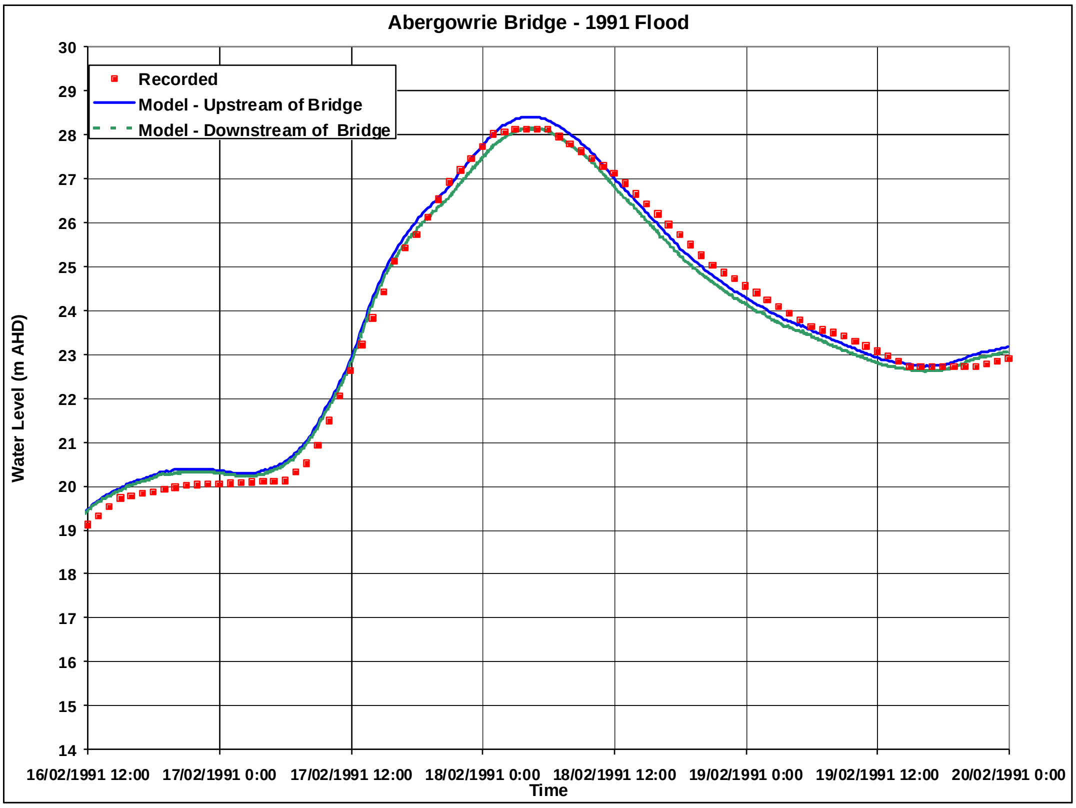


Figure 4-9 Comparison Between Recorded Heights and Model Results at Abergowrie Bridge– 1991 Flood

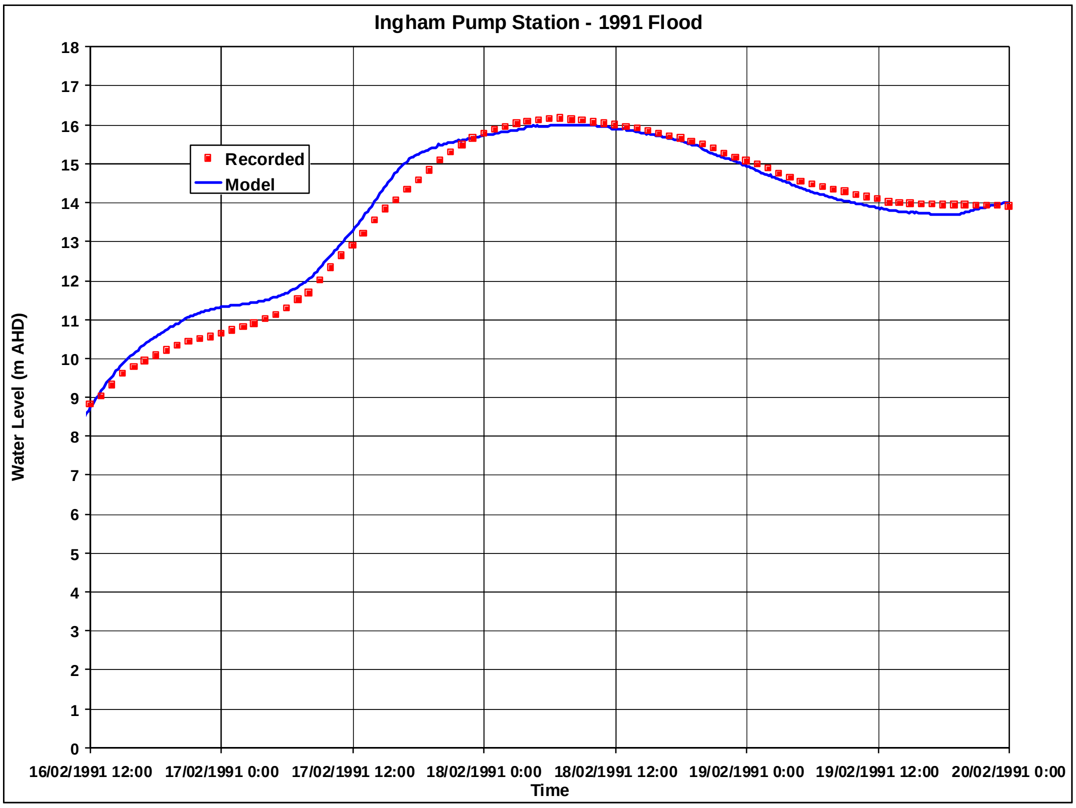


Figure 4-10 Comparison Between Recorded Heights and Model Results at Ingham Pump Station– 1991 Flood

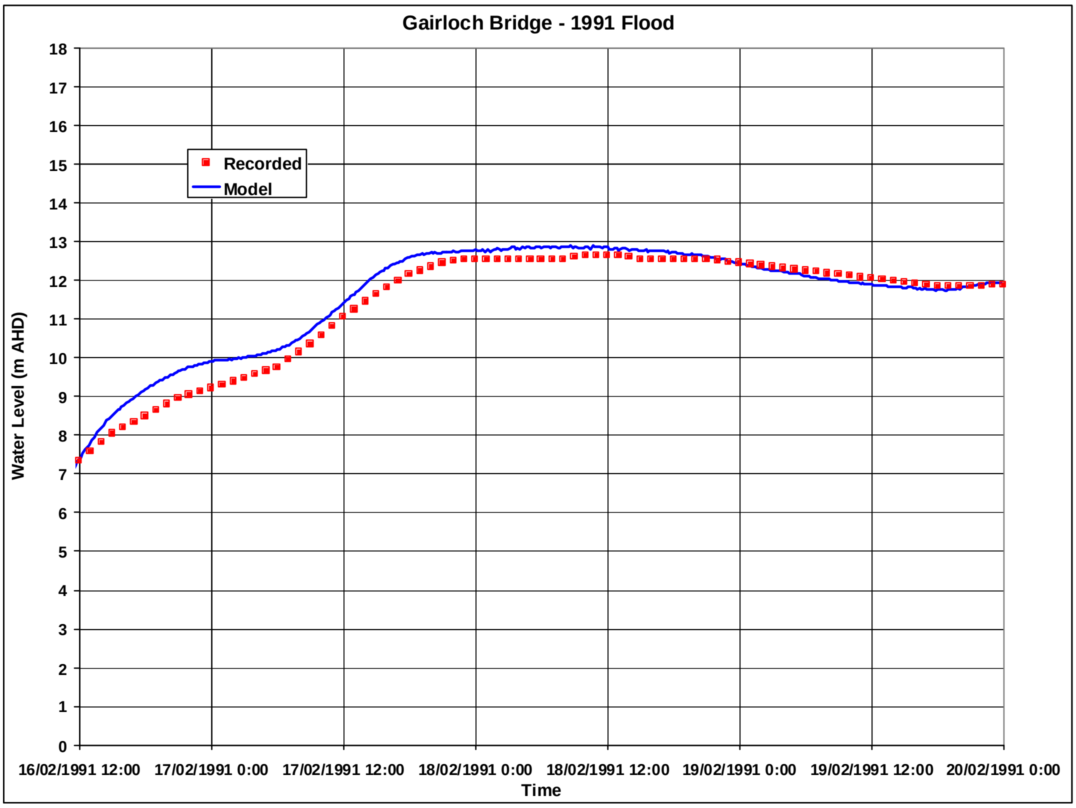


Figure 4-11 Comparison Between Recorded Heights and Model Results at Gairloch Bridge – 1991 Flood

The calibration of the model to the 399 surveyed peak flood levels in the floodplain is given in Drawing 4-3. Rather than display numbers, the difference between the modelled peak flood level and the surveyed level is colour coded according to the legend in the drawing. A positive number indicates that the modelled level is higher than the recorded level. The yellow colour indicates where the model is within the range ±0.2 m, and hence, is considered calibrated. It can be seen from Drawing 4-3 that the model is well calibrated over most parts of the floodplain. A statistical analysis of the calibration given in Table 4-1 shows that 72% of the 399 calibration points were within the ± 0.2 m tolerance, also indicating a good calibration.

Table 4-1 Statistical Analysis of 1991 Calibration

Range	Percentage of Calibration Points within Range (%)
< 1.0 m	0
-1.0 m to –0.6 m	1
-0.6 m to –0.4 m	3
-0.4 m to –0.2 m	13
-0.2 m to +0.2 m	72
0.2 m to 0.4 m	8
0.4 m to 0.6 m	2
0.6 m to 1.0 m	1
> 1.0 m	1

Discussions with local residents during the community open sessions and the resident survey provided an overview of the flooding patterns of the Lower Herbert River floodplain. The comments listed below are specific to the 1991 flood event.

- Most water that flowed out onto the floodplain was via backed up creeks and other outlets to the Herbert River. However, there were a number of places where the river broke its banks. Major break-outs occurred at the following locations:
  - over the north bank in the vicinity of Ingham;
  - through the Gairloch Washaway;
  - over the south bank just downstream of the Gairloch Bridge; and
  - through the Halifax Washaway.
- Floodwaters broke over the riverbank downstream of Abergowrie Bridge and flowed across Long Pocket before rejoining the main channel. This resulted in a number of houses being isolated.
- There was a break-out from the Herbert River on the south bank upstream of the Stone River mouth. Other than this, the Herbert River did not break its banks between the Stone River and Long Pocket.
- Floodwaters backed up Hawkins Creek from the Herbert River and flowed overland parallel to Hawkins Creek Rd before rejoining the main channel just upstream of the Hawkins Creek community.
- The Trebonne Creek system did not receive much water. A ridge runs parallel to Trebonne Creek from Trebonne to Ingham and this ridge prevented water that had broken over the bank of

the Herbert River from flowing into Trebonne Creek. The Stone River broke its banks and some water flowed down into Trebonne Creek from this direction.

- The Herbert River broke its banks opposite Trebonne (i.e. in the vicinity of the Hawkins Creek community) and water flowed over Hawkins Creek Road down into the Ripple Creek system.
- Floodwaters broke out of Palm Creek near the Victoria Mill and flowed across the Ingham-Forrest Beach Road.
- At Macknade, water initially backed up Macknade Creek from the direction of the Halifax Bridge. As the floodwaters in the main channel continued to rise, water broke the banks of the Anabranch and flowed over to Macknade from this direction.
- The river did not break its banks in the vicinity of the Macknade Mill.
- Large volumes of water flow out of the Herbert River through both the Gairloch Washaway and the Halifax Washaway.

Computer animations were created for the 1991 floods, based on the results of the computer flood modelling. These animations indicate that the computer model exhibits flooding patterns that match the observations listed above. The computer animations were also scrutinised in detail by the community at the open sessions in September 2001. At each of the open sessions, there was general agreement from the community in attendance that the models were reproducing the flooding behaviour as they remembered.

4.7.3 1967 Model Verification

For this verification run, the model topography was unchanged from that used in the 1991 calibration model because of lack of data, but the roughness (Manning's n) was adjusted to represent, as best as could be ascertained, floodplain development of the time. No recorded data was available at Abergowrie for the 1967 flood. Therefore, the upstream boundary of the model was set at the Abergowrie Bridge rather than at Abergowrie as was used for the 1991 calibration event. The recorded H-t data at the bridge was adopted for the boundary condition.

Figure 4-12 and Figure 4-13 show a comparison between the recorded height-time histories and the model predictions at the Ingham Pump Station and the Gairloch Bridge. Figure 4-12 shows that the model is reproducing the rising limb of the hydrograph well at the Pump Station. The peak flood level calculated by the model is about 0.2 m lower than the peak recorded level. At the Gairloch Bridge (Figure 4-13) there are some small differences in the rising and falling limbs of the hydrograph, but the difference in peak level is only about 0.05m.

A comparison between the 797 surveyed peak flood levels and the model flood levels is given in Drawing 4-4. In Ingham the model is generally well calibrated or slightly high except on the eastern side where the model is lower than the recorded. This could be a result of changes to the structures in town such as the QGR rail line that have occurred since the 1967 flood, but not adjusted in the model. In Halifax there is good agreement in the southern parts and the model is low in the northern parts, probably because of the construction of the levee in north Halifax since 1967. Across the remainder of the floodplain the results are mixed. It is likely that the differences are a result of the construction of structures on the river bank and floodplain since 1967, but not adjusted in the model. A statistical analysis of the calibration given in Table 4-2 shows that 51% of the calibration points were within the

± 0.2 m tolerance. Overall this is a good result for a verification event with agreement achieved with about 400 points.

The comments on flooding patterns listed below are specific to the 1967 flood event. The flooding patterns observed during the 1991 flood (refer Section 4.7.2) are also applicable to the 1967 flood.

- There was a break-out at Hawkins Creek and a large volume of water flowed over Hawkins Creek Road and down into the Ripple Creek and Seymour River systems.
- Floodwaters from the Herbert River backed up the Stone River. As a result, the Stone River broke its banks and a large volume of water flowed down into the Trebonne Creek system. This water then flowed along Trebonne Creek and around to the south of Ingham. There were high velocities and significant erosion in the vicinity of the Ingham racecourse.
- The floodwaters flowed down Palm Creek with a high velocity and there was a large break-out behind Warrens Hill.

Computer animations were created for the 1967 floods, based on the results of the computer flood modelling. These animations indicate that the computer model exhibits flooding patterns that match the observations listed above.

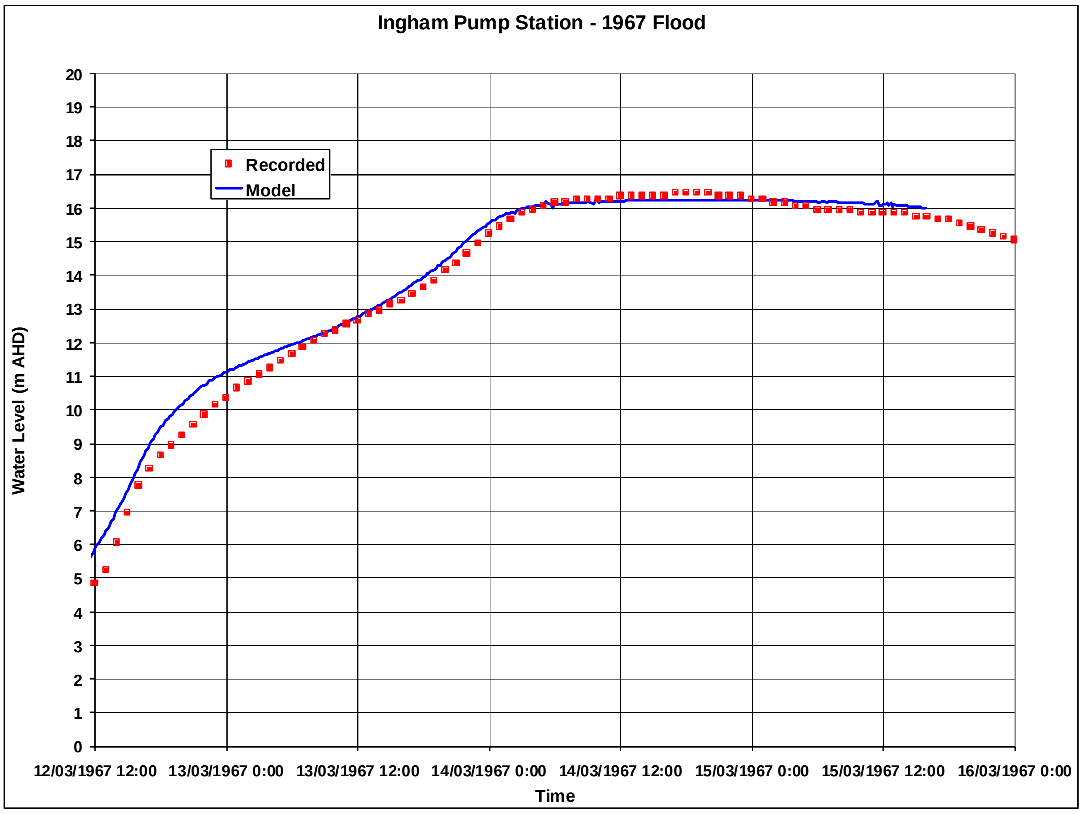


Figure 4-12 Comparison Between Recorded Heights and Model Results at Ingham Pump Station – 1967 Flood

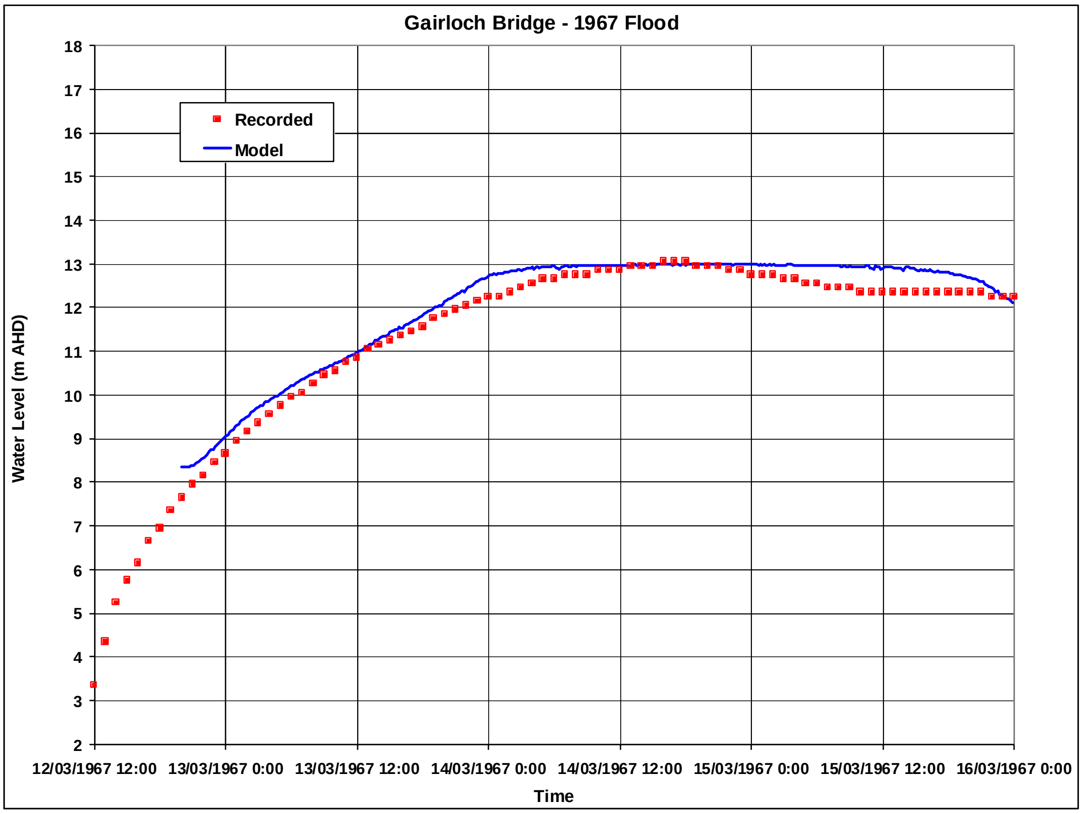


Figure 4-13 Comparison Between Recorded Heights and Model Results at Gairloch Bridge – 1967 Flood

Table 4-2 Statistical Analysis of 1967 Calibration

Range	Percentage of Calibration Points within Range (%)
< 1.0 m	4
-1.0 m to –0.6 m	4
-0.6 m to –0.4 m	4
-0.4 m to –0.2 m	13
-0.2 m to +0.2 m	51
0.2 m to 0.4 m	10
0.4 m to 0.6 m	3
0.6 m to 1.0 m	2
> 1.0 m	1

4.8 Summary

A TUFLOW hydraulic model has been developed for the Lower Herbert River and Floodplain. The model was calibrated to the 1991 flood and verified using the 1967 flood event. Overall, satisfactory agreement between recorded and model flood levels was obtained across the floodplain in both events.

5 DESIGN FLOODS

5.1 Background

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, every 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 5 year, 10 year, 20 year, 50 year and 100 year ARI floods and the Probable Maximum Flood (PMF) were analysed.

In Section 4 it was demonstrated that the flood model of the Herbert River reliably reproduces the flooding characteristics of the lower Herbert 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 Herbert River system.

5.2 Design Hydrology

The approaches available to determine the inflow boundaries for the hydraulic model for each of the design flood events were as follows:

1. The URBS hydrologic model - design rainfall events and temporal patterns are derived from AR&R (2001);

2. As for 1., but hydrographs factored to match peak flows derived from a flood frequency analysis;

3. Synthetic hydrographs developed from historical floods factored to match peak flows derived from a flood frequency analysis;

4. A combination of the above.

Option 4 was adopted for the 5 year through to 100 year ARI design floods and option 1 was adopted for the PMF. Design flows for the major inflow boundary at Abergowrie on the hydraulic model were determined by undertaking a flood frequency analysis of the Herbert River at Abergowrie. The flood frequency analysis provided a peak flow for each design event but not a hydrograph giving the variation in flow with time. A synthetic hydrograph was developed from historical hydrographs at this location on the Herbert River. Flows for all other inflow boundaries were obtained from the URBS hydrological model. The magnitude and timing of the other inflow boundaries were adjusted using an iterative process so that the peak flow in the hydraulic model at the Ingham Pump Station approximately matched the peak flow ascertained for the pump station.

The adopted approach is detailed further in Sections 5.2.1 to 5.2.3.

5.2.1 URBS Hydrological Model

The 100 year ARI event was run in the URBS model using temporal patterns (variation of rainfall over time) and rainfall intensities for the Herbert River catchment as described in Australian Rainfall and Runoff (AR&R, 2001); this is the standard approach for the development of design hydrographs in Australia. The 100 year ARI design hydrograph at Abergowrie developed using this approach is plotted in Figure 5-1 along with historical hydrographs; the 1967 hydrograph is from Abergowrie Bridge not Abergowrie because no data was available for Abergowrie in the 1967 flood. The design hydrograph has three distinct peaks and very steep rising limbs, a shape that is not consistent with the historical hydrographs. This raises concerns over the suitability of the standard approach to hydrological modelling and/or the URBS model for the development of design hydrographs for this catchment. Consequently, the URBS model was further reviewed in the following key areas that create uncertainties in hydrological models:

1. catchment response;

2. representation of rainfall, temporally and spatially;

3. representation of rainfall losses.

The review indicated that the URBS model was adequately representing catchment response and rainfall losses, which indicates that the spatial and temporal variation of the design rainfall adopted from AR&R (2001) may not be suitable for this catchment. The latter is considered further.

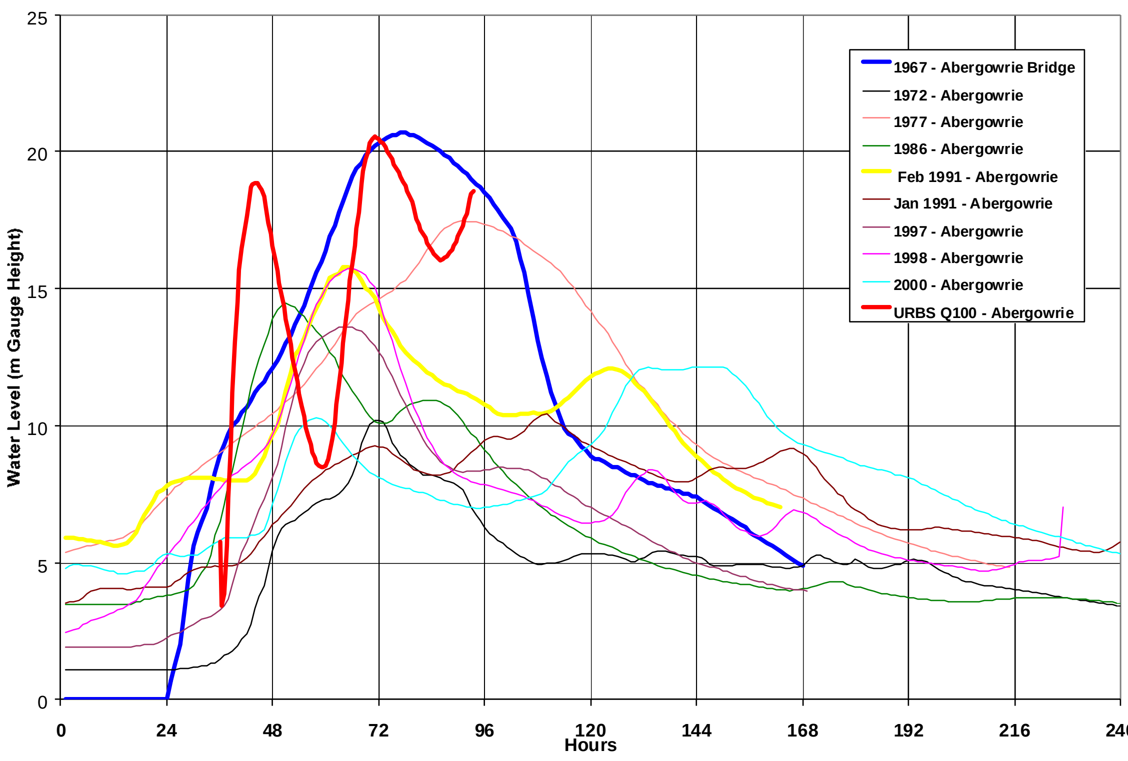


Figure 5-1 Hydrographs at Abergowrie and Abergowrie Bridge

The representation of spatial and temporal variation of rainfall over the catchment can affect the calibration of a hydrologic model. The addition of rainfall stations in the catchment in recent times has improved the representation of rainfall in the model for recent flood events, although more rainfall stations would further improve the spatial representation. When analysing design floods, temporal patterns and rainfall intensities are normally adopted from AR&R (2001).

WBM has found on other large catchments with long storm durations that the standard Queensland temporal pattern for long duration storms is not always appropriate. This would appear to be the case on the Herbert River catchment. For example, about 25% of the total rainfall falls in the first four hours of a 72 hour storm and then another 29% over an eight hour period about mid-way through the storm. The first two peaks in 100 year design hydrograph at Abergowrie presented in Figure 5-1 reflect this pattern.

Design temporal patterns from the northern NSW coastal zone can sometimes better represents rainfall patterns on some Queensland catchments. This temporal pattern was trialled and improved the shape of the hydrograph, but was still not considered suitable for the generation of a design hydrograph at Abergowrie.

From this review it was concluded that the URBS design hydrographs at Abergowrie would not be appropriate for inflows to the hydraulic model and that a revised approach was required. To provide a more realistic design hydrograph at Abergowrie, an historical hydrograph was adopted and factored to match the peak design flows determined from the flood frequency analysis.

AR&R (2001) provides a methodology for selecting an appropriate historical hydrograph for such a purpose. The method considers both the historical peak discharges and hydrograph volumes. This method, when applied to the historical hydrographs at Abergowrie, indicated that the 1991 hydrograph was appropriate.

The hydrograph at Abergowrie was used as the inflow boundary to the hydraulic model at Abergowrie and represented the flow off the catchment above Abergowrie. The catchment inflows below Abergowrie were taken from the URBS model. The magnitude and timing of the inflow boundaries adopted from URBS were adjusted using an iterative process so that the peak flow in the hydraulic model at the Ingham Pump Station approximately matched the peak flow determined from the flood frequency analysis. The NSW temporal pattern was used in URBS model to give a more realistic hydrograph shape.

The flood frequency analysis is described in the following section.

5.2.2 Flood Frequency Analysis

An annual series flood frequency analysis was carried out on the peak discharges at Abergowrie and the Ingham Pump Station. Stream rating curves (a plot which gives the discharge at a particular height) were developed for each station to determine the discharge for a recorded height. For this analysis, stream ratings were developed using a combination of the stream gauging carried out by DNRM and output from the TUFLOW hydraulic model. This resulted in a rating curve that differed from the DNRM and BoM ratings at higher flood levels, but is considered more reliable.

At Abergowrie, annual peak flood levels were available from 1970 to 2001 and at the Ingham Pump Station from 1917 to 2001. The records at Abergowrie were extended back to 1967 by transferring discharges estimated at the Abergowrie bridge. To determine if the additional records at Ingham Pump Station (1917 to 1966) influenced the analysis, the data was analysed for two periods, 1917 to 2001 and 1967 to 2001. The Log Pearson III distribution was fitted to the data. It was found that the peak discharges for the 5 year ARI to the 100 year ARI floods were about 5% lower in the analysis using the 1967 to 2001 data. Therefore, the LPIII peak discharges at Abergowrie were increased by 5%. The adopted design peak discharges are summarised in Table 5-1.

Table 5-1 Design Peak Discharges

ARI (years)	Peak Discharge (m^3/s)	
	Abergowrie	Ingham P.S.
2	2,200	2,450
5	5,500	5,500
10	7,900	7,700
20	10,300	9,800
50	13,300	12,600
100	15,400	14,500

The design peak discharge for the 10 year ARI and larger events is greater at Abergowrie than Ingham. This is not uncommon in river systems where the flood is confined to a narrow valley and then breaks into a large floodplain. The discharges at a section across the floodplain may be lower because of the storage effect of the floodplain.

A list of recent major floods and their return period as determined using the flood frequency analysis is given in Table 5-2. The 1967 flood is the largest recorded at the gauge. However, there is anecdotal evidence that the 1927 flood was larger in the floodplain than the 1967 even though it was 0.34 m lower at the gauge.

Table 5-2 Return Period of Historical Floods

Year	Abergowrie		Ingham Pump Station	
	Discharge (m^3/s)	ARI (years)	Discharge (m^3/s)	ARI (years)
1955	-	-	9100	15
1967	13500	50	11670	40
1977	11950	35	9360	17
1986	7350	9	6920	8
1991	9010	15	8100	12
1997	6460	7	5070	4
1998	8940	15	7640	10
1999	5520	5	5555	5
2000	5080	4	6170	6

5.2.3 Probable Maximum Flood

One method for determining the PMF flow is to apply the probable maximum precipitation (PMP) to a catchment and route the excess rainfall through the catchment. AR&R(2001) recommends the Generalised Tropical Storm Method (GTSM) for catchments of the size and location of the Herbert River catchment to determine the PMP. Temporal patterns for the GTSM are obtained from

AR&R(1987). The PMP was calculated for the upper and lower catchments for a range of storm durations. The results are presented in Table 5-3.

Table 5-3 Probable Maximum Precipitation

Storm Duration (hrs)	Total Rainfall Depth (mm)	
	Upper Catchment	Lower Catchment
48	920	1280
72	1240	1660
96	1500	2000

The URBS hydrological model was used to route the excess rainfall and provide inflow boundary conditions for the TUFLOW hydraulic model. The 48, 72, and 96 hour storms were run in the model. An initial loss of 0 mm and a continuing loss of 1.5 mm/hr were adopted. The 72 hour storm gave the peak discharge. Although the URBS model was not adopted for the smaller design floods, the PMF hydrograph shape produced by the model was satisfactory and the peak discharge was within the bounds of rule of thumb calculations for the PMF. For example, the PMF peak discharge is typical 1.5 to 3 times that of the 100 year ARI discharge. The peak PMF discharge at Abergowrie of approximately 38,000 m^3/s is about 2.5 times the 100 year ARI discharge at Abergowrie. Another rule of thumb is that for a catchment of about 7500 km^2 (the catchment at Abergowrie), the ratio of the peak PMF discharge to the catchment area is typically within the range 1.5 to 7. This gives a PMF discharge range of 11,000 m^3/s to 52,000 m^3/s.

5.2.4 Summary of Adopted Design Hydrology

In summary the following approach was adopted to provide design inflows to the hydraulic model:

1. at the upstream boundary of the hydraulic model at Abergowrie, a flood frequency analysis was used to determine the peak design discharge and the hydrograph from the 1991 flood was factored so that the peak discharge matched the peak discharge determined from the frequency analysis;

2. the tributary inflows below Abergowrie were obtained from the URBS model and an iterative adjustment to the timing of the inflows was undertaken such that the flow past the Ingham Pump Station in the hydraulic model approximated the peak design flow determined from the flood frequency analysis.

5.3 Design Hydraulic Model

The calibrated 1991 TUFLOW hydraulic model was adopted as the design model, but was updated as follows to represent the current floodplain topography:

1. Tyto wetlands added;

2. Ingham Dump added;

3. Castorina's Levee added;

4. Mombelli's Levee added;

5. levee south of Halifax added;

6. levees in Halifax added;

7. washaway chutes added.

In setting a tidal ocean boundary, consideration of storm surge and wave setup is required. The Beach Protection Authority Queensland publication “Storm Tide Statistics – Lucinda 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 commencement of landfall. The size of the Herbert catchment means that the peak flood heights typically occur at least 2 days after the commencement of landfall. Therefore, storm surge is unlikely to influence the peak flood levels. Nonetheless, a sensitivity test was undertaken on the 1991 flood to determine the extent of influence of the ocean level on peak flood levels. This is described in Section 5.4.

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.

5, 10, 20, 50 and 100 year ARI design floods and the PMF were run on the hydraulic model. Flood level contours for the entire study area for each of the design floods are presented in Drawing 5-1 to Drawing 5-6 respectively.

In the lower valley, the Herbert River is a perched river with significant overflow into its floodplain, most of which does not return to the main channel. To demonstrate the distribution of flow to the floodplain, Table 5-4 and Table 5-5 present the flow in the main channel at a number of locations in the river as a flow rate (m^3/s) and as a percentage of the flow in the channel at Abergowrie Bridge respectively. For example, in a 100 year ARI flood event, there is only about 1/3 of the flow remaining in the main channel downstream of the Gairloch Washaway..

Table 5-4 Flow in Main Channel of Herbert River

Location	Flow in Main Channel of Herbert River (m^3/s)				
	5 year ARI	10 year ARI	20 year ARI	50 year ARI	100 year ARI
Abergowrie Bridge	5300	7600	10000	12900	15100
D/S Dalrymple Ck	5300	7600	10300	12700	14800
D/S Long Pocket	5300	7600	10000	12000	13200
D/S Stone River	5700	7800	10100	11300	12200
Ingham Pump Station	5600	7600	7700	7800	8000
D/S John Row Bridge	5400	6100	6300	6400	6500
U/S Gairloch Washaway	5300	6000	6200	6200	6300
D/S Gairloch Washaway	4600	4900	4900	4900	5000
Approx. 4.7 km d/s of Halifax Bridge	1700	1700	1700	1800	1800

Note: D/S indicates downstream and U/S indicates upstream

Table 5-5 Flow in Main Channel of Herbert River as a Percentage

Location	Flow in Main Channel of Herbert River (m^3/s)				
	5 year ARI	10 year ARI	20 year ARI	50 year ARI	100 year ARI
Abergowrie Bridge	100%	100%	100%	100%	100%
D/S Dalrymple Ck	100%	100%	103%	98%	98%
D/S Long Pocket	100%	100%	100%	93%	87%
D/S Stone River	108%	103%	101%	88%	81%
Ingham Pump Station	106%	100%	77%	60%	53%
D/S John Row Bridge	102%	80%	63%	50%	43%
U/S Gairloch Washaway	100%	79%	62%	48%	42%
D/S Gairloch Washaway	87%	64%	49%	38%	33%
Approx. 4.7 km d/s of Halifax Bridge	32%	22%	17%	14%	12%

Note: D/S indicates downstream and U/S indicates upstream

5.4 Sensitivity Testing of Hydraulic Model

Sensitivity of peak flood levels to the following parameters was undertaken:

- the ocean boundary;
- the Manning's "n" in the main river channel;
- the Manning's "n" of the sugar cane.

The ocean boundary for the 1991 flood was uniformly increased by 1.0 m. A comparison between the peak flood heights in the 1991 flood with increased ocean boundary levels and the peak flood heights using the recorded ocean boundary levels is given in Drawing 5-7. Over most of the study area there was no effect, but in the lower areas of the model close to the coast, there were increases of up to 0.8 m. This was a particularly severe test with the peak ocean level above the 2000 year ARI level at Lucinda. Therefore in the design floods the impact in the lower regions would be less than indicated in this sensitivity test. Further, the impacts occur because the increase of 1.0 m was combined with the flood peak, a combination that is not likely to occur in reality as discussed in Section 5.3. From this investigation it was clear that a storm surge will not significantly impact on the peak flood levels over most of the study area and that it is unlikely that a storm surge will be evident by the time the peak of the flood occurs in the lower areas.

The adopted Manning's roughness coefficient n for the main river channel was 0.03 in the lower reaches and 0.035 in the upper reaches. For the sensitivity test, n was increased to 0.05 along the entire river. The impact on peak 100 year ARI flood levels is given in Drawing 5-8. Although this is an extreme sensitivity test, it does indicate that flood levels in the floodplain are sensitive to the roughness in the main channel. For example, it indicates that increased vegetation in the river could result in increased flood levels over most parts of the floodplain, or conversely, clearing of vegetation within the river channel could result in increased flood levels in the lower parts of the floodplain around Halifax.

The adopted n for sugar cane was 0.15, which represents a typical stand of sugar in the February March period when larger floods typically occur. For the sensitivity test, n was decreased to 0.10 in all areas of sugar cane. The impact on peak 100 year ARI flood levels is given in Drawing 5-9. This analysis indicates that if a flood occurred earlier in the growth cycle of sugar cane, flood levels would be lower over most parts of the floodplain. One interesting exception is the western parts of Ingham where there would be increases in flood level.

5.5 Accuracy of Hydraulic Model

The accuracy of the hydraulic model's ability to predict absolute flood levels, the relativity of absolute flood levels (eg. the difference between 50 year and 100 year ARI levels), and the impact of changes to topography (eg. levees) on flood levels is considered. Factors that affect the accuracy of the model include, in no particular order:

1. inflow (upstream) boundaries;
2. assumptions in the computational scheme of the software;
3. ocean water level (downstream) boundaries;
4. accuracy of topographical data;
5. resolution of 2D model grid;
6. level of detail of control structures in model;
7. Manning's roughness coefficient n.

All computer software for modelling flooding incorporate assumptions as they approximate reality. The 2D schemes, as used for this study, have significantly less assumptions than 1D or quasi-2D schemes, and utilise a database of information typically 1,000 times higher in resolution. They are therefore the most accurate modelling approach available today. The assumptions inherent in the TUFLOW software is considered to be insignificant when compared with the other key factors discussed that have a significant uncertainty.

The inflow boundaries are determined from the hydrological analysis. As was detailed in Section 5.2, the inflows were primarily developed using an analysis of historical floods. The accuracy of this analysis is dependent on factors such as the length of available record and the reliability of the estimate of peak discharges. There was 84 years of data available at the Ingham Pump Station, which means that the results should be used with caution on larger floods such as the 50 year and 100 year ARI floods. As more data is collected, it is possible that the analysis would change with the result that, for example, what is currently considered a 100 year ARI flow may become smaller or larger. In smaller events such as the 5 and 10 year ARI floods, a significant change over time would be less likely because more is known about these more frequent events. Changes to the hydrological analysis would impact on the absolute flood levels and the relativity of the absolute flood levels between smaller and larger events.

The sensitivity test presented in Section 5.4 demonstrated that the peak flood levels in the area of interest in this study are not sensitive to the ocean boundary.

The accuracy of absolute flood levels will be sensitive to the accuracy of the topographical data, especially in areas where there is shallow flow. The accuracy of the photogrammetry as discussed in Section 3 was ± 0.125 m for that covering Ingham and Halifax and ± 0.5 m over the remainder of the floodplain, although it was noted that checks by JSC surveyors had found that in open areas the accuracy was probably ± 0.1 m across the floodplain. It was also noted that additional ground survey of key control structures was obtained. The 1991 flood calibration demonstrated that the model was reproducing flood levels to within ± 0.2 m over most of the floodplain, which is consistent with the accuracy of the survey. Therefore, it can be concluded that the accuracy of the absolute flood levels is ± 0.2 m for this size event. Generally, topographical data will not significantly influence the relativity of flood levels, when investigating “what-if” scenarios, because the topography is the same in both before and after model runs. A similar comment applies to the effect of the topographical data on an impact analysis where there is only a localised change in topography related to the structure being analysed.

The grid size (40 m) is considered sufficient for the purposes of this broad-scale study. This is supported by the calibration.

As noted previously, ground survey was obtained for key control structures to ensure accurate representation within the model.

In Section 5.4 it was demonstrated that the absolute flood levels are strongly influenced by Manning’s n. However, the model is well calibrated indicating that the selected values are appropriate. Generally, Manning’s n will not significantly influence the relativity of flood levels or the impact analysis of a control structure. It could have a significant influence on an impact analysis of a changed vegetation type.

In conclusion, it is estimated that the tolerance on absolute flood levels is ± 0.2 m for the more frequent events (eg 5 year ARI). For rare events (eg. 100 year ARI) there are greater uncertainties in the rainfall-runoff predications which would decrease the accuracy of the flood level predictions. The estimated tolerance on relative flood levels for an impact analysis would vary from ± 0.05 m to ± 0.10 m, depending on the local topographic uncertainties and complexity of flow patterns.

5.6 Summary

The methodology for determining the design hydrographs was revised once it became evident that the URBS model was not producing hydrograph shapes that were consistent with historical floods at Abergowrie. The revised methodology factored the 1991 hydrograph to match the peak design discharges determined from the flood frequency analysis. The tributary inflows below Abergowrie were obtained from the URBS model and an iterative adjustment to the timing of the inflows was undertaken such that the flow past the Ingham Pump Station in the hydraulic model approximated the peak design flow determined from the flood frequency analysis.

The hydraulic model was updated so that the topography in the model represents the current floodplain. It was found that the peak flood levels in the study area are unlikely to be affected by storm surge. Predictions for the 5, 10, 20, 50 and 100 year ARI and the PMF floods were produced by the hydraulic model and the results for the entire study area mapped.

6 EXISTING FLOOD DAMAGES ASSESSMENT

6.1 Background

To improve floodplain management on the Lower Herbert River and more importantly, 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 which are used to quantify the benefits of certain mitigation measures (eg. levees).

The Herbert River Valley is a primary industry based economy, serviced by the towns of Ingham, Halifax and Trebonne. The region comprises predominantly floodplain lands used for sugar cane 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, representing 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;
- floor and property level data;
- available GIS layers defining landuse of the area supplemented by aerial photography;
- previous damages assessments completed for the HRIT by Kinhill Cameron McNamara (1991) and Cameron McNamara (1980);
- details of infrastructure (HSC, DMR, CSR, Telstra, ERGON, etc.). .

6.2 Previous Investigations

Cameron McNamara undertook a damages assessment as part of the Herbert River Flood Management Study (Cameron McNamara, 1980) and Kinhill Cameron McNamara (1991) documented flood damages from the 1991 flood.

The 1980 study determined an Average Annual Damages (AAD) for the delta and documented flood losses in the Hinchinbrook Shire for the 1977 flood. This information is reproduced in Table 6-1 and Table 6-2 respectively. Kinhill Cameron McNamara (1991) did not assess the 1991 flood damages comprehensively, but obtained some indication of damages. This data is reproduced in Table 6-3.

Table 6-1 Average Annual Damages in Delta (from 1980 Flood Management Study)

Sector	AAD ($ - 1980)
Sugar Industry	340,000
Other rural (incl. public utlities)	150,000
Ingham Residential	52,000
Ingham Commercial	60,000
Ingham Public Utilities	25,000
Total	**627,000**

Table 6-2 1977 Flood Losses in Hinchinbrook Shire (from 1980 Flood Management Study)

Sector	AAD ($ - 1980)
Sugar Industry	6,121,000
Other rural	353,000
Urban	789,000
Public Utilities	1,614,000
Total	**8,877,000**

Table 6-3 Approximate 1991 Flood Losses in Hinchinbrook Shire (from 1991 Flood Damages Assessment)

Sector	AAD ($ - 1991)
Hinchinbrook Shire Assets	1,900,000
Herbert River Improvement Assets	800,000
Department of Main Roads	300,000
CSR Tramway Damage	700,000
Residential and Commercial	No Assessment
Sugar Cane	Estimates vary ~ 30,000,000

6.3 Tangible Damages

The methodology for the calculation of rural and urban damages is explained in the following sections.

6.3.1 Rural Damages

As indicated in previous studies, rural flood damages contribute significantly to the total flood damages in the Lower Herbert region. Rural flood damages were calculated using the following steps.

- Identify the areas inundated for the range of design flood events (5, 10, 20, 50 and 100 year ARI and PMF). The flood areas were calculated using the flood extent determined using the TUFLOW hydraulic model.
- Define existing rural landuses:
  - ⇒ cane farming
  - ⇒ beef grazing
- Review latest research on flood damage to sugar cane and beef grazing.
- Apply damage relationships to the areas inundated and determine the AAD.

6.3.1.1 Damages Extent

The extent of the damages assessment was based on the coverage of the fully two-dimensional model (refer Drawing 4-2). 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 structure mitigation option. This covers an area of approximately 740km^2.

Not all of the 740km^2 area 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 6-4. These data were interpreted within the GIS to determine the areas of specific landuses inundated during the various floods. Other data extracted for the damages analysis included flood depth for calculating the sugar cane damages as described in Section 6.3.1.3.

Table 6-4 Inundated Areas

ARI Event (years)	Total Area Inundated (ha)
PMF	67,700
100	58,490
50	55,970
20	52,100
10	43,000
5	32,690

6.3.1.2 Rural Landuse

Rural flood damage varies according to the different landuse across the floodplain. The Herbert Valley is predominantly used for sugar cane farming, but there is a small beef grazing industry. A detailed description of landuse was obtained from the CSIRO 1996 landuse GIS layer. Limited cross-checking of this data with 1997 aerial photography found the information to be reliable.

Analysis of this information in combination with the inundated areas described in Section 6.3.1.1 was carried out to determine the inundated areas for each landuse (see Table 6-5).

Table 6-5 Landuse Areas Inundated

ARI Event (years)	Total Area Inundated (ha)	Breakdown of Landuse Inundated	
		Sugar Cane (ha)	Beef (ha)
PMF	49630	42190	7440
100	43390	37030	6360
50	41430	35310	6120
20	38240	32520	5720
10	32440	27840	4600
5	21720	18190	3530

6.3.1.3 Sugar Cane Growing

To determine the most appropriate methodology for calculating damages to sugar cane, discussions were held with John Reghenzani and Graham Kingston of the BSES and Peter Sheedy from 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). There is considerable scatter in the data, with a correlation coefficient of about 0.8. 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 Herbert River because Herbert 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. An average height of 1.1 m was assumed for the analysis.

3. An average inundation time above the growing point was assumed for all of the floodplain for each design flood:

    a. 4 days for the 5 year ARI flood;

    b. 4.25 days for the 5 year ARI flood;

    c. 4.5 days for the 20 year ARI flood;

    d. 5 days for the 50 year ARI flood;

    e. 6 days for the 100 year ARI flood;

f. 8 days for the PMF.

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

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

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

A number of assumptions and averages across the floodplain were required for the analysis. The average height, the % fallow, yield and average price were supplied by the Canegrowers association as being typical and significant variation from these values would not be expected. Apart from the average growing height, any variation to these parameters would lead to an equivalent percentage change to the damages calculation. The inundation time may vary significantly from flood to flood due to natural variations in the length of the flood, even if the magnitude is similar. A sensitivity test indicated that if the inundation period was increased by 50%, there would be about a 45% increase in damages.

6.3.1.4 Beef Grazing

WBM (2001) and PBP (1995) provide 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 6-6 and as a probability damages curve in Figure 6-1. It was assumed that zero damages occur in a 2 year ARI (50% AEP) event. An average annual rural damages of $2.2 million was then estimated by calculating the area under this curve. These results are presented in Table 6-7.

Table 6-6 Rural Probability-Damage

ARI Event (years)	Damages per Landuse ($2002)		
	Sugar Cane ($)	Beef ($)	Total ($)
PMF	$ 17,640,000	$ 1,790,000	$ 19,430,000
100	$ 9,580,000	$ 1,530,000	$ 11,110,000
50	$ 7,750,000	$ 1,470,000	$ 9,220,000
20	$ 6,830,000	$ 1,370,000	$ 8,200,000
10	$ 5,780,000	$ 1,100,000	$ 6,880,000
5	$ 3,770,000	$ 850,000	$ 4,620,000

Table 6-7 Average Annual Damage - Rural

ARI Event (years)	AEP	Existing Case ($2002)	
		Total Damages	Incremental Area Under Total Damages Curve
PMF	0%	$ 19,430,000	
100	1%	$ 11,110,000	$ 153,000
50	2%	$ 9,220,000	$ 102,000
20	5%	$ 8,200,000	$ 261,000
10	10%	$ 6,880,000	$ 377,000
5	20%	$ 4,620,000	$ 575,000
2	50%	$ 0	$ 693,000
Average Annual Damage			**$2.1M**

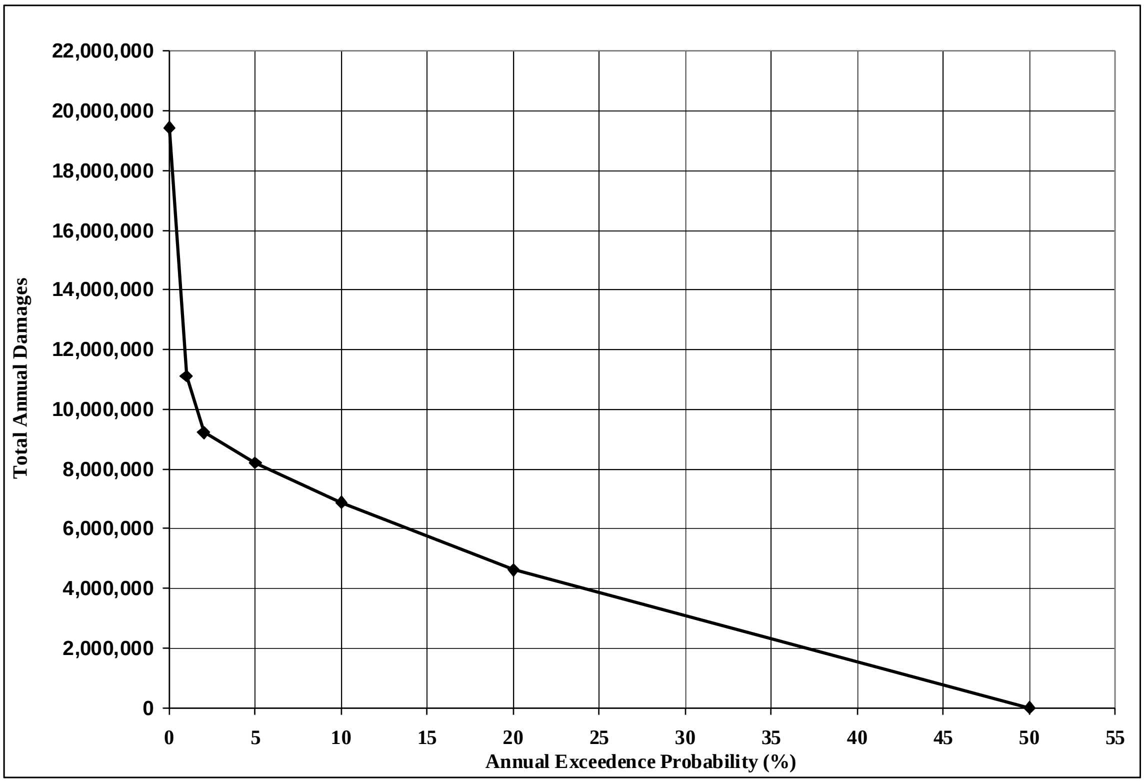


Figure 6-1 Rural Probability–Damage Curve

6.3.2 Urban Damages

Urban damages in the Lower Herbert are concentrated in the three townships of Ingham, Halifax and Trebonne and other smaller communities such as Macknade, Bemerside, Cordelia, Blackrock and Toobanna. Also included in this analysis is the damage to residential properties outside of the townships and communities. The damage to these urban areas is principally to property and can be categorised with residential, commercial and industrial sectors. The derivation of urban damages has utilised existing property survey and descriptions within the townships and stage-damage relationship developed over the last 20 years from other studies.

A basic procedure for these calculations used is provided below.

- Determine the damages due to a particular flood event using the floor levels of dwellings which are potentially flood-affected.
- Calculate the depth of flooding within each dwelling for each ARI event;
- 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 a range of flood events for both residential and commercial/industrial properties using a spreadsheet.
- Sum damages for all dwellings for each ARI event and present the results in a probability-damage graph.
- Calculate the average annual damages using the area under the graph.

6.3.2.1 Floor Levels

In order to determine damages due to flooding, it was necessary to firstly determine at what level floodwaters are able to enter buildings. Floor level survey data was not directly available. Therefore, floor levels were derived from either roof levels or the ground level. Roof height data was available in Ingham and Halifax from the respective mapping projects from which a floor height was derived by adopting the higher of the roof height less 3 m or the ground height plus 0.5 m: the latter was included to ensure that the floor level of low level buildings was not unrealistically low. For properties outside Ingham or Halifax, 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 digital elevation model at the house.

During the course of the study, the SAG 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 Council by estimating the height of the floor above the ground level; the estimate was a visual assessment from the road corridor. Surveyed floor levels of properties at Trebonne were also obtained for the analysis of a levee around Trebonne. These revised floor levels were used later in the study.

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 previously 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 longer warning times. 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.

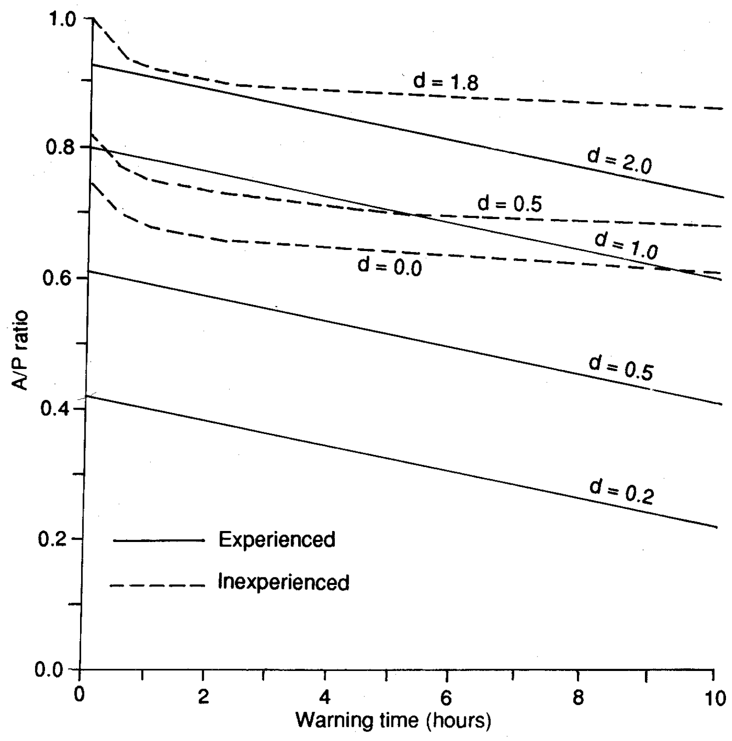


Figure 6-2 Relationship of actual/potential ratio to overfloor depth and flood experience – Sydney Flood 1986 (Smith, 1994)

Stage-damage relationships within the Herbert River region 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 considerably 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 Herbert River. However, the warning provided by EM (1999) is still applicable and until such time as more accurate curves are developed, results should be interpreted in light of these uncertainties.

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) and WBM (2002).

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. For the 20 and 5 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 Herbert 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 year and 5 year events, it is assumed that residents will be prepared, and hence, the prepared curves apply.

In addition to the number of storeys and condition of the building, the commercial damages curves also categorise the building into small, medium and large. All commercial properties were assumed to be of medium size, 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 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. The probability-damage curve for urban damages is presented in Figure 6-3.

An average annual damages of $2.2 M was then determined by calculating the area under this curve as presented in Table 6-8.

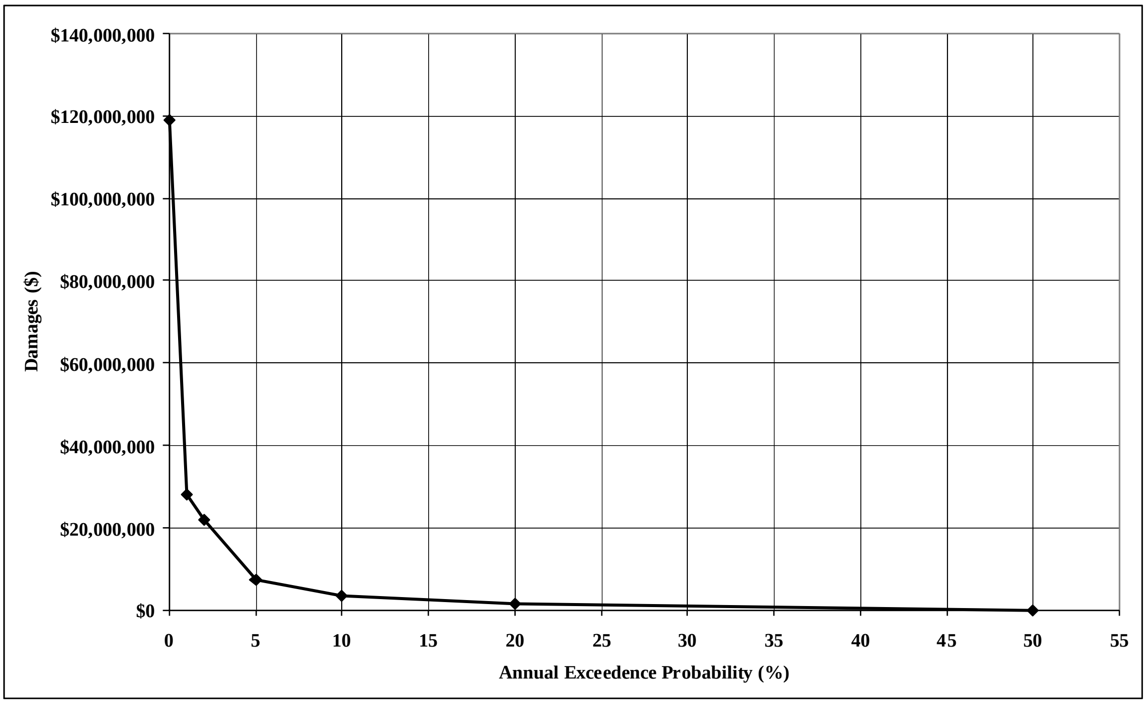


Figure 6-3 Probability Damage Curve – Residential/Commercial

Table 6-8 Average Annual Damage, Residential & Commercial

ARI Event (yr)	AEP	Existing Case ($2002)	
		Total Damages	Incremental Area Under Total Damages Curve
PMF	0%	$ 119,000,000	
100	1%	$ 28,000,000	$ 735,000
50	2%	$ 21,900,000	$ 250,000
20	5%	$ 7,400,000	$ 440,000
10	10%	$ 3,540,000	$ 274,000
5	20%	$ 1,500,000	$ 252,000
2	50%		$ 225,000
Average Annual Damage			**$2.2M**

6.3.3 Infrastructure Damages

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

6.3.3.1 HSC

HSC is responsible for various assets which are affected by flooding such as the sewage system, local government roads and DMR roads maintained by HSC under contract to DMR. Historical damages to HSC assets are provided in Table 6-9. The data does not include damage to DMR roads maintained by the HSC. As indicated in the table, the damages are for a full year. Therefore, they may include more than one flood event and may also include damage from local flooding, not just Herbert River flooding. Saturation and flood damage are defined in NDRA (2001). Flood damage is that which occurs as a result of the watercourse rising and overflowing.

Table 6-9 Historical Damages to HSC Assets

Year	Saturation Damage ($)	Flood Damage ($)	Total ($)
2000-2001	147,985	936,520	1,084,505
1999-2000	582,896	2,523,592	3,106,488
1998-1999	788,763	2,154,375	2,943,138
1997-1998	0	2,624,123	2,624,123
1996-1997	57,805	390,890	448,695
1995-1996	1,590,214	219,745	1,809,959
1994-1995	90,483	596,058	686,541
1993-1994	0	308,721	308,721
1992-1993	631,348	0	631,348
1991-1992	270,900	993,098	1,263,998
1990-1991	0	1,635,231	1,635,231

6.3.3.2 HRIT

The Herbert River Improvement Trust manages assets along the river that are affected by flooding such as floodgates and river treatment works. Historical damages to the HRIT assets are provided in Table 6-10.

Table 6-10 Historical Damages to HRIT Assets

Year	Total ($)
2000-2001	489,078
1999-2000	1,002,232
1998-1999	703,104
1997-1998	248,148
1996-1997	0
1995-1996	301,955
1994-1995	340,268
1993-1994	0
1992-1993	375,416
1991-1992	845,554
1990-1991	0

6.3.3.3 Telstra

Telstra is responsible for landline and some mobile phone communications assets through the Herbert River study area. The main assets that are affected by flooding are underground cables and above ground joining boxes. Most exchange buildings in flood prone areas including the Ingham exchange are raised approximately 4 m above the ground level to minimise flood impacts. Internal pumps minimise the impacts of flooding in the Halifax exchange, which is not raised. Access is a key issue for Telstra during flooding because costs increase when non-vehicular access is required. Discussions with Telstra personnel (Geoff Gianotti) have identified measures taken by Telstra to minimise the impacts of flooding. On notification of a flood event, vehicles are called back and moved to safe areas. Checks are made to identify which areas will be most affected by flooding, for example risk isolation or power loss, and generators are transported to those areas to service the exchanges during a blackout.

Telstra records infrastructure damage costs through booking accounts that are created for repairs due to flood events. These accounts cover the entire Townsville service region including the Herbert River area. Telstra has advised that the data can not be separated into regions, and therefore, they can not provide data specific to the Herbert region.

6.3.3.4 Ergon

Ergon is responsible for the assets required for provision of electricity through the Herbert River Study area. The main substation affected by flooding is the Ingham substation. To mitigate against flooding the control room is located on the second storey of the building. To minimise damage during a flood event, power lines are constructed 5m above the 1967 flood. The main impact of flooding identified during discussion with Ergon personnel (Darren Hoffensetz) is access, and costs are incurred from flood events when boats or helicopters are required to reach areas isolated by flooding. Once a flood warning or notification is received from the SES denoting that significant flooding is occurring in any given region, all vehicles and equipment are moved to minimise damage and maximise access to all areas.

6.3.3.5 DMR

Maintenance of DMR roads within Hinchinbrook Shire Council is undertaken by DMR, but also, HSC is contracted to maintain some sections of the DMR network. Data on flood damages to DMR roads within Hinchinbrook Shire were supplied by both DMR and HSC. The DMR data is specific to flood events and are given in Table 6-11. Data supplied by HSC are given in Table 6-12. There is some discrepancy between the data.

Table 6-11 Historical Damages to DMR Assets (Roads 10M and 10N) – Data supplied by DMR

Year	Work Undertaken by DMR ($)	Work Undertaken by HSC ($)	Total ($)
Cyclone Steve, Feb 2000	80,131	38,078	118,209
Cyclone Rona, Feb 1999	279,027	62,422	341,449
Cyclone Sid, Jan 1998	93,855	122,674	216,599
Cyclone Justin, Mar 1997	0	26,000	26,000

Table 6-12 Historical Damages to DMR Assets – Data supplied by HSC

Year	Damage ($)
2000-2001	0
1999-2000	29,971
1998-1999	169,122
1997-1998	233,124
1996-1997	344,850
1995-1996	0
1994-1995	0
1993-1994	0
1992-1993	26,075
1991-1992	0

6.3.3.6 CSR

CSR infrastructure includes the sugar rail network and the Macknade and Victoria Mills. Flood damages to the train network for floods since 1997 is given in Table 6-13. The data is grouped into years ending the 31 March. However, CSR has advised that the data cannot be relied upon to attribute to a specific flood event. For example, repairs following a flood in March may not occur until after the end of March and would therefore be included in the following year. For most years there is two entries because repairs were undertaken on separate occasions through the year.

Table 6-13 Historical Damages to CSR Train Network – Data supplied by CSR

Year Ending 31 March	Damage ($)
2000	235,800
2000	253,300
1999	243,000
1998	1,001,000
1998	411,500
1997	391,700
1997	279,200

6.3.3.7 Summary

Although it was not possible to obtain monetary damages for each of these infrastructure, it is clear from the data available that the damage to public infrastructure as a result of flooding is significant. For example, the damage to the DMR network in February 1999 from Cyclone Rona was approximately $341,450, the damage to HSC assets in that year (1998-1999) was $2,943,000 (not all necessarily Cyclone Rona), the damage to HRIT assets for the year was approximately $700,000 and the damage to CSR assets was possibly $243,000, but some of the repairs from this flood might also be included in the figure for 2000(1). This gives a total damages for that year of approximately $4,000,000 and it is likely that most of the damages were associated with Cyclone Rona. The hydrologic and hydraulic analysis indicates that the flood associated with Cyclone Rona has an ARI of about 5 years. Therefore, a relatively small flood can cause substantial damage to public infrastructure.

The flood associated with Cyclone Sid in January 1998 has an ARI of about 15 years and the total damages for that year was also about $4,000,000, although the distribution amongst the various public infrastructures was different for this flood. This indicates that it cannot be assumed that damages to public infrastructure increases with increasing magnitude of flood.

In the Lower Herbert, a 5 year ARI flood causes inundation of substantial areas of the floodplain and hence public infrastructure. Therefore, the condition of the infrastructure prior to flooding might be more significant in terms of the damages sustained than the size of the flood. For example, if there has been a long dry period leading up to a flood, the grass on road batters might be degraded, and hence, will not provide the same level of protection had it been in a healthy condition.

6.4 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.

6.5 Total Damages

Total flood damage is calculated by summing the damages for each of the damage types assessed in the previous sections, except for infrastructure and intangible damages. The probability-damage curve for total flood damages is given in Figure 6-4 and the AAD calculations in Table 3-1. An AAD of about $4.3 M was determined for total damages.

Table 6-14 Total Average Annual Damage, Total Damages

ARI Event	AEP	Existing Case ($2002)	
(yr)		Total Damages	Incremental Area Under Total Damages Curve
PMF	0%	$ 138,540,000	
100	1%	$ 39,110,000	$ 890,000
50	2%	$ 31,070,000	$ 350,000
20	5%	$ 15,600,000	$ 700,000
10	10%	$ 10,420,000	$ 650,000
5	20%	$ 6,110,000	$ 830,000
2	50%	$ 0	$ 920,000
Average Annual Damage			**$4,340,000**

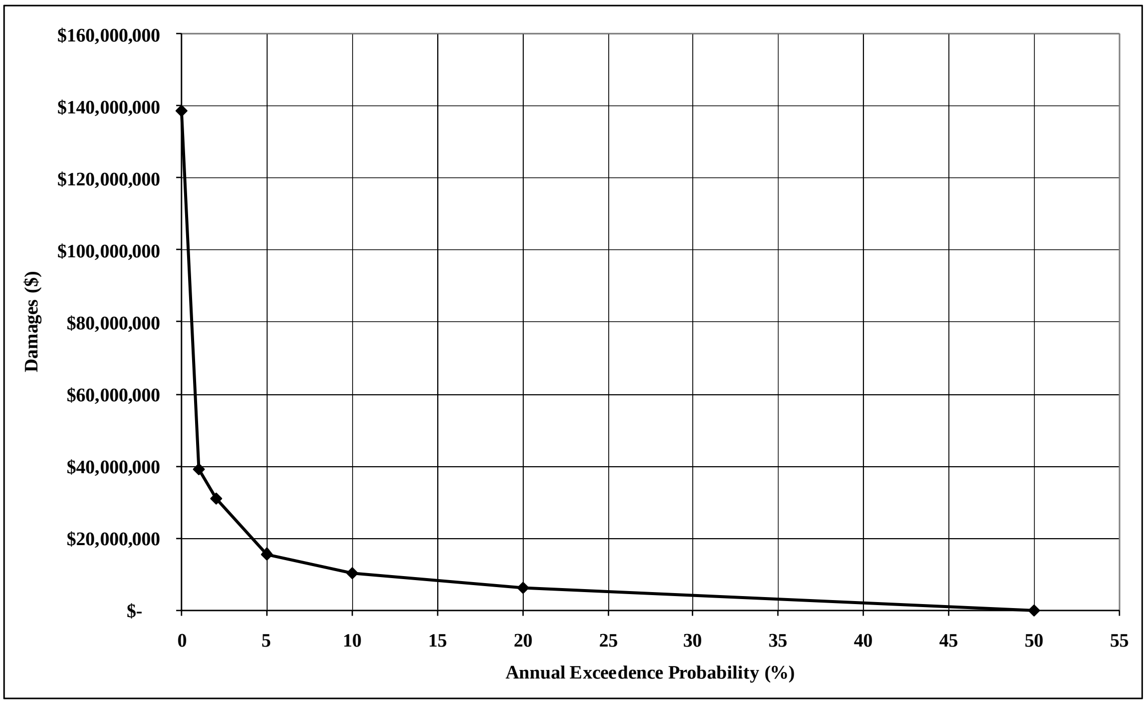


Figure 6-4 Probability - Damage Curve – Total Damages

7 ASSESSMENT OF HISTORICAL FLOODPLAIN WORKS

7.1 Background

As on most floodplains, there have been changes, both natural and man-made, to the Herbert River floodplain. The community is concerned that some of these changes may be altering the flooding characteristics. In this regard, investigations were undertaken into the following:

1. Tyto Wetlands;
2. Ingham Tip;
3. Mombelli's Levee;
4. Embankments and Levees;
5. Catherina and Ripple Creeks Floodgates.

The first four were investigated using the flood model. The investigation into the floodgates is presented in Section 8.

7.2 Tyto Wetlands and Ingham Tip

The Tyto Wetlands and the Ingham Tip are located to the south of Fairford Road on the western outskirts of Ingham. Both the wetlands and tip have created obstructions to the flow in the floodplain. In the context of the floodplain, the wetlands and the dump are in close proximity to each other and so were investigated together. Four scenarios were considered:

1. no wetlands or tip (base case);
2. only the wetlands in place;
3. only the tip in place;
4. both the tip and the wetlands in place;

Each of these scenarios was run for the 5 and 100 year ARI design floods. The analysis indicates that the tip and wetland do not significantly influence peak flood levels in the 5 year ARI flood event (Drawing 7-1 to Drawing 7-3) either individually or combined. In a 100 year ARI event (Drawing 7-4) the tip alone increases peak flood levels with the impacts extending across to the northern side of Fairford Road. At the properties on the northern side of Fairford Road the increase in flood level is in the range of about 30 to 60 mm. Floor levels of these properties would be required to determine if this increase is significant.

Drawing 7-5, shows that the wetlands alone do not significantly increase peak flood levels in a 100 year ARI flood event. However, when combined with the tip (Drawing 7-6), the impact on peak flood levels is greater than with the tip alone. The extent of impact is slightly greater and the increases in water level at the properties to the north of Fairford Road are in the range 35 to 75 mm.

7.3 Mombelli's Levee

The Mombellis own a sugar cane farm at the elbow of the Herbert River at the Halifax Washaway. This is a natural overflow point of the Herbert River during flooding. As reported in Kinhill (1991), the 1991 flood caused considerable damage to the property. In 1992 the Mombellis constructed a levee along the northern and western boundaries of their property to protect against further damage. Concerns have been raised as to the impact of the levee on flooding patterns.

To investigate the impact of the levee, the hydraulic model was run with and without the levee in place for the 1991 flood and the 5 and 100 year ARI design floods. The results presented in Drawing 7-7 to Drawing 7-9 show that the levee is impacting on peak flood levels. To the south of the levee there are widespread reductions in water level as a result of reduced inflow to the Victoria Creek region. To the north there are increases in water level. On the southern outskirts of Halifax, the increases are typically around 50 to 60 mm in each of the three floods modelled. A survey of floor levels would indicate if this increase is significant.

A review of the impact of the levee on velocity indicates increases of up to about 0.3 m/s locally around the levee and increases less than 0.1 m/s further away. Change in velocity at the peak of the 100 year ARI flood is shown in Drawing 7-10. This pattern was similar in the three floods investigated. Increases of these magnitudes do not typically alter the scouring regime in vegetated areas.

It is likely that the construction of other levees along the river prior to 1991 resulted in an increase in the flow through the Mombelli property during the 1991 flood and also increases in flood level across the floodplain. Therefore, long-term increases in flood levels in the lower parts of the floodplain should not be solely attributed to the Mombelli levee. Further, it is likely that the increases reported here are accentuated by other floodplain works, ie. if other levees had not been constructed, the impacts of the Mombelli levee may not be as substantial.

7.4 Floodplain Embankments and Levees

A large number of embankments and levees exist on the Herbert River floodplain. The significant embankments are associated with major infrastructure such as the Queensland Government Rail Line, state and local roads and CSR tramlines. Levees in the region have typically been constructed by sugar cane farmers to protect their property against flood damage. These levees and embankments may alter the flooding patterns resulting in re-distribution of flows and increases in flood levels.

To obtain an overview of the impacts of the various embankments and levees, an analysis was undertaken that calculated the change in water level across three grids in the TUFLOW model. This was not intended to give an absolute value to the increase in water level associated with a particular structure, but rather to flag structures that may be having a significant influence on flood levels for consideration in developing floodplain management measures. The analysis does not indicate the extent of influence of an increase in water level caused by an embankment.

The 100 year ARI flood was used for the analysis from 0 hours through to a time past the peak. This was carried out because the maximum impact of a blockage such as a levee typically occurs when the levee is just overtopped. As the structure becomes more submerged, the impacts normally decrease.

Drawing 7-11 to Drawing 7-13 present the results at 20 hours, 40 hours and 60 hours respectively. In the drawings there are significant water level variations shown at a large number of locations. Many of these are adjacent to the river or other watercourses where the water is "spilling" out onto the floodplain and hence there is a steep hydraulic gradient. This is not unusual at the banks of a river which is naturally elevated above the floodplain like the Herbert River, but the effects may be accentuated by the presence of man-made levees.

Along many of the road, rail and tram embankments there are substantial water level differences indicating a drop in water level across the embankment. At some locations in Ingham, changes in water level are evident along the QG rail line.

Along some embankments the water level difference varies with time. For example, there is a significant variation in water level shown at the QG rail line on the southern outskirts of Ingham at 20 hours. However, by 60 hours the water level difference is not evident. This indicates that the embankment is sufficiently submerged or "drowned out" by 60 hours as not to be significantly effecting flood flows.

A cautionary note is required in reviewing these results. A water level difference in these drawings of say 0.8 m at an embankment does not necessarily indicate that the embankment is increasing water levels by 0.8 m or that removing it would reduce upstream water levels by 0.8 m. The methodology in calculating these water levels is only intended as a tool to highlight structures that might be impacting on water levels.

7.5 Summary

An analysis of existing floodplain works has found:

1. the Ingham Tip and Tyto Wetlands do not significantly influence peak flood levels in the 5 year ARI floods;
2. the Tyto wetlands alone have no significant impact on peak flood levels;
3. in a 100 year ARI flood, the tip increases flood levels locally with increases in the range 30 to 60 mm to the north of Fairford Road;
4. in a 100 year ARI flood, the tip and wetlands combined increase flood levels locally with increases in the range 35 to 75 mm to the north of Fairford Road;
5. Mombelli's levee increases flood levels, predominantly to north of the levee, and reduces flood levels to the south;
6. Mombelli's levee does not significantly change the magnitude of the velocity across the broader floodplain, but does change the magnitude of the velocity locally around the levee;
7. an overview of embankments in the floodplain indicate the QG rail line, some road embankments and some of the CSR tramline network act as controls on flood flows and affect flood levels.

8 CATHERINA AND RIPPLE CREEK FLOODGATES

Floodgates on Catherina and Ripple Creeks were recommended as part of the 1985 floodplain management scheme (Cameron McNamara, 1985). Although the scheme was not fully implemented, the construction of these floodgates was completed. Over the years since the construction of the gates, various members of the community have raised concerns over the impacts of the floodgates. These concerns led to the inclusion of an investigation into the impacts of the floodgates into this study. The following aspects were considered:

1. The operational procedures;
2. The hydraulic efficiency of the gates;
3. Mechanical aspects of floodgate operation;
4. Flood mitigation benefits.

The brief also identified concerns relating to the impact of the gates on fish passage. In the course of the study, other environmental considerations have also been raised by the community. Although this aspect of the review was identified as a provisional item in WBM's proposal, some general comments are documented.

8.1 Operational Procedure

The current operational procedure for the floodgates requires that the gates be closed if the water level is predicted to exceed the Gairloch Bridge deck level (RL 4.6 m AHD and gauge level 4.1 m). This occurs during the first flood of the wet season. For cost reasons, safety aspects and peace of mind, the gates are not opened again until the end of the wet season, which is typically some time from April to June. During the dry season, they are permanently opened in about a 45° position.

The trigger flood level was set by the HRIT in 1993 following discussions with the Ripple Creek and Forestholme Drainage Boards and is set primarily for practical reasons relating to the gate lowering mechanism, and flood mitigation reasons. It should be noted that lowering of the gates occurs when the flood level is predicted to exceed the deck level rather than when it has exceeded the deck. Therefore, it is likely that the gates will be lowered before any significant back flow up the creeks has occurred.

The lowering mechanism is discussed in detail in Section 8.3, but in short, considerable effort is required to raise or lower the gates. If there is water flowing through the gates from the Herbert River, the force on the gates is increased making the process more difficult and potentially dangerous to the operator.

By shutting the gates early, the quantity of floodwater entering the floodplain behind the gates is potentially reduced. This may be significant in smaller floods, but as the magnitude of the flood increases, this benefit decreases.

This operational procedure has raised separate concerns from different members of the community relating to the hydraulic efficiency whilst the gates are open and the environmental impact of the closed gates during the wet season.

The operational procedure all but eliminates the flushing of the creeks with floodwater directly from the river and limits the passage of fish into the creek during floods. A revised procedure that delays the closure of the gates to allow some floodwaters into the creeks would potentially benefit the ecology of the creeks. This would be further enhanced if the gates were opened and closed with each flood rather than remaining closed for the wet season.

Members of the Foresthome Drainage Board believe that significantly more efficient drainage of their farms would occur if the gates were physically raised out of the water at times of low flow in the Herbert River rather than left at 45°. This matter is discussed further in Section 8.2.

8.2 Hydraulic Efficiency

Burrows and Emmonds (1998) reports on a laboratory experimental program on the energy head losses associated with single hinged circular flap gates. The research is directly applicable to the floodgates at Catherina and Ripple Creeks, except perhaps for the condition that the flapgate is fully submerged by the tailwater, ie, the pipes are discharging into a water body with a surface level at or above the top of the pipes. This may not be the case in these creeks when the Herbert River is low and there is significant flow through the floodgates from the local catchment. Burrows and Emmonds (1998) does not offer any indication of the likely effect this scenario would have on their experimental data, although it is possible that the loss would be greater because the flapgates may not be submerged and hence their net weight would be greater.

The design chart presented in Burrows and Emmonds (1998) was used to assess the additional head loss generated by the flapgates. As the local design flow condition for the floodgates is not known, a sensitivity test covering the range of likely velocities was tested. The results presented in Figure 8-1 show that the additional head loss decreases as the velocity increases. The maximum additional head loss calculated is approximately 0.1 m at 1 m/s. 1 m/s was the smallest velocity that could be analysed using the design chart. The additional head loss is likely to continue to increase as the velocity decreases, but it is unlikely that the velocities in the pipes are less than 1 m/s when there is a substantial local catchment flow. It is emphasised that the head loss shown in this figure is the additional head loss, not the total head loss.

These findings indicate that the additional head loss generated by the flapgates is probably not significant, but that they may be more than indicated here. Mechanisation of the raising and lowering mechanism discussed in Section 8.3 may allow the gates to be easily raised out of the water thereby allaying the concerns of the Drainage Boards.

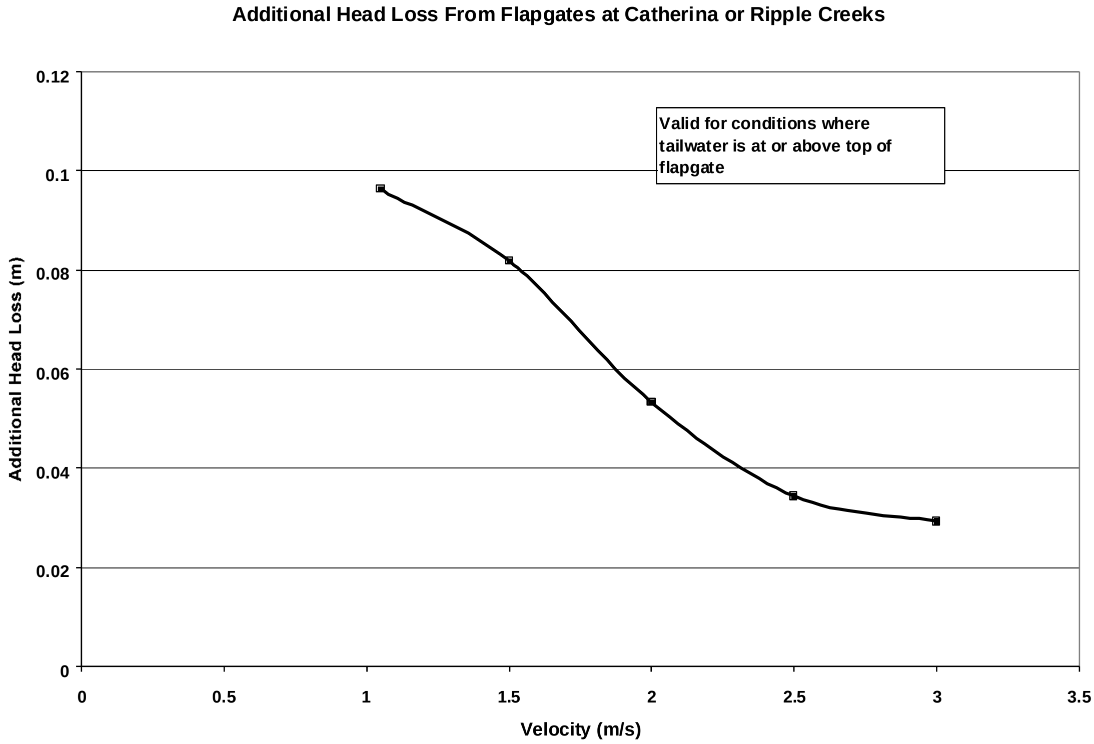


Figure 8-1 Additional Head Loss From Flapgates

8.3 Mechanical Aspects of Floodgate Operations

WBM was asked to consider possible means of mechanising the operation of the floodgates on Catherina Creek and Ripple Creek. This would enable closing of the gates under flood conditions, and reopening of the gates after each flood event. It would also potentially allow closing of the gates after a certain amount of flood water had been allowed to pass up the creeks, which may be desirable for environmental reasons.

At Ripple Creek a portable hydraulic power unit is used to power the winches, via an adaptor. At Catherina Creek, the gates are operated manually using a manually powered winch to open and close them. An incident occurred at Ripple Creek prior to converting to the power unit. When the gates were being closed during a flood event, the force of the water moving upstream caught the gate and caused it to close rapidly. This caused the winch to rotate at excessive speed, and the removable handle flew off putting the operator at risk. A similar incident could occur at Catherina Creek.

Because of the geometry of the winching mechanism, considerable effort is required in raising the gates to the open position. An attempt was previously made to mechanise the lifting of the gates, using a portable electric motor powered by a battery which was supplemented with the generator of a vehicle. This was apparently unsuccessful because of the current draw, causing overheating and depleting the battery charge too rapidly.

8.3.1 Mechanical Considerations

8.3.1.1 Force Required to Open Existing Gates

Without carrying out detailed calculations, it is nevertheless possible to state that the rigging arrangement of the ropes and pulleys used to open the gates in the existing configuration provides little mechanical advantage, particularly when the gate nears horizontal. This is the time at which the weight of the gate has the greatest moment arm, and hence requires the greatest moment to resist it. Although this is not currently required under the operational procedures, it may be a desirable feature to reduce the hydraulic losses as discussed in Section 8.2.

There are two aspects to this. Firstly, since the gate is pivoted, the mechanics of opening are dominated by moment considerations. The moment arm of the weight increases as the gate is raised, and the centre of gravity moves sideways away from the line of the hinge. To counter this moment, the rope force would be minimised if it had the maximum possible moment arm. This would be achieved by attaching the rope at the bottom extremity of the gate, increasing the mechanical advantage of the rope.

The angle of the rope as it approaches the gate from the pulley is also significant. Since the weight is acting vertically downward, the rope must provide a vertical force to balance the weight (and its moment). If the rope is at an angle other than vertical at the point of attachment, then the rope force will act along the rope, and only the vertical component of that force will be available to balance the weight. The further that the rope is from vertical, the greater must be the rope force to achieve a vertical component of the required magnitude. This principle is shown in Figure 1.

For this reason it would be best if the pulley of the rope rigging were located vertically above the point of attachment at the time when the moment of the weight is greatest. That is, the pulley should be vertically above the point of attachment when the gate is horizontal. It is difficult to judge from the available photographs and drawings, but it appears that the pulley locations may be roughly vertically above the points of attachment when the gates are horizontal, but that the points of attachment are roughly in the centre of the gates, not at the extremity.

To increase the mechanical advantage it would therefore be necessary to move the pulleys further out from the vertical wall, which would require a more robust support structure.

8.3.1.2 Counterbalancing

One approach to reducing the amount of effort required to raise the gates would be to counterbalance the gates (see Figure 2). This would reduce the force needed to tip the gate from the closed position. Currently the full deadweight of the gate acts to keep it closed. By counterbalancing, it would be possible to arrange for the gate to be opened by a chosen amount of force. This would also equate to a chosen upstream head which would open the gate. For instance, it would be possible to provide counterbalancing such that the gate opened when the upstream head was 100 mm greater than the downstream static head.

Counterbalancing would require either a weight roughly equal to the weight of the gate, located at a distance above the pivot point equal to the distance to the centre of the gate (approximately 2 metres for Ripple Creek), or a larger weight at a smaller distance from the pivot. The counterbalance could consist of a steel block 1 m x 500 mm x 760 mm, or a concrete block 1.1 m cube. Such a mass suspended in the air would present a safety hazard to operators and the public, and would require guarding to separate people from the hazardous area (such a hazard already exists in the case of the gates when open).

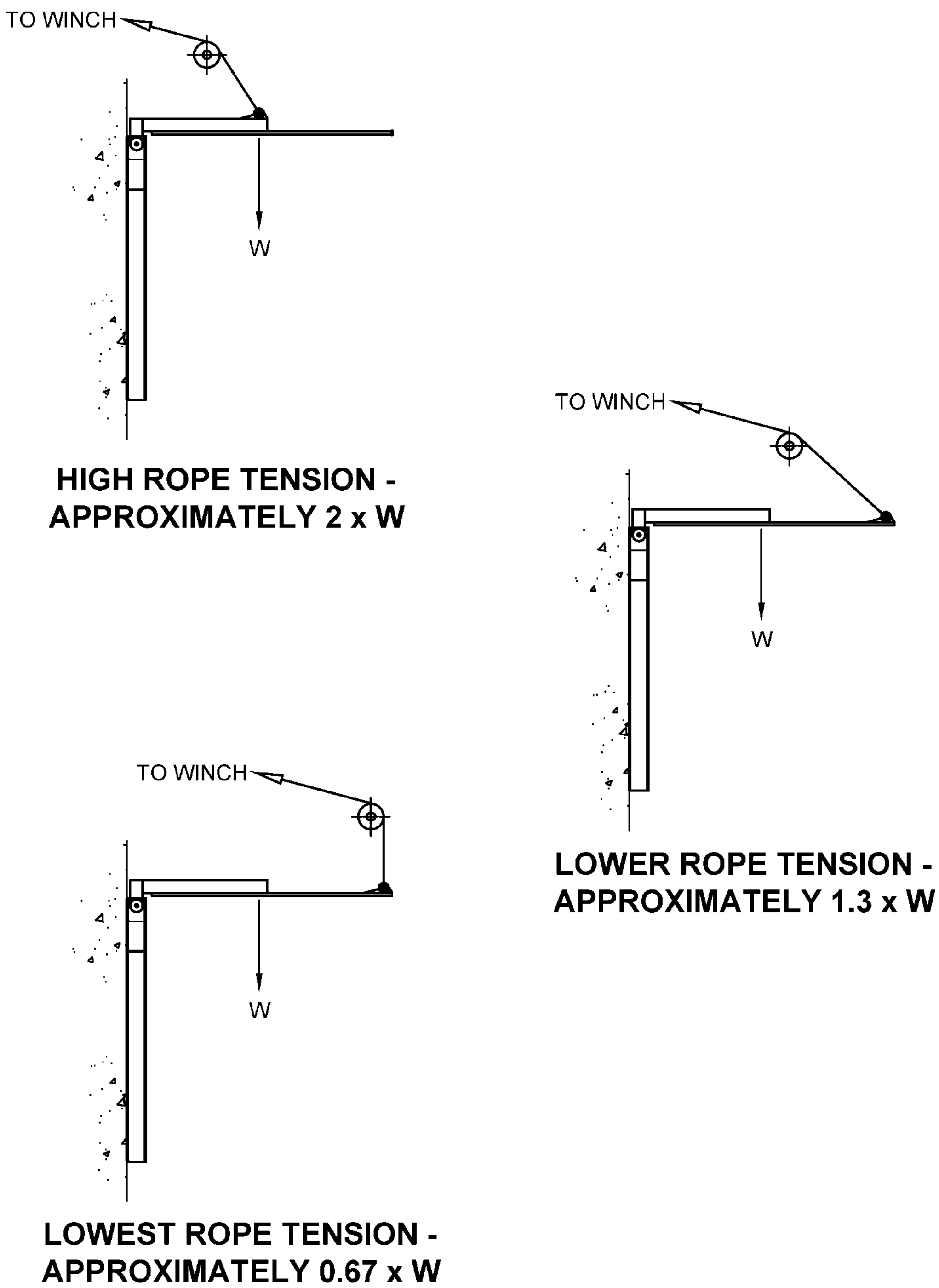


FIG 1 Effect of Rope Attachment and Pulley Position

8.3.1.3 Reeved Pulley System

The rope pull force required to lift the gate could be decreased by using a reeved system, with pulley blocks at the support point and at the attachment point on the gates, with multiple falls of rope between (see Figure 3). The rope pull force required to lift the gate would be reduced by a factor approximately equal to the number of falls of rope. However, such a system would require that the lower pulley block be at the attachment point on the gate, which would be underwater some of the time. This could cause problems with corrosion and may render the system unworkable when needed.

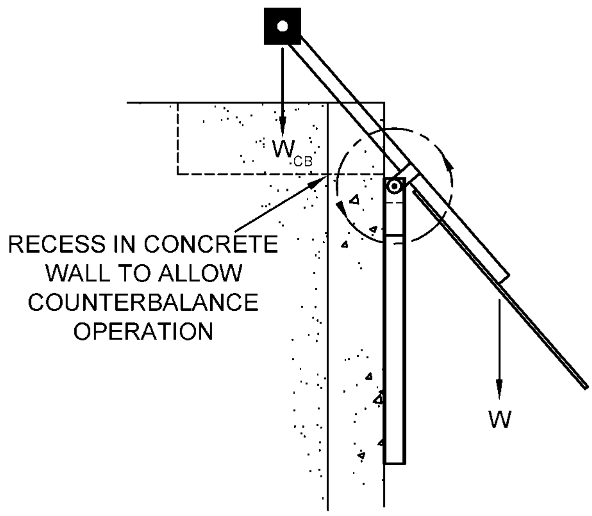


FIG 2
COUNTERBALANCING OPTION

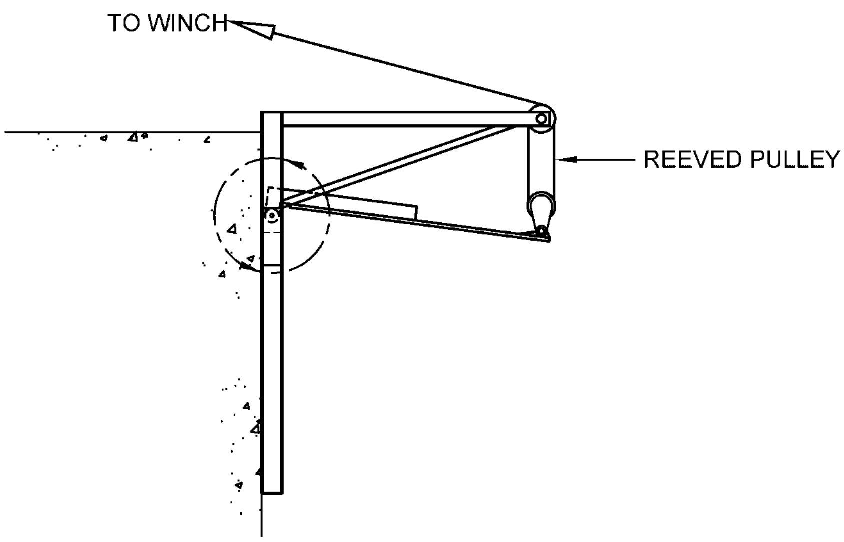


FIG 3
REEVED PULLEY OPTION

8.3.2 Powered Operating Systems

8.3.2.1 Power Requirement

The weight of the larger existing gates is estimated at 3 tonnes. This weight is lifted through approximately 1.9 m to open the gates. This represents work of 55.9 kJ. This could be achieved in 10 seconds with a power of 5.6 kW (ignoring friction and other inefficiencies), or in one minute with a power of 1 kW. This is a substantial power requirement for manual operation, but is not a particularly significant requirement for a powered device.

This power requirement could be substantially reduced by counterbalancing of the gates. A reasonable estimate would be 20% of the powers listed above.

8.3.2.2 Options

Options which have been suggested include:

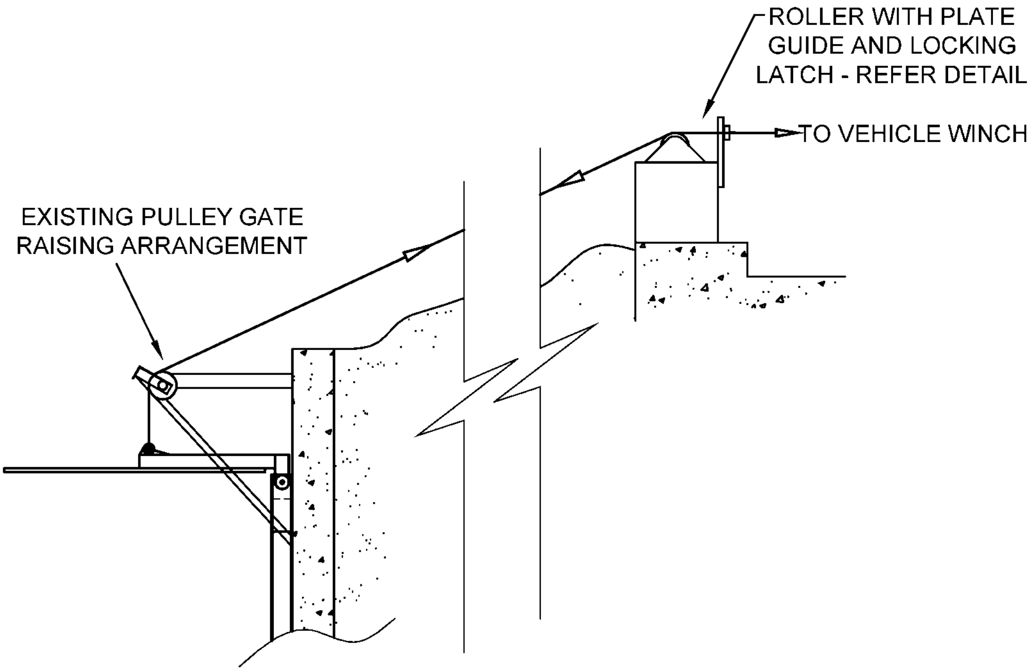
- Use of a vehicle mounted winch (typical 4WD winch) (see Figure 4).
- A fixed winch at each location, with power from a belt drive on a vehicle. The permanent installation would include suitable pulleys.
- A fixed winch at each location, with power from a pneumatic motor (similar to torque wrenches used for fitting tyres), or hydraulic motor, powered by a compressor or pump on a vehicle. The power unit would be located at each winching station in turn and locked into position to enable operation (see Figure 5).
- A fixed winch at each location, with an electrically powered motor supplied from a portable generator, or from a vehicle generator. The power unit would be located at each winching station in turn and locked into position to enable operation.
- A fixed electrically powered winch at each location, with permanent wiring and remote control.

Another option which has been suggested is the use of glass fibre reinforced plastic construction for the gates. This would potentially reduce the weight of the gates, and thereby the power required for opening, and/or the weight needed for counterbalancing. However, more investigation would be necessary to determine whether gates of adequate robustness could be manufactured using this material. A sandwich construction with considerable thickness of foam would be necessary to resist the pressure of water at maximum head.

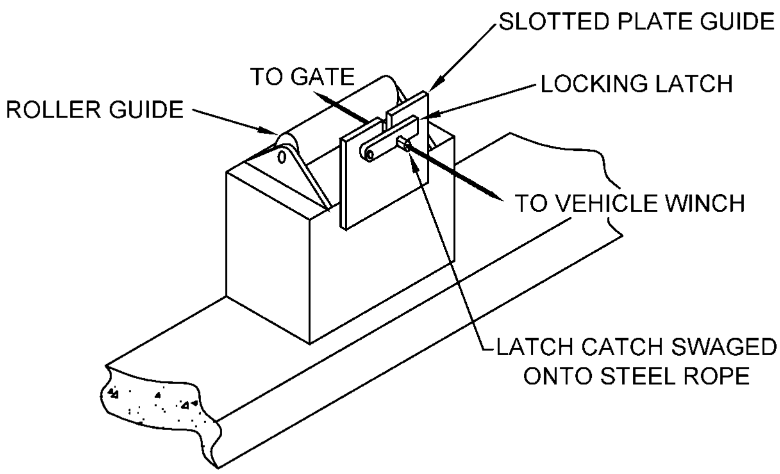
8.3.2.3 Previous Electrically Powered System

It is understood that an electrically powered portable system was tried, using a starter motor from a motor vehicle as the actuator. It is understood that while this provided adequate torque to lift the gates, the current draw was such as to overheat the motor and the vehicle battery being used to drive the system. It is likely that this occurred because typically starter motors do not have a high continuous current rating, since they usually operate for only a few seconds at a time.

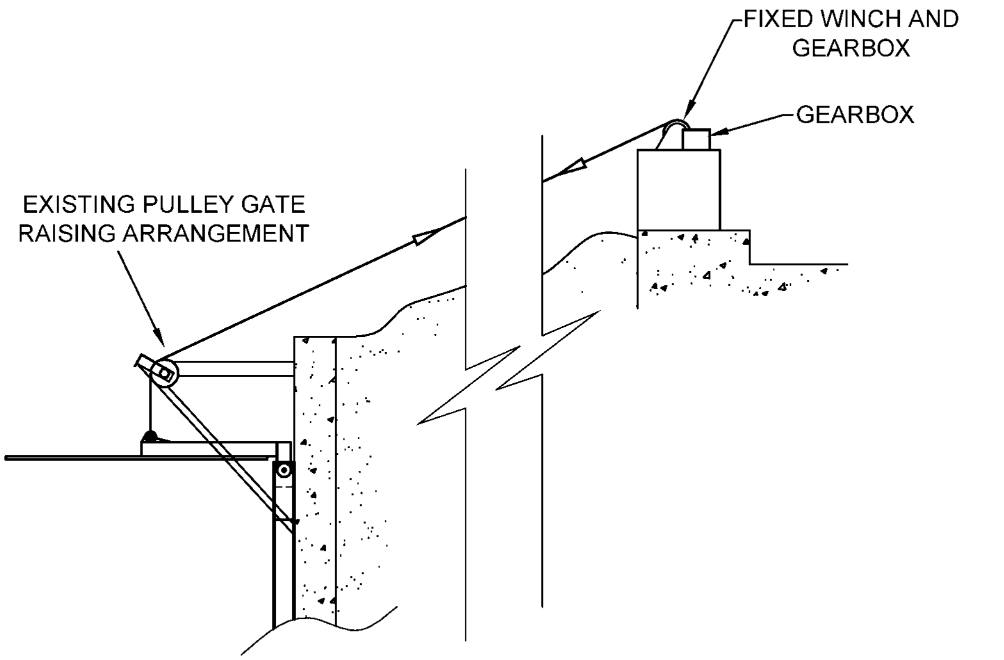
Selection of a more suitable motor, with an appropriate continuous current rating, may overcome this difficulty with the proposed system. It may be necessary also to consider altering the gear ratio between the motor and the winch drum. This could be achieved by adding reduction gears to the winches, or to the motor.



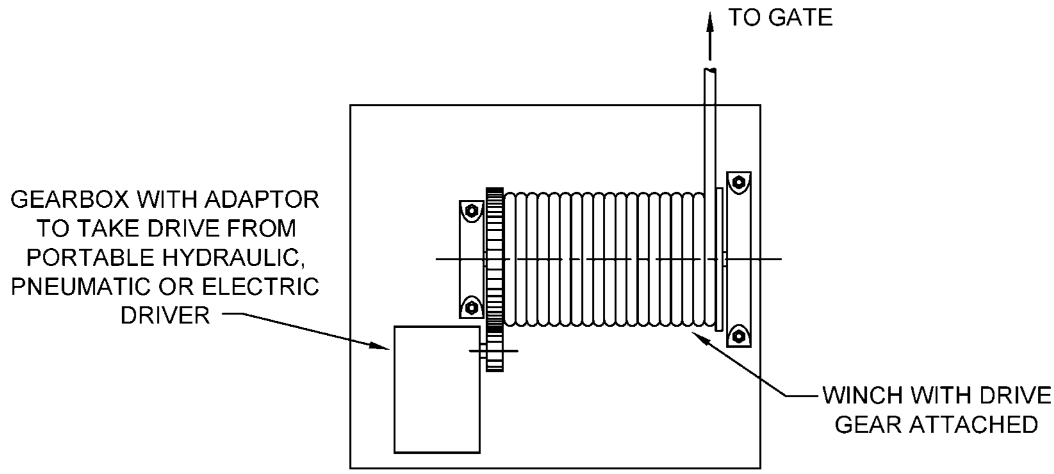
VEHICLE WINCH GATE RAISING GENERAL ARRANGEMENT



ROLLER & PLATE GUIDE WITH LOCKING LATCH



FIXED GEARBOX WITH PORTABLE DRIVE - GATE RAISING GENERAL ARRANGEMENT



PLAN VIEW OF WINCH-GEARBOX LAYOUT ON EXISTING PLINTH

8.3.2.4 Discussion

Based on the above considerations, it is quite possible to develop a system whereby a portable power unit is brought to each of the gate winches in turn. Various power sources are possible. All of these suggestions would require the relocation of the drive unit from gate to gate. It is understood that this is already done in the case of Ripple Creek, using a hydraulic power unit. The alternative would be to permanently locate a power unit (electric, pneumatic or hydraulic motor) at each gate winch. This is the only way that the effort and time required of the staff closing or opening the gates can be significantly reduced. However, such an approach would introduce problems of maintenance, corrosion and interference and tampering by the general public.

The optimum approach would therefore seem to be to locate adequate reduction gearing at each gate, so that the power source could be extremely light and portable, such as a pneumatic wrench. The operator could then easily carry the power unit from winch to winch, with the main power source being located on the vehicle. In principle this means providing additional fixed reduction gearing at each winch, so that the input speed can be higher, and the power can be provided by a high speed, low torque power unit, such as a pneumatic wrench or conventional electric motor.

An alternative system to those already discussed could replace one of the flaps with a mechanised sluice with the other flaps left as a manually operated. The manually lowered gates could be lowered under the current operational procedure and the mechanised gate lowered later in the flood thereby overcoming some of the environmental concerns with the current operational procedure. The shutting of the sluice gate could be remotely operated or automated using differential pressure sensors. This may be a cheaper option than mechanising all gates.

8.3.3 Summary of Mechanical Aspects

- The effort required to open the floodgates could be significantly reduced by adding counterbalance weight to the gate operating arms.
- If counterbalancing were introduced, then the gates could be left to automatically open and close in response to flood water flows. This would achieve the goals of preventing significant ingress of flood waters into the creeks, and allowing free flow of floodwaters out of the creeks. However, it would not allow the initial flush of flood water from the river into the creeks.
- The effort required to open the gates (and the resistance required to control closing) could be reduced by modifying the mechanical design of the lifting system. This could include moving the attachment point of the rope to the bottom of the gate and introducing a reeved lifting system. Both of these would require modification of the existing structure holding the rope sheaves.
- A number of methods of powering the operation could be developed. These would allow a portable prime mover (pneumatic, hydraulic or electric) to be brought to each winch, using a power source on the vehicle to drive it. Such as scheme is already in use at one location.
- To make portable prime movers small enough to be readily portable, additional reduction gearing would be needed at each fixed winch.
- An alternative, which would be significantly more costly, would be to place fixed motors at each winch, with remote control operation. Presumably these would be electrically powered, which may be a disadvantage in times of flood due to cyclonic action, since the power supply may be interrupted, however, portable generators could be used to enable closing of the gates in these circumstances.
- An alternative replacing one of the flaps with a mechanised sluice gate was suggested with the remaining flaps left unchanged. This would potentially address some of the environmental concerns.

8.3.4 Cost Estimates for Remote Control Powered Gate Operation

As requested, budget cost estimates have been prepared for the installation of independent powered winches at each location, with the possibility of remote operation from the central office control location in Ingham. A cost estimate for the alternate sluice gate scheme are not included.

8.3.4.1 System Parameters

For the purpose of budget costing, the following system configuration and features were assumed:

- One electrically powered winch per gate
- Arranged for sequential operation to limit electric power demand
- Power supply available within 2 km from each site
- Speed of operation to open or close each gate in one minute

The possibility of remote control operation was considered, with the following features:

- Main control station in Ingham
- Communication by radio link
- Feedback of gate position (open/closed) for each gate

A further option would be to use the communications link set up to transmit video images from a local camera.

8.3.4.2 Equipment Description and Cost Estimates

The following would be required to complete the installation:

Basic Locally Controlled Powered Winches

• 6 winches	$20 000
• Modifications to steelwork and reeving	$21 000
• Counterbalancing of gates	$20 000
• 20 m (Ripple Ck) & 100 m (Catherina Ck) power line installed	$ 3 000
• 2 x motor starter and control equipment	$50 000
Total	**$114 000**

Option for Remote Radio Control

- Base station + 2 x remote receiver/controllers $15 000
- Position switches to indicate gate position (x12) $ 9 000

Total **$24 000**

8.3.4.3 Further Option for Site Camera

If a radio or other communication link were installed, it may be possible to use the same link to transmit video information as to gate positions and local water levels.

- Camera & transmission equipment (x2) **$20 000**

8.3.4.4 Design and Supervision Costs

To achieve the above, engineering design and specification would be needed, plus supervision during installation and involvement in commissioning. Costs for this are estimated as:

- Specify equipment, produce drawings and specifications suitable for tender/purchase $17 500
- Supervision and commissioning $12 500

Total **$30 000**

8.4 Flood Mitigation Benefits

The floodgates were installed to reduce the backflow from the Herbert River and thereby reduce the frequency and magnitude of flooding in parts of the floodplain. The hydraulic model was run to determine the impact of removing the floodgates on flood behaviour. This also gave an indication of the benefits and disbenefits of the installation of the floodgates. This analysis is reported in Section 10.3.7.

8.5 Conclusion

The mechanising of the floodgates would allow changes to the current operational procedure that would reduce the potential environmental impact of the floodgates, reduce the likelihood of injury to operational personnel and reduce the head loss under local catchment flow conditions. A revised closing trigger could be analysed using the flood model. An alternative sluice gate system was also suggested, although not costed.

9 SEDIMENT TRANSPORT ASSESSMENT

9.1 Background

A preliminary overview assessment of the geomorphological processes operating in the river has been undertaken. The principal reason for this study component was to determine the need for a more detailed sediment transport study of the Herbert River, primarily related to major extraction proposals near Ingham and shoaling issues in the lower reaches near Halifax.

9.2 General Considerations

The Herbert River system supports substantial sediment loads during flood events. Sand is supplied from the upper catchment and deposited/ reworked in the lower estuary and adjacent foreshore area as part of the natural processes. Erosion and deposition of sediments can have a significant influence on the stability of the bed and banks of the river and alter the flood hydraulic characteristics. Such processes are complex and dynamic and can result in ongoing natural changes. Shoaling for example can lead to navigation difficulties and may elevate flood levels. Erosion on the other hand may lead to direct threat to property and/or structures.

Man-made influences can also have either a direct or indirect influence on sediment transport processes with associated flow-on effects. The sustainability of sand extraction is a key issue which needs to be considered. If extraction from the river system occurs and is not managed appropriately, it can lead to adverse impacts. Extraction of sediment from the active morphological system can result in changes to the deposition and re-working patterns. Such changes could include large-scale long-term alterations to the overall regime of the river and beach system. Within the overall broad picture, there is also the potential for short-term localised changes as the system evolves to the changing conditions.

In developing a future management plan for the river, there is a need to understand and consider both the long-term overall morphological processes as well as the short-term localised effects in assessing opportunities and constraints for sand extraction. This may require a detailed overall sediment budget assessment as well as consideration of potential localised impacts. The need for and extent of such studies is dependent to a certain degree on the relativities between the overall sediment supply/transport and the amount of extraction proposed or being carried.

The intent of this present study is to provide a preliminary investigation of the overall sediment transport processes in the context of sand extraction options and shoaling concerns with a view to identifying the need for a more detailed sediment budget type study.

As well as direct impacts such as sand extraction, flood mitigation strategies such as levees and floodways have the potential to change the hydraulic regime with associated flow-on effects for sediment transport processes. A broad understanding of the sediment transport regime, including erosion or accumulation trends and the potential influences of past or proposed flood mitigation works, is therefore a key consideration in determining future flood management strategies.

9.3 Review of Historical Trends

Available historical information in the form of aerial photographs, surveys and reports has been collated and reviewed to gain an understanding of past trends of erosion and deposition. This gives an insight into the relative nature and extent of changes and the degree of sediment transport.

9.3.1 Aerial Photography Assessment

The main river channel contains extensive sand and gravel shoals along most of its length with a lower flow channel meandering through the bed in certain reaches. Geo-referenced vertical aerial photographs dated 1943, 1961 and 1997 have been examined and compared to ascertain any trends or major changes in shoal and/or channel movements as an indicator of sediment transport patterns.

This visual assessment is somewhat limited by potential variations in the level and clarity of the water at the time of photography. Furthermore a direct indication of the height of the shoals cannot be obtained unless a detailed photogrammetric analysis is undertaken. Nevertheless it gives an indication of significant overall changes.

Upstream of Macknade, the majority of the river has remained remarkably stable in terms of its broad configuration over the 54 year photographic record period. The main shoals and channels are generally in the same locations with only minor localised changes in some areas. A notable feature is that a number of shoals appear to be exhibiting longer term stability, evidenced by the presence of vegetative cover in the more recent 1997 photographs. This may be a reflection of ongoing vertical growth and/or the fact that there has not been a major flood event for a number of years. While the overall configuration remains essentially the same, there are likely to be localised areas of channel migration and bank instability which are beyond the scope of this study to identify.

Downstream of Macknade, the aerial photography record indicates some changes in channel and shoal configurations. There appears to be growth and migration of some of the shoals and again substantial vegetative cover has developed on some shoals.

Downstream of the bifurcation of the river into the Enterprise and Seaforth Channels, there have been some substantial changes. The Seaforth Channel has cut off a meander and appears to be developing into the main channel with shoaling and mangrove growth, substantial in some areas, occurring in the cut-off channels and the Enterprise Channel. There have also been some substantial channel migrations and breakthroughs at the main entrances. These changes may have significant short and longer term implications for sediment transport as outlined below.

The photography also illustrates extensive shoals covering a broad area between the two entrances. Patterns have changed in these shoals over time and it is not possible to gain an accurate assessment of any net accumulation. However, the broad extent of the shoals has not significantly altered. Furthermore, there does not appear to be any direct linkages to adjacent beaches.

9.3.2 Survey Cross-Section Assessment

A number of cross-sections of the main river channel have been surveyed at various dates since 1968. Similar to the aerial photography, this survey data indicates relative stability with no broad scale changes in the location of the channels and shoals. A visual analysis of cross-section plots indicates some fluctuations with an apparent build up of some shoals, particularly at and downstream of the junction with the Anabranch. The general observations made from the aerial photography outlined above are supported by this data.

9.3.3 Historical Reports

A summary of the historical accounts of the river is provided in the Stream Management Plan (ID&A, 1993). The majority of the reports dating back to early explorers and settlers support that the Herbert River has always been shallow, sandy and difficult to navigate.

9.4 Assessment of Sediment Transport Regime

9.4.1 General Processes

Sediment in the form of gravel, sand and finer silts is eroded from the catchment and upper reaches of the river system and transported downstream during times of significant rainfall and runoff. The coarser sand and gravel particles are typically transported as bed load along the bed of river with some in suspension close to the bed. This is generally referred to as the total bed material load. The finer silty material is generally transported in suspension along with the flow and is generally referred to as the wash load. These two combined together form the total sediment load for the river.

It is important to recognise the difference in these sediment transport parameters particularly when interpreting quoted sediment discharges. This particular study is related to the coarser bed material load component which is typically transported gradually down the river system during flood events and stored along the river bed between events. The wash load component is often a significant, if not the major, contributor to the total sediment load. However, the majority of this material is either deposited on the floodplain in lower velocity areas or transported out through the entrance and deposited offshore, again where velocities are lower.

The sediment transport potential of the river is dependent on the hydraulic parameters in terms of flow, velocity, depth of water and the hydraulic roughness of the bed. The actual transport is dependant on these parameters as well as the characteristics (typically grain size) and availability of the sediment. All of these parameters vary along the river and from flood to flood and accordingly the sediment transport regime is complex and dynamic.

Erosion typically occurs where the sediment transport capacity increases and deposition where the capacity decreases. Coarser gravel sediments are typically found in the upper reachers where the flow and sediment transport capacity is greater, and are only transported further downstream during large flood events. The finer sandy sediments are mobilised under most flow regimes and are therefore typically reworked and found along the majority of the river bed.

Bank erosion may also occur where the channel location is shifting laterally such as where the river meanders. In such case, sediment is often deposited on the opposing side of the river, and there may be no net differential in the sediment transport capacity if the erosion and accretion quantities are matched.

General geological evidence suggests that, over the longer term (ie. past 6,000 year Holocene period) with sea levels at their present post-glacial high, there has been progressive deposition of both coarser bed load and fine wash load on the coastal floodplain. This is consistent with a process of reducing transport capacity along the river leading to deposition in some reaches.

9.4.2 Existing Information

Various estimates of the sediment budget of the Herbert River have been previously made and reported as follows.

The Stream Management Plan (ID&A, 1993) quotes an estimate of the average annual sediment discharge through the river mouth of 2 million tonnes, referenced as K. Smettem(CSIRO, pers. comm. 1992). It also quotes an annual sediment load of approximately 1,000,000 m^3 (1,590,000 tonnes) elsewhere although the source or nature of this estimate is not stated. The magnitude of these estimates are such that they are likely to reflect the total sediment discharge including wash load.

The Great Barrier Reef Catchment Water Quality Action Plan quotes a current export of suspended sediment as determined by the National Land and Water Resources Audit of 664,787 tonnes per year.

As well as the above estimates, the Department of Natural Resources has instigated a program of sediment sampling of major Queensland Rivers to assess sediment transport rates based on direct flood measurements. This program was initiated to assist in management of the rivers, particularly with respect to defining sustainable extraction rates. The findings to date are summarised in a progress report of Queensland Riverine Sediment Transport Rates (DNR, 1998).

The Herbert River has one measurement site referenced as Ingham which has been gauged during 4 moderate floods from 1994 to 1998. The discharges during these measurements were between about 250 and 2,500 m^3/s and were typically on the falling leg of the flood. Based on these measurements and the flow record from 1982 – 1998, the above report quotes an average bed material load of 80,200 tonnes/year (50,100m^3/yr) and an average wash load of 248,000 tonnes/year.

It is recognised that these rates are based on limited data and during a period in which flood events did not exceed an ARI of 12 years (refer Table 5-2). Hence, long term average rates are likely to be higher than those calculated.

9.4.3 Preliminary Calculation of Sediment Transport Rates

As part of the present study, preliminary calculations of sediment transport rates (bed material load) and variations along the river have been carried out. There are a number of empirical and quasi-theoretical relationships which have been developed for the purposes of estimating sediment transport rates. The complexity and variability of the processes are such that it is generally accepted that calculations of this nature can only be used as a guide.

For the purposes of the study, the method of Engelund-Hansen (1967) was adopted. An assessment of sediment transport potential was calculated at the Ingham Pump Station section for the 100 year ARI design flood using hydraulic parameters from the flood model.

As discussed above, the river transports a range of grain sizes from fine sands and silts up to coarser gravels. The particle grain size is an important parameter and significantly affects the calculated rates. Accordingly, two grain sizes have been used as part of this initial preliminary assessment (1.0 and 2.0mm) based on a visual assessment as no detailed grain size information was available.

Relationships between discharge and sediment transport rate were determined using the 100 year ARI design flood results and applied to the other design flood events (50, 20, 10 and 5 year ARI). The instantaneous transport rates were cumulated over the whole hydrograph to give a total sediment (bed material load) discharge at the section for each design flood and grain size. The total sediment discharges for each event were then combined in a probabilistic manner to determine an annual average sediment transport rate. The results are summarised in Table 9-1.

Table 9-1 Calculated Sediment Transport Discharges – Ingham Pump Station

Design Flood		Sediment Transport (cu m)	
ARI (yrs)	AEP (%)	1mm Sand	2mm Sand
100	1	917539	376040
50	2	717339	293991
20	5	478848	196249
10	10	310044	127067
5	20	145560	59656
Annual Average		**90454**	**37071**

As well as the above, the instantaneous sediment transport rate at the time of peak flow for the 100 year ARI flood event was determined at 27 cross-sections to give an indication of the variations along the river. The locations of these cross-sections are shown in Drawing 9-1 while the results are presented in Figure 9-1. These results are for a 1mm grain size sediment.

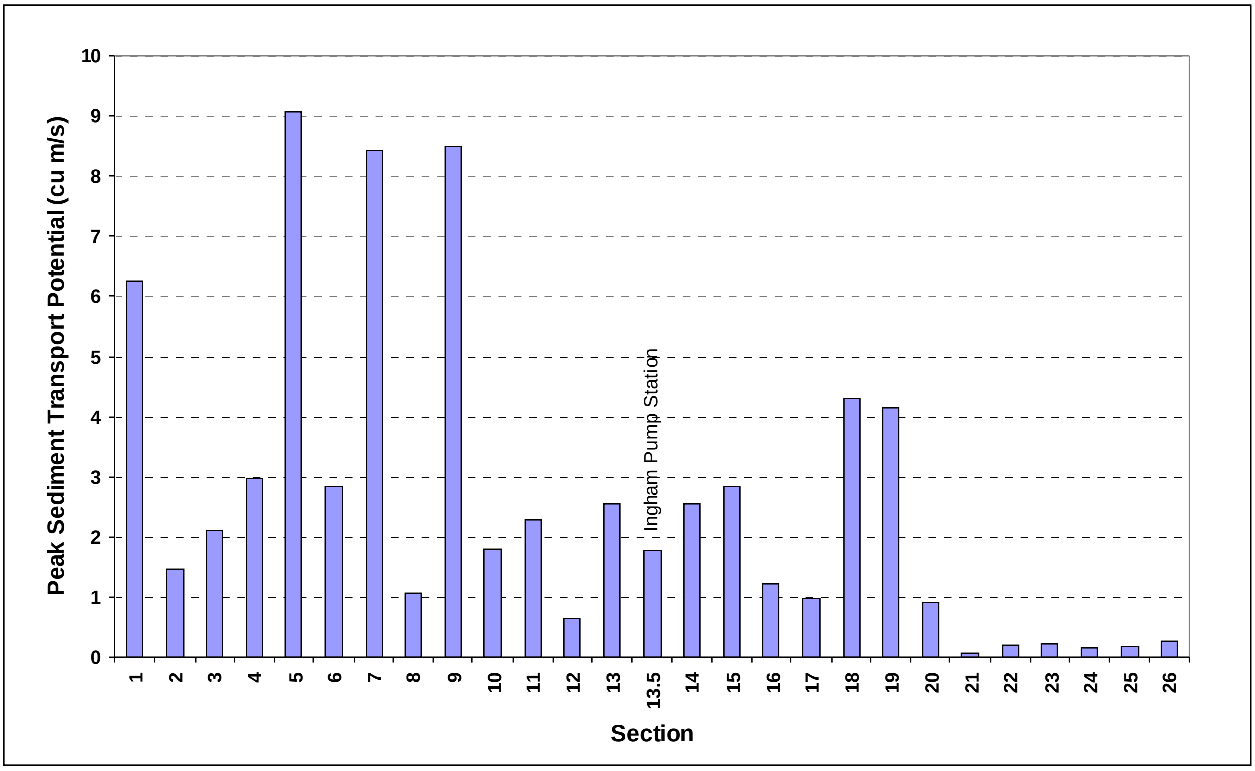


Figure 9-1 Peak Sediment Transport Potential Along River

9.4.4 Discussion of Findings

The previous reports, measured transport rates and calculated transport potentials illustrate the degree of variability and uncertainty with such assessments. Given the nature and limited time base of the rate based on measurements (50,100m^3/yr), it is considered likely that this would be at the lower end. The basis and nature of the high rates quoted in the Stream Management Plan are unclear.

The calculated transport rate capacities illustrate the importance of grain size with annual averages varying from 37,000m^3/yr for 2mm sized sand to 90,000m^3/yr for 1mm sized sand at the Ingham Pump Station. It is considered that a rate of 100,000m^3/yr would be suitable to adopt for initial planning purposes. The influence of major flood events is evident, with a transport potential of 900,000m^3 predicted for 1mm sand at the Ingham Pump Station in a 100 year ARI flood while the calculated potential at this location for a 5 year ARI event is around 150,000m^3.

Variations along the river are to be expected and are evident in the calculated rates at the various cross-sections. As well, the calculated transport capacities may not be met if there is insufficient sand of that size available, leading to exposure and transport of coarser sediments and potential ‘armouring’ of the bed. As a general trend, the rates are higher in the upper reaches and more confined sections where velocities tend to be higher. Lower transport rates are predicted in the broader reaches where velocities tend to be lower. These areas are typically deposition zones as evident by broad shoals in those areas.

Of note is the relatively low transport rates calculated downstream of Macknade in comparison to the remainder of the river. This would tend to indicate that this area would be zone of greater deposition which is somewhat supported by the aerial photography and survey analysis as discussed in Section 9.3. Such deposition may lead to the substantial channel meandering or switching in location as indicated. It is also possible that significant deposition may occur in this area during the more frequent moderate flood events with the sand ultimately being carried further downstream to the entrance region during less frequent major flood events.

9.5 Considerations for Extracting Options

The potential implications of sand extraction with respect to the sediment transport regime of the river relate primarily to the extent of the extraction relative to the supply and hence recharge capacity of the river.

As described above, the river sediment transport processes are highly variable and dynamic and are related to the size and frequency of flood events. The sediment is gradually moved down the river system and ultimately deposited in the lower reaches and extensive shoals at the entrance to the river. The magnitude, and place of any such deposition will be dependent on the characteristics of the flood.

In broad terms, if more sand is extracted than is being transported down the river, there will be long-term degradation of the bed. This could potentially initiate or exacerbate bank stability problems and/or other environmental impacts. If the extraction rate is less than or equal to the rate of supply in the long-term, it will tend to slow the rate of accumulation.

The evidence is that there is net deposition of sediment on the coastal floodplain. However, there is considerable bank erosion and channel switching probably promoted by the net deposition. Thus, even in that case, any extraction of material, while acceptable in principle, would need to be carefully planned to avoid impacts on properties.

It should be recognised that the supply and transport of sediment is dependent on flood events whereas extraction would more than likely occur at a more uniform rate. Extraction activities would typically leave sections of the river with a greater cross-sectional area than its equilibrium condition and hence sediment would tend to redeposit or recharge that area during flood events. However, the sediment trapped would starve downstream reaches which have a greater sediment transport potential thereby leading to erosion in those areas. Localised changes in the hydraulic characteristics may also induce localised effects. As an example, the flood gradient will be flatter through the extraction zone and steeper both upstream and downstream. This typically induces higher velocities and erosion upstream and downstream while velocities through the extraction zone will be reduced leading to deposition.

9.6 Consideration of Downstream Shoaling Issues

As discussed above, the sediment supplied and transported down the river is typically deposited in the lower reaches and entrance area where the velocities and transport potentials reduce. This is evident by the extensive shoals at the river entrance as well as in the lower reaches.

The extent of such deposition is dependent again on the long-term average supply rate but will occur variably depending on the occurrence of flood events. Furthermore, the location will be dependent on variations in sediment transport potential. While a supply rate of, on average, 100,000m^3/year may seem quite high, it is deposited over a large area.

The visible shoals at the river entrance cover an area of about 8km^2. Incorporating the lower estuary as well gives a total deposition area of the order of 10km^2. Assuming that the whole supply rate discussed above is deposited over this area equates to 1cm/yr on average or 1m over a 100 year period. The actual deposition would not manifest itself as purely vertical growth nor would it be expected to be uniform over the whole area. Some lateral growth of shoals is likely to occur with channels being maintained by tidal flows in the lower estuary.

Historical evidence indicates that the river has always been shallow and difficult to navigate in the lower reaches. Ongoing natural processes of continued net deposition could be expected to maintain or exacerbate that situation. If dredging is undertaken with a view to improving navigation, the natural sediment transport processes would act to infill those dredge areas and restore them to their pre-existing natural configuration in time. This would necessitate ongoing dredging to maintain navigation channels in the long-term with other potential implications as discussed above.

9.7 Consideration of Management Measures

One of the flood mitigation measures assessed incorporates substantial dredging of the riverbed downstream of Ripple Creek. Peak sediment transport potentials have been assessed for this scenario utilising results form the 100 year ARI design flood simulation as discussed above.

The results indicate a reduction in the sediment transport potential in the dredge area and a slightly higher potential immediately upstream which reflect the altered hydraulic conditions. As such, it is likely that this would lead to deposition in the dredge area with some depletion immediately upstream and further downstream compared to the natural situation. If no further works are undertaken, it would be expected that in time, the natural processes would gradually redistribute the sediment loss along the riverbed with an overall depletion equivalent to the quantity extracted.

Consideration also needs to be given to the impacts of extraction programs on downstream areas where there may be no extraction occurring. If extraction is generally maintaining or increasing the capacity of the main channel and further downstream there is deposition of material occurring over time, this may lead to an increase in flood level at the downstream location due to the relative constriction.

9.8 Conclusions

The Herbert River contains substantial quantities of sand and gravel. Preliminary estimates of the likely sediment transport rates have been made based on available information and the results of the hydraulic flood modelling. These indicate that the long-term average transport of sand sized particles is likely to be of the order of 100,000m^3/yr. Substantial variations and differentials occur along the river and from flood to flood resulting in zones of deposition and erosion. Broad deposition of this material is likely to be occurring in the lower estuary and entrance region of the system where extensive shoals have developed. However, it is unlikely that any adjacent beach areas rely on an ongoing supply of sand from the Herbert River.

The Stream Management Plan (ID&A, 1993) considers extraction rates at that time (10,000m^3/yr) as being a very small percentage (1%) of the estimated annual sediment load (1,000,000m^3). As such, it concludes that there is unlikely to be a significant impact on river morphology unless extraction rates are significantly increased. As outlined above, this present study indicates that the actual bed material sediment load is likely to be substantially less than the value used in drawing that conclusion. As the sediment supply rates is a key consideration in many management issues such as sand extraction, it is considered that there would be merit in confirming that rate.

In the context of a depositional environment, it may be expected that some sand extraction could occur without any long-term detrimental effects provided that the quantity does not exceed the supply rate. However, such extraction in a localised area may induce localised impacts and hence would need to be planned and executed on the basis of more detailed studies. These studies would need to confirm the broader processes in more detail as well as address the localised issues.

Extraction of a large quantity of sediment or extraction at rates substantially greater than the supply rate could be considered as not being sustainable from the point of view of degradation of the river bed. However, if large scale extraction is warranted on other grounds (eg. flood mitigation purposes), detailed studies could be undertaken to ascertain the extent of any likely degradation and any other associated impacts. Such studies may find that the impacts are small and/or manageable with appropriate planning.

The dynamic sediment transport processes occurring in the river are likely to be contributing to a range of existing river management issues such as bank instability. A more detailed study of such processes would assist the understanding of the issues and potential mitigation options. This would again require more detailed assessment of the broader processes as well as those contributing to the local issues.

In summary, it is concluded that more detailed studies of the sediment transport processes would be beneficial in providing a better understanding of those processes and a basis for assessing a broad range of river management issues. These benefits would include:

- confirming the important bed material load proportion of the total sediment discharge;
- identifying differentials in transport rates along the river and zones of erosion and deposition;
- allowing potential extraction zones to be identified in the knowledge of the processes and other issues;
- identifying how the morphological processes may be contributing to existing river management issues; and
- allowing existing problems and the beneficial or adverse effects of sand extraction to be considered together in a co-ordinated approach.

10 FLOOD MODIFICATION MEASURES

10.1 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. The SAG elected to undertake the investigation into flood modification measures using the following process.

1. Compile possible measures using input from the community, SAG and WBM.

2. SAG reviewed measures and selected some for preliminary investigation.

3. Flood height impacts of measures were analysed by WBM, typically using one large flood. A flood height impact animation was produced.

4. SAG reviewed flood height impact animations and selected those measures for which a preliminary benefit-cost analysis was to be undertaken using the Rapid Appraisal Method (RAM).

5. Benefit-cost analysis by WBM.

6. SAG reviewed the preliminary BCR and costs of each measure and selected those measures for which detailed analysis was required.

7. Detailed assessment by WBM.

8. SAG reviewed detailed analysis and recommended measures for inclusion in the Floodplain Management Plan.

10.2 Flood Modification Measures Considered

A workshop was incorporated into each of the community open sessions held in December 2001 to allow the study team to obtain ideas from the community on possible flood modification measures. This information was compiled and presented to the SAG following the open sessions at which time the SAG contributed additional ideas. Table 10-1 lists all the flood modification measures identified and the SAG’s decisions on the level of analysis and recommendations. Some of the items in the table are comments from the community rather than measures. Further details of the measures that were modelled are given in subsequent sections.

Table 10-1 Flood Modification Measures & SAG Decisions

Measure	Preliminary Hydraulic Analysis	Preliminary BCR	Detailed Analysis	Recommended by SAG
River/Creek Maintenance				
Dredging rivers Option 1 Option 2 Option 3	 Yes Yes Yes	 Yes No No	 No No No	 No No No
Clearing of creeks – especially at outlets	No	No	No	No
Permanent permits to keep waterways clean	No	No	No	No
Catchment management (Identify source of sediment in upper catchment)	Beyond scope of this study			
Diversion Channels				
From Washaway to Victoria River	Yes	No	No	No
From Stone River into Trebonne River	No	No	No	No
Increase flow through Palm Ck	No	No	No	No
Short-circuit Anabranch to increase efficiency of flow into Seymour river	No	No	No	No
Levee Banks				
Levee banks at appropriate position – levee rationalization	Yes	No	No	No
No more levees	No	No	No	No
Remove levee at Managers Hill	No	No	No	No
Cane rail line at Sachs Lane	Local drainage issue			
Remove all Levees	No	No	No	No
Remove Floodgates	Yes	No	No	No
Manage construction of levees	Planning Issue			
Remove Mombelli's levee but construct diversion channel to Victoria River	No	No	No	No
Levee bank from washaway to Halifax township	Yes	No	No	No
Groyne in Washaway to direct flow to inside of bend	No	No	No	No
Ingham Levee Option 1 Option 2	 Yes Yes	 No Yes	 No No	 No No
Levees at Trebonne, Toobanna, Blackrock, Cordelia, Bemerside, Macknade	Yes	Yes	No	No
Levees at Halifax	Yes	Yes	Yes	Yes
Kingsbury Ck Floodgate	Yes	Yes	Yes	Yes
Miscellaneous				
Dam	No	No	No	No
Repair 1927 washout at Gairloch	No	No	No	No
Bradfield Scheme	No	No	No	No

10.3 Preliminary Analysis

Those measures for which a preliminary hydraulic and BCR analysis only was undertaken are presented in Section 10.3. The two measures selected for detailed analysis, the presentation of the preliminary analysis is left until Section 10.4.

10.3.1 Methodology

10.3.1.1 Calculation of Impacts

The preliminary analysis only investigated the impact of the proposed flood modification measures on flood heights using one design flood. Other impacts such as change in velocity were considered at the detailed analysis stage. For most measures the 100 year ARI design flood was used because it also gave an indication of the impacts in smaller events by reviewing the results during the rise and fall and the flood. The TUFLOW hydraulic model was used to assess the impact on flood levels by modifying the base case (calibrated) model with the proposed scheme. For example, a levee would be added to the base case model. The flood levels from the modified case were then compared with those from the base case to determine the change in flood level predicted to occur if the proposed scheme was implemented.

Typically, only one scheme was modelled at a time so that the full impacts of the measure could be determined without interference from another measure. The exception to this was the analysis of the town levees where it was considered that some were far enough apart that they could be combined into one run.

10.3.1.2 Determination of Benefits

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

- increased flood immunity of properties protected by the measure;
- 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. Of the factors listed above, the change in flood damages is the only one that can be easily quantified in monetary terms. In Section 6.3, the flood damages for the existing situation have been calculated. The reductions (or increases) in these damages have been calculated to quantify the monetary benefit of each measure.

10.3.1.3 Determination of Costs

The costs associated with the construction of the floodgates and levees include the items listed below. The approximate unit rates are provided in Section 10.3.1.6.

Earth Levee Construction
- earthworks construction with the material obtained from within a 10km radius

Road Reconstruction
- road removal
- kerb and channel
- rubble road base (in place)
- road sealing

Landscaping
- provision of mounds and mulched gardens including trees and shrubs
- provision of a 50mm layer of loam sown with couch grass seeds and maintained for 6 months
- provision of general architectural treatment to the surface of a concrete levee

Floodgates
- estimate of construction costs for floodgate structure
- operation of floodgates

Mowing
- mowing of earth batters

Garden Maintenance
- estimate for maintenance of gardens

Community Education
- in relation to levee

Levee Monitoring
- ensuring levee remains intact
- annual survey to determine levee heights are maintained

Dredging
- dredging to stockpile within 3 km
- washing
- transporting to market

On-Cost
- design
- survey
- geotechnical investigations
- contract administration

10.3.1.4 Explanation of Benefit-Cost Analysis and Present Worth of Benefits

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. 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 previously been awarded 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 10-2 shows the present worth of the annual benefit realised at different times over a 50 year period.

Table 10-2 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.

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

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 non-monetary 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.

10.3.1.5 Social and Environmental Issues

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

- *Impacts on flood response and evacuations;*
- *Impacts on riverbank stability*;
- *Public utility impacts* – 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.

10.3.1.6 Unit Rate Costs of Works

The unit rate costs are from the *Lismore Levee Scheme EIS – Volume 3, Working Paper No 10* (SKM 1999), unless specified otherwise.

Table 10-3 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
Road Construction		
- kerb and channel	m	$ 21.40
- median	m	$ 21.40
- road removal	m^2	$ 1.60
- rubble road base (in place)	m^2	$ 9.00
- road sealing	m^2	$ 19.25
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 10-4 Unit Rates for Levee Maintenance Costs

Item	Unit	Annual Rate
Mowing		
- mowing of earth batters	m	$ 4.00
Garden Maintenance		
- estimate for maintenance of gardens	m	$ 10
Gate Structure Maintenance		
- maintenance of "openings" through the levee	gate	$ 2,000
Pump Station Maintenance		
- estimate of maintenance of pumps	pump	$ 1,000
Community Education		
- in relation to levee		$ 4,000

Table 10-5 Unit Rates for Levee Operation Costs

Item	Unit	Annual Rate
Gates		
- operation of openings in the levee	gate	$ 40.00
Floodgates		
- floodgate operation	gate	$ 10.00
Levee Monitoring		
- ensuring levee remains intact and regular surveying of levee (every 5yrs or so)	m	$ 2.50
Electricity		
- pump stations estimate		$ 100

Table 10-6 Unit Rates for Dredging Operations

Item	Unit	Annual Rate
Dredging to Stockpile (up to 3 km)	m^3	$ 4.50
Washing Sand	m^3	$ 2.50
Transport		
- Ingham to Cairns	m^3	$ 25.00
- Ingham to Townsville	m^3	$ 20.00
Selling Rates		
- from stockpile	m^3	$ 1.00
- to market	m^3	$ 15.00

10.3.2 River Dredging

10.3.2.1 Benefit Cost Analysis

In Section 9, with reference to sand and gravel it was concluded that, "*Broad deposition of this material is likely to be occurring in the lower estuary and entrance region of the system where extensive shoals have developed*'. The lower estuary was considered to be below the Anabranch. The long-term deposition of material in this region of the river is likely to have increased flood levels because of a reduction in the conveyance capacity of the river channel. Discussions at the community open sessions indicated that this is a belief that is held strongly by the community and consequently the community indicated a strong desire for dredging to be modelled. The following three options were modelled.

Option 1. Reduction of bed level by 2 m and reduction of shoals to RL 0.0 m AHD in the Herbert River from approximately Trebonne to the Hinchinbrook Channel through both the Seaforth and Enterprise Channels (approx 16,000,000 m^3 of material removed).

Option 2. Reduction of bed level by 1 m and reduction of shoals to RL 0.0 m AHD in the Herbert River from approximately Ripple Creek to the Hinchinbrook Channel through both the Seaforth and Enterprise Channels.

Option 3. Reduction of bed level to minimum RL –3.0 m AHD and reduction of sand shoals to RL –1.0 m AHD in Herbert River from the Washaway to the end of sugar cane region downstream of Halifax. Downstream of sugar cane in Seaforth channel the bed level was reduced by 1.0 m (approx 3,000,000 m^3 of material removed).

Option 1 was not considered a realistic option, but was assessed to demonstrate the upper limits of possible benefits as a result of dredging the river bed. Options 2 and 3 were considered more realistic, although they would still require the removal of substantial quantities of bed material.

The extent of dredging and the change in peak 100 year ARI flood level for each of the options is shown in Drawing 10-1 to Drawing 10-3. It can be seen from each of these drawings that there are reductions in flood level over large areas of the floodplain, but also regions where the flood level would increase. Options 1 and 2 both show that there would be increases in flood level in the lower reaches of the river around Halifax and in Option 1 there would be increases at Ingham .

Based on the results from the preliminary investigation on the hydraulic model, the SAG decided that the investigation into Options 1 and 2 would not continue beyond the flood level impact analysis, but that the analysis of Option 3 would continue through to the calculation of a preliminary BCR.

The BCR calculations considered three scenarios: dredge to stockpile and sell locally; dredge to stockpile and transport to Townsville and/or Cairns; and sell to overseas markets. A dredge quantity of 3,000,000 m^3 with a 10% silt content was assumed for the analysis. The unit rates for the calculations are given in Table 10-6. The analysis does not include maintenance dredging which would further reduce the BCR. In calculating the benefits, it was assumed that the Halifax levee is not constructed. The results of the analysis are presented in Table 10-7. Because of commercial confidentiality, data was not available for the option to sell overseas, but the indications from Pino

Giandomenico was that it would be more cost effective than selling to the Townsville or Cairns market.

Table 10-7 BCR Analysis of Dredging Options

Dredge Scenario	Upfront Funding Required Assuming all Sand Sold ($)	BCR
Dredge to stockpile and sell locally	14,000,000	0.2
Dredge to stockpile and sell to Cairns and Townsville	43,000,000	0.5
Dredge and sell to overseas market	Less than second scenario	N/A

10.3.2.2 Impacts of Dredging on Estuarine Ecology

Potential impacts of channel dredging on the estuarine ecology of the Herbert 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. Hydraulic 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 Herbert River experiences high concentrations of suspended solids, particularly during ebbing tides, floods and under certain wind/wave conditions. 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 Herbert River is unknown. However, any seagrasses present already experience periods of elevated turbidity in response to wind/wave action and flood-related turbid river plumes.

10.3.2.3 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.

10.3.2.4 Approval Requirements

Preliminary assessment indicates that the following approvals/permits could be required for the dredging works:

1. For works in tidal waters, approved plans under Section 86 of the Harbours Act 1955;
2. For dredging, ERA 19a approval under the Environment Protection Act 1994;
3. If any material is to be placed on land, a Marine Land Dredging By-law 1987 permit;
4. 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 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.

10.3.2.5 Summary

The SAG decided against proceeding with further analysis of this measure because of the significant upfront costs of the dredging, potentially significant maintenance dredging costs and the hydraulic impacts in the lower areas of the floodplain.

The SAG's decision is that river dredging should not be included in the Floodplain Management Plan.

10.3.3 Washaway Diversion Channel

A diversion channel from the Washaway upstream of Halifax across into Victoria River was proposed. The intent of the diversion channel was to alleviate the impact of levee construction and long-term sediment deposition on flood levels in the lower reaches by diverting flow from the Herbert River. The path of the diversion channel is shown in Drawing 10-4, which also shows the change in the 100 year ARI peak flood level. The channel was modelled as an 80 m wide maintained overland flow path rather than a constructed channel, ie, a flow path of maintained grass was assumed to be cleared through the sugar cane. This would provide a channel of significantly increased hydraulic efficiency through the cane to Victoria River. The Herbert River bank was not lowered which means that flow into the channel would commence under the same flood conditions in the Herbert River as it currently does.

Drawing 10-4 shows that the diversion channel only has limited impact on the peak flood levels with small decreases to the north of the channel and small increases to the south. Following review of this analysis, the SAG elected not to proceed to the calculation of a preliminary BCR.

The SAG's decision is that a detailed analysis should not be undertaken and that this measure should not be included in the Floodplain Management Plan.

10.3.4 Ingham Levee

10.3.4.1 Option 1

A ring levee around Ingham was proposed as a means for protecting the main urban area of Ingham from floodwaters. The levee was set above the 100 year ARI flood level and tested using the 100 year ARI design flood. The location of the levee is shown in Drawing 10-5 along with the change in peak 100 year ARI flood level. The significant increases in flood level shown in this drawing at the peak of the 100 year ARI were expected given that several significant overland flow paths were blocked by the levee.

However, the intent of running a 100 year ARI flood was also to investigate if there was a stage earlier in the 100 year flood that the impacts were not widespread. Drawing 10-6 shows that during the rise of the 100 year flood there are significant increases in flood level, but that they are localised around Ingham: this is approximately equivalent to a 5 year ARI flood event. This indicates that a levee set at the 5 year ARI level would provide some reduction in damages to Ingham but at the expense of the properties immediately outside the levee. The SAG elected not to proceed with the calculation of a preliminary BCR, but agreed to an investigation into a multiple ring levee option

10.3.4.2 Option 2

The second levee option for Ingham considered a series of ring levees protecting Ingham, but maintaining the existing major floodways through Ingham in an attempt to reduce the impact of the levee system on the wider floodplain. Although the major floodways were not blocked, they were partially restricted by the levees and hence the levee system did not maintain the full conveyance capacity of the flow paths through Ingham. The levees were set above the 100 year ARI flood level and tested using the 100 year ARI design flood. The location of the levees are shown in Drawing 10-7 along with the change in peak 100 year ARI flood level. The system of levees was successful in limiting any significant increase in 100 year ARI peak flood to the proximity of the levees. Therefore, the SAG elected to proceed with the calculation of a BCR for this levee system.

A summary of the BCR calculations for the Ingham levees are given in Table 10-8. For the purposes of this preliminary calculation, all levees were assumed to be of earth construction. In reality, space limitations may require that sections of the levee be concrete. Levee height, and hence the volume of earthworks, were based on ground elevations in the DEM. Costs were calculated for levees to provide 5 year, 20 year and 100 year ARI immunity. A 300 mm freeboard was assumed.

The average height of the 100 year ARI levee would be about 2.7 m. Therefore, a relatively flat side slope would be required for maintenance purposes and the top width needs to be sufficient for vehicular access for maintenance purposes. The sensitivity of the analysis to top width and side slope was investigated.

The committee elected not to proceed with a detailed analysis of this levee option because of the low BCRs and the substantial construction costs.

Table 10-8 BCR Analysis of Ingham Option 2 Levee – SS=1:4 & W=3m

Levee Height	Average Annual Benefit	Total Benefit over 50 Years	Construction Costs	On-Going Costs over 50 Years	Total Cost	BCR
5 Year ARI	$374,000	$5,160,000	$10,000,000	$2,740,000	$12,740,000	0.4
20 Year ARI	$702,000	$9,700,000	$25,420,000	$2,740,000	$28,160,000	0.35
100 Year ARI	$769,000	$10,610,000	$35,150,000	$2,740,000	$37,890,000	0.28

SS=Side slope of levee (eg 1:4 is 1m vertical to 4 m horizontal); W is top width of levee

Table 10-9 BCR Analysis of Ingham Option 2 Levee – SS=1:5 & W=3m

Levee Height	Average Annual Benefit	Total Benefit over 50 Years	Construction Costs	On-Going Costs over 50 Years	Total Cost	BCR
5 Year ARI	$374,000	$5,160,000	$11,900,000	$2,740,000	$14,640,000	0.35
20 Year ARI	$702,000	$9,700,000	$30,535,000	$2,740,000	$33,275,000	0.29
100 Year ARI	$769,000	$10,610,000	$42,410,000	$2,740,000	$45,150,000	0.24

SS=Side slope of levee (eg 1:4 is 1m vertical to 4 m horizontal); W is top width of levee

Table 10-10 BCR Analysis of Ingham Option 2 Levee – SS=1:5 & W=6m

Levee Height	Average Annual Benefit	Total Benefit over 50 Years	Construction Costs	On-Going Costs over 50 Years	Total Cost	BCR
5 Year ARI	$374,000	$5,160,000	$14,170,000	$2,740,000	$16,910,000	0.31
20 Year ARI	$702,000	$9,700,000	$35,250,000	$2,740,000	$37,990,000	0.26
100 Year ARI	$769,000	$10,610,000	$48,220,000	$2,740,000	$50,960,000	0.21

SS=Side slope of levee (eg 1:4 is 1m vertical to 4 m horizontal); W is top width of levee

10.3.4.3 Summary

The SAG decided against proceeding with further analysis of this measure because of the significant hydraulic impacts, high upfront costs and the low BCR.

The SAG's decision is that these levees should not be included in the Floodplain Management Plan.

10.3.5 Small Township Levees

Ring or open levees were investigated for Trebonne, Toobanna, Blackrock, Cordelia, Macknade and Bemerside. Blackrock, Cordelia and Macknade were combined into one run and Macknade Trebonne, Toobanna into another run. The levees were set above the 100 year ARI flood level and tested using the 100 year ARI design flood. The locations of the levees are shown in Drawing 10-8 and Drawing 10-9 along with the change in peak 100 year ARI flood level. All levees except for the Cordelia levee would have only minor impacts on the peak flood levees outside of the area benefited by the levee. The Cordelia levee will increase flood levels in the Macknade area and decrease flood levels to the south of Cordelia.

A summary of the BCR calculations for the Trebonne levee is given in Table 10-11. The preliminary BCR of the Trebonne levee presented during the study indicated that the levee may be economically viable, but was cost prohibited and would only benefit a small section of the community. The preliminary analysis was based on the assumption that the habitable floor level was the ground level plus 0.5 m, which may have been overstating the BCR. The SAG decided that, although the project was not feasible at this stage, there was merit in refining the BCR in the event that funding becomes available in the future. Therefore, the HSC surveyors collected floor level and building type data for all buildings in Trebonne. The base case damages assessment was revised using the new building data.

For the purposes of this preliminary calculation, all levees were assumed to be of earth construction. In reality, space limitations may require that sections of the levee be concrete. Levee height, and hence the volume of earthworks were based on ground elevations in the DEM. These costs do not allow for additional drainage works. Costs were calculated for levels to provide 20 year and 100 year ARI immunity and in the low lying areas, a 5 year ARI analysis was also undertaken. A side slope of 1V:5H, a top width of 3 m and a 300 mm freeboard were assumed.

The BCRs and preliminary total costs (construction plus on-going) for the other small township levees are provided in Table 10-12 and Table 10-13 respectively. Unlike the Trebonne levee analysis, the analysis for these levees was based on the assumption that the habitable floor level was the ground level plus 0.5 m.

The SAG elected not to proceed with a detailed analysis of these small township levees because of the low BCRs and the substantial costs.

Table 10-11 BCR Analysis of Trebonne Levee

Levee Height	Average Annual Benefit	Total Benefit over 50 Years	Construction Costs	On-Going Costs over 50 Years	Total Cost	BCR
100 Year ARI	$102,000	$1,407,000	$2,359,000	$263,000	$2,622,000	0.54

Table 10-12 BCR Analysis for Small Township Levees

Name	BCR		
	5 Yr Immunity	20 Yr Immunity	100 Yr Immunity
Cordelia			
SS=5, W=6m	.03	.10	.13
SS=5, W=2m	.04	.13	.16
SS=2, W=2m	.26	.20	.06
Macknade			
SS=5, W=6m	.10	.17	.11
SS=5, W=2m	.11	.20	.13
SS=2, W=2m	.17	.34	.21
Bemerside			
SS=5, W=6m	-	.25	.08
SS=5, W=2m	-	.31	.10
SS=2, W=2m	-	.37	.18
Toobanna			
SS=5, W=6m	.22	.23	.19
SS=5, W=2m	.28	.27	.23
SS=2, W=2m	.55	.46	.41
Blackrock			
SS=5, W=6m	-	.14	.08
SS=5, W=2m	-	.17	.09
SS=2, W=2m	-	.27	.17

Table 10-13 Preliminary Total Costs (Construction + On-going) for Small Township Levees

Name	Costs ($)		
	5 Yr Immunity	20 Yr Immunity	100 Yr Immunity
Cordelia			
SS=5, W=6m	$1,780,000	$1,800,000	$1,830,000
SS=5, W=2m	$1,450,000	$1,470,000	$1,490,000
Macknade			
SS=5, W=6m	$1,600,000	$1,800,000	$2,200,000
SS=5, W=2m	$1,300,000	$1,500,000	$1,800,000
SS=2, W=2m	$850,000	$900,000	$1,100,000
Bemerside			
SS=5, W=6m	-	$320,000	$1,120,000
SS=5, W=2m	-	$250,000	$920,000
Toobanna			
SS=5, W=6m	$1,500,000	$1,900,000	$2,300,000
SS=5, W=2m	$1,200,000	$1,600,000	$1,900,000
SS=2, W=2m	$630,000	$920,000	$1,100,000
Blackrock			
SS=5, W=6m	-	$1,600,000	$3,200,000
SS=5, W=2m	-	$1,250,000	$2,660,000
SS=2, W=2m	-	$800,000	$1,500,000

Although these projects would bring considerable intangible benefits to the community, the capital costs are prohibitive in the context of the funds available through the State Government.

The SAG's decision is that these levees should not be included in the Floodplain Management Plan.

10.3.6 Levee Rationalisation

In recognition that the uncontrolled construction of levees over the years has resulted in some areas getting additional flood protection at the expense of other areas, the SAG proposed that a an investigation into a levee rationalisation scheme be undertaken, predominantly concentrating on the lower reaches. Two schemes were considered, 5 year ARI and 3 year ARI levee systems, to help overcome the apparent disparity along the river system.

10.3.6.1 Five Year ARI River Levee

A levee along both banks of the Herbert River from upstream of Ingham to below Halifax was considered in an attempt to achieve rationalisation of the existing levees along the river. The scenario was initially tested using a 5 year ARI design flood. The results from this analysis are presented in Drawing 10-10. The analysis showed that significant portions of the floodplain would be protected in a 5 year ARI flood, as would be expected, but that there would be significant increases in flood level in the river as a result of containing the flood to the river system. In the lower reaches of the river, the increases are up to about 3 m. This would require the construction of a substantial levee system and was not considered practical.

Because of the impracticalities of this measure, the SAG decided against proceeding undertaking a preliminary BCR.

10.3.6.2 Three Year ARI Levee in Lower Reaches

In light of the findings of the 5 Year ARI levee analysis, a levee on both banks of the river at approximately the 3 year ARI height from Macknade to the end of the sugar cane region downstream of Halifax was trialled on the hydraulic model. The alignment of the levee and the change in peak 100 year ARI flood levels are shown in Drawing 10-11. The levee increased flood levels on the Macknade side of the river and decreased flood levels on the Southern and Eastern side of the river. Given the detrimental impacts in the Macknade area, the SAG elected not to proceed to a preliminary BCR.

10.3.6.3 Summary

The SAG's decision is that these levee systems should not be included in the Floodplain Management Plan.

10.3.7 Removal of Ripple and Catherina Creek Floodgates

Some members of the Herbert River Floodplain community are concerned that the Ripple and Catherina Creek floodgates are negatively impacting on flooding and on the ecological system within these creeks and that removal of the floodgates would provide a flood benefit. The investigation into potential flood benefits is reported in this Section and environmental issues were discussed briefly in Section 8.

The Catherina Creek and Ripple Creek floodgate flaps were removed for this analysis rather than removing the entire structure (ie the levee and culverts were assumed to remain). This was done for the practical reason that if the HRIT elected to take some action, it would most likely remove only the flaps because of the costs associated with removing the entire structure.

The effects of removing the flaps in the 100 year ARI flood are presented in Drawing 10-12 to Drawing 10-14. Drawing 10-12 shows the impact of the removal at the peak of the 100 year ARI flood, Drawing 10-13 shows the impact 14 hours into the 100 year ARI event (approximately equivalent to a 5 year ARI event) and Drawing 10-14 is 22 hours into a 100 year ARI event (approximately equivalent to a 10 year ARI event). This analysis shows that the removal of the floodgates does not significantly impact on the peak 100 year ARI flood levels. However, the impact of the removal of the floodgates is more significant early in the flood (or in smaller flood events) as indicated in Drawing 10-12 and Drawing 10-13. The area of land that would be negatively impacted in terms of increased flood heights by the removal of the floodgate flaps is approximately equal to the area of land on which flood heights would be reduced.

The SAG elected not to proceed to the calculation of a BCR because the preliminary analysis indicated that there would not be a significant flood benefit in removing the floodgates.

The SAG's decision is that the removal of the levees should not be included in the Floodplain Management Plan

10.4 Detailed Analysis

The SAG selected the Kingsbury Creek Floodgate and the Halifax Levee for detailed analysis. The detailed analysis required that the measures be tested using all design floods; the preliminary analyses only investigated the impact of the measure on flood levels using one flood event. 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 BCR was refined using the full range of floods.

10.4.1 Kingsbury Creek Floodgate and Levee

Flood waters from the Herbert River back up through Kingsbury Creek and across into North Ingham in floods as small as a 5 year ARI and possibly smaller. A floodgate and levee at a 5 year ARI flood height at the mouth of Kingsbury Creek was proposed to reduce the nuisance flooding in this area of Ingham. This height was adopted without a freeboard for the detailed analysis. A freeboard is normally provided to allow for uncertainty in the modelling when the structure is protecting a community. In this case, it is a low level structure that is intended to reduce nuisance flooding and damages in smaller events. Providing a freeboard would effectively increase the level of protection to above a 5 year ARI event, but would also increase the impacts elsewhere in the floodplain. Therefore a freeboard was not adopted.

10.4.1.1 Impacts on Flood Behaviour

The impacts of the proposed floodgate on the peak 5 year, 20 year and 100 year ARI flood levels are shown in Drawing 10-15 to Drawing 10-17 respectively. Drawing 10-18 shows impact of the floodgate on velocities 48 hours into the 10 year ARI flood. An animation of velocity impacts was viewed to select a time during the flood that was typical of the velocity impacts.

In a 5 year ARI event, the floodgates and levee prevent water from backing up through Kingsbury Creek, flowing through North Ingham and then down through Gairloch to Mt Cordelia. This is shown in Drawing 10-15 by the pink shading (areas that would be flooded under existing conditions and would not be flooded if the measure is implemented) and the green shading (areas where the flood level has decreased). In the river in the vicinity of the floodgate there are increases in flood level of about 40 mm to 60 mm. In the Ripple Creek area, the increases in water level are typically in the range 30 mm to 80 mm.

In a 20 year ARI event, the floodgates and levee reduce, but do not stop, flood waters backing up through Kingsbury Creek, flowing through North Ingham and then down through Gairloch to Mt Cordelia. This is demonstrated in Drawing 10-16 as reduced flood levels in this region. The area over which there are reductions in flood levels extends down to Victoria Creek and the Mandam region, although the reductions being typically in the range 30 mm to 60 mm are not significant. In the Ripple Creek area, the increases in water level are typically in the range 30 mm to 40 mm. This is a smaller impact than in the 5 year ARI flood because water is able to overtop of the levee and so the increase in level is not as great, but the increases are occurring at a higher flood level.

The pattern of reductions and increases in flood level in the 100 year ARI event (Drawing 10-17) are consistent with the 20 year ARI event, but less extensive. Although the increases in flood level in the Ripple Creek area are not significant, the owners of the affected properties should be consulted before the scheme is adopted.

Drawing 10-18 shows the change in velocity at 48 hours into the 10 year ARI flood event. Generally, the levee does not significantly increase the velocity. The only area where there is an increase in velocity is in the caneland to the west of the proposed floodgate. This is occurring because the water level in the river is higher, resulting in an increase in the flow out of the river at this location, and the water level in the cane is lower. The flow across the bank in the reach immediately upstream of the John Row bridge increases by about 15% with the introduction of the levee. The increases in velocity are typically in the range 0.2 m/s to 0.6 m/s. The absolute velocities in the existing case model in the areas where the increases are predicted to occur are typically in the range 0.4 m/s to 0.6 m/s. These velocities would increase to 0.6 to 1.2 m/s. Increases of up to 0.6 m/s should not significantly damage sugar cane, except perhaps for some minor scouring. In a fallow paddock, the existing velocities would cause scouring and the predicted increases in velocity are likely to increase the scouring that would occur in these paddocks. Farming practices could be reviewed to ensure that fallow paddocks in this area are surrounded by cane to help to minimise scouring.

10.4.1.2 Benefit-Cost Analysis

The annual average damages used in the detailed analysis were calculated using all design floods as described in Section 6. A description of the cost basis was given in Section 10.3.1.6 and a breakdown of the levee costs is given in Table 10-14. Note that the maintenance and operational costs are totals for a 50 year period. The costs presented in Table 10-14 do not include scour protection for the levee and the estimate for the floodgate is approximate only. The costs do not allow for automation of the floodgate. The revised BCR is presented in Table 10-15.

Table 10-14 Cost Breakdown - Kingsbury Ck Floodgate

Item	Cost
Levee Construction	$ 100,000
Floodgate	$ 75,000
Landscaping	$ 10,000
Sub-Total 1	$ 185,000
Contingencies (25%)	$ 46,000
Sub-Total 2	$ 231,000
Consultation, Survey, Admin	$ 20,000
Engineering	$ 20,000
CONSTRUCTION TOTAL	**$ 271,000**
Mowing and Gardening	$ 5,000
Community Education	$ 55,000
MAINTENANCE TOTAL	**$ 60,000**
Levee Monitoring, Gate Operation	$ 5,000
OPERATION TOTAL	**$ 5,000**
TOTAL	**$ 336,000**

Table 10-15 BCR Analysis of Kingsbury Creek Floodgate

Average Annual Benefit	Total Benefit over 50 Years	Construction Costs	On-Going Costs over 50 Years	Total Cost	BCR
$128,000	$1,775,000	$271,000	$65,000	$336,000	5.3

10.4.1.3 Environmental Considerations

General Comments

The HRIT have advised that their current philosophy on floodgates dictates that they are only closed during flood events. At all other times they remain open. Responsibility for this operational procedure rests with the HRIT. This operational procedure helps, but does not fully alleviate, the environmental impacts of floodgates. The following list describes typical environmental impacts of floodgates.

- Floodgates can change the inundation patterns of land upstream of the gates or increase localised flooding due to additional ponding upstream of the gates. The additional head and resistance to flow from the floodgates can increase inundation periods upstream, which may result in a change of vegetation species. For example, melaleuca trees tolerate short periods of inundation however are unable to colonise and thrive when inundated for long periods. The resultant change in vegetation due to changed drainage patterns may also have commensurate effects on the fauna associated with that habitat.

- Floodgates can be a barrier to the movement of aquatic fauna, such as fish, frogs and other mobile benthic species. Fish which require large habitat area and/or varying conditions for breeding and growing may not be able to be supported by the waterway following the introduction of the flood gates.

- If significant pollutant loads are entering the waterway upstream of the floodgates, then their introduction could reduce flushing of the waterway and cause water quality degradation upstream of the floodgates. Water quality is a key aspect of the environment, as it has a major influence on biological processes and ecological habitat provision. Waters with high pollutant concentrations may remain standing upstream of the floodgates for extended periods, leading to stagnation, elevated pollutant concentrations, algal blooms and potentially even fish kills. Various aquatic species may be sensitive to water quality degradation, thus preventing their survival if the quality of upstream waters decreases.
- Lower velocities in upstream waterway areas may result as waters pond upstream of the floodgates. This has implications for siltation in the waterway, as slower velocities allow suspended sediment particles to settle. This siltation could smother sessile aquatic organisms and plants as well as causing maintenance issues for the flood gates.
- Drawing 10-15 shows that there is a significant area of predominantly sugar cane land to the north-east of Ingham that would once have been inundated from the Herbert River in a 5 year ARI event, but with the construction of the floodgate, this will no longer occur. While this potentially reduces the inundation damage to the cane in this area, it will also mean that the area will not be replenished with silt as often as it is currently.

It will be necessary to determine the referral agencies for such a construction and determine if any of these approvals trigger a requirement for an environmental assessment. The cost of an environmental assessment is not included in the economic analysis.

10.4.1.4 *Intangible Benefits*

The floodgates will reduce the frequency of minor flooding and provide minor reductions in flood level in larger events. Residents will be required to clean up after floods less frequently. Therefore, the floodgates will lower the health, social, and psychological trauma associated with flooding in smaller events.

Another benefit the floodgates may bring relates to the Queensland Government Rail Line. Residents in Dalrymple Street are concerned that flooding has been worsened over the years by a perceived raising of this line associated with re-ballasting and by a reduction in drainage capacity through the line as a result of the reconstruction of the drainage structures a number of years ago (this was not investigated as part of this study). The reduction in frequency of flooding and flood levels that the floodgates would bring may reduce the anxiety levels of the residents in this area.

10.4.1.5 *Summary*

The hydraulic analysis indicates the proposed floodgate will provide benefit to the northern parts of Ingham with only minor disbenefits elsewhere. The economic analysis and intangible benefits further supports the benefits of the floodgate.

The SAG's recommendation is that the Kingsbury Creek Floodgate be included in the Floodplain Management Plan.

10.4.2 Halifax Levee

Halifax is sited on the bank of the Herbert River in the lower reaches. The town has a long history of flooding. However, local residents are concerned that, in recent times, flood levels are now higher in Halifax than they were say 15 or 20 years ago in a flood of similar magnitude further up the river. Although this study has not specifically investigated this claim, the extent of river bank levees constructed upstream of Halifax and the to a lesser extent the long term deposition of sediment in the lower reaches would support such a claim. To alleviate the flooding in Halifax, Maunsell McIntyre (1999) proposed the construction of a levee around Halifax. The levee did not proceed at the time, but was investigated as part of this study.

It is proposed that the levee be constructed at the 100 year ARI flood level plus a 300 mm freeboard along the alignment shown in Drawing 10-19. Along some of the proposed levee alignment north of Farrell Drive, there is an existing levee that currently provides immunity in a 100 year ARI flood. There are also low sections along this part of the levee that do not provide this level of immunity and would require augmenting. The levee is shown crossing Musgrave Street at the southern end of the main levee and Farrell Drive. These gaps could be sand-bagged during floods or the roads raised. The SAG has indicated a preference for the latter. The levee as proposed would also cross the tramline thereby presenting a similar problem. Raising the tramline would be relatively expensive, as it would need to be done over a long distance to satisfy grade requirements for the tram. Sandbags or a gate could be used at this location. An alternative would be to stop the levee before it crosses the line, but this would slightly reduce the effectiveness of the levee, especially for those houses in the south-eastern parts of Halifax.

The southern levee is shown as open in Drawing 10-19. An alternative would be to construct a fully enclosed levee assuming access difficulties could be resolved and the residents agree. This would provide additional protection without a significant increase in cost and would eliminate the build-up of mud around the houses during most floods. This would not create any significant additional flood impacts.

10.4.2.1 *Impacts on Flood Behaviour*

The impact of the Halifax Levee on the peak 5 year, 20 year and 100 year ARI flood levels are shown in Drawing 10-19 to Drawing 10-21 respectively. An animation of velocity impacts was viewed to select a time during the flood that was typical of the velocity impacts. The typical impact of the levee on velocities is shown in Drawing 10-22.

The impact of the levee on peak flood height is consistent across the range of floods modelled. There are small increases of about 35 mm in and across the river to the west of the levee and decreases of about 70 mm to the east of the levee. Although these increases in flood level are not significant, the owners of the affected properties should be consulted before the scheme is adopted.

As shown in Drawing 10-22, the levees will not significantly increase velocities across the floodplain. As would be expected, the velocities around the alignment of the levee will increase, and across some of the roads near the southern levee there are small increases of typically 0.2 to 0.3 m/s. The impact of the increase in velocity along the alignment of the levee may lead to bank stability problems. This issue should be considered in future investigation and design work.

10.4.2.2 Benefit-Cost Analysis

The annual average damages used in the detailed analysis was calculated using all design floods as described in Section 6. A description of the cost basis was given in Section 10.3.1.6 and a breakdown of the levee costs is given in Table 10-16. The cost to raise the road and tramline is not included in these costs. Maunsell McIntyre (1999) undertook a preliminary costing for the augmentation of the existing levee system. Maunsell McIntyre assumed a typical levee section comprising a 2m top width with 1V:2H side slope on the river side and 1V:6H side slope on the town side to give a total construction cost of $182,000. WBM calculated a construction cost of $685,000 using levels in the DEM and assuming a 2 m top width with 1V:2H side slope on both sides of the levee. Some of the difference is a result of higher unit rates adopted by WBM, but the major contributing factor is a significant difference in the earthworks quantities. Maunsell McIntyre quantities are likely to be more accurate because they are based on a long section traverse of the existing levee, rather than the elevation in the DEM. Therefore, two BCRs were calculated, one using the Maunsell McIntyre construction cost in combination with WBM on-going costs and the other using all WBM costs.

The maintenance and operational costs are totals for a 50 year period. As shown in Table 10-16, $179,000 has been allowed for mowing and gardening. A considerable portion of the levee will be behind private properties in currently vegetated sections of the river bank. It is likely that rather than mowing the levee, it would be left to become heavily vegetated. If this is the case, the allowance for mowing could be considerably reduced. The BCR are presented in Table 10-17.

Table 10-16 Levee Construction Costs, Halifax Levee

	Maunsell McIntyre/WBM	WBM
Levee Construction	$ 85,800	$ 377,000
Landscaping	$ 11,200	$ 112,000
Sub-Total 1	$ 97,000	$ 489,000
Contingencies (25%)	$ 24,000	$ 122,000
Sub-Total 2	$ 121,000	$ 611,000
Consultation, Survey, Resumptions, Admin	$ 20,000	$ 20,000
Engineering & Geotech	$ 35,000	$ 35,000
CONSTRUCTION TOTAL	**$ 176,000**	**$ 666,000**
Mowing and Gardening	$ 179,000	$ 179,000
Community Education	$ 55,000	$ 55,000
MAINTENANCE TOTAL	**$ 234,000**	**$ 234,000**
Levee Monitoring incl. Annual Survey	$ 112,000	$ 112,000
Sand Bags for Road Crossings	$ 20,000	$ 20,000
OPERATION TOTAL	**$ 132,000**	**$ 132,000**
TOTAL	**$ 542,000**	**$ 1,032,000**

Table 10-17 BCR Analysis of Halifax Levee

Cost Basis	Average Annual Benefit	Total Benefit over 50 Years	Construction Costs	On-Going Costs over 50 Years	Total Cost	BCR
Maunsell McIntyre/WBM	$46,000	$634,400	$182,000	$345,000	$527,000	1.2
WBM	$46,000	$634,400	$685,000	$345,000	$1,030,000	0.6

10.4.2.3 Environmental Considerations

As was noted in Section 10.4.2.1, the proposed levee will potentially increase velocities along the river bank which may lead to bank stability problems. No other environmental considerations were identified.

It will be necessary to determine the referral agencies for such a construction and determine if any of these approvals trigger a requirement for an environmental assessment. The cost of an environmental assessment is not included in the economic analysis.

10.4.2.4 Intangible Benefits

Levees lower the health, social, and psychological trauma associated with flooding. In addition, it is less likely that people residing in dwellings inside the levee require evacuation and they may not need to remove or raise their possessions. All of these factors reduce the impact of flooding. The levee will also reduce the level of stress within the community relating to the following issues:

1. an apparent increase in flood levels over the years for similar size floods – although this study has not attempted to quantify this, it is likely given the construction of levees upstream and the build up in sediment in the lower reaches of the river;

2. an equity in flood protection across the town – the northern part of town is currently offered some protection by river bank levees.

10.4.2.5 Summary

The hydraulic analysis indicates the proposed levee will provide benefit to Halifax with only minor disbenefits elsewhere. The economic analysis and the intangible benefits further supports the benefits of the levee.

The SAG's recommendation is that the Halifax Levee be included in the Floodplain Management Plan.

10.5 Summary

An investigation into a wide range of flood modification measures was undertaken. The analysis ranged from a desktop review through to a detailed hydraulic and economic analysis with environmental consideration documented. Of the measures considered, the SAG has recommended that the Kingsbury Creek Floodgate and Halifax Levee be included in the Floodplain Management

Plan. This requires an hydraulic and economic analysis of the two measures in combination. This analysis is presented in Section 13.

Both measures indicated some minor flooding impacts that will require resolution with affected property owners.

11 PROPERTY MODIFICATION

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.

11.1 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.

11.1.1 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.

11.1.2 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 Herbert River Floodplain.

11.1.2.1 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 as 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.

11.1.2.2 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 11-1.

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 11.1.2.1.

The consideration of depth and velocity is based on curves presented in the DLWC (2001) and shown in Figure 11-1 and Figure 11-2. 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 11-1 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.

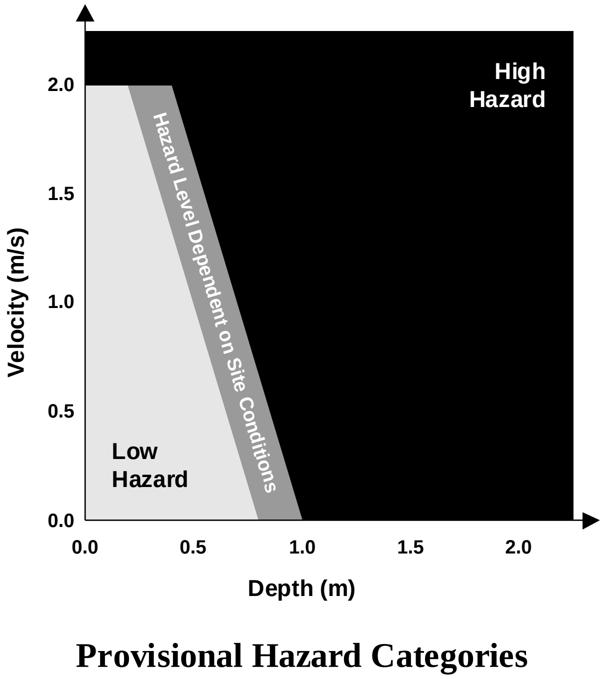


Figure 11-1 NSW Floodplain Management Manual Hazard Categories

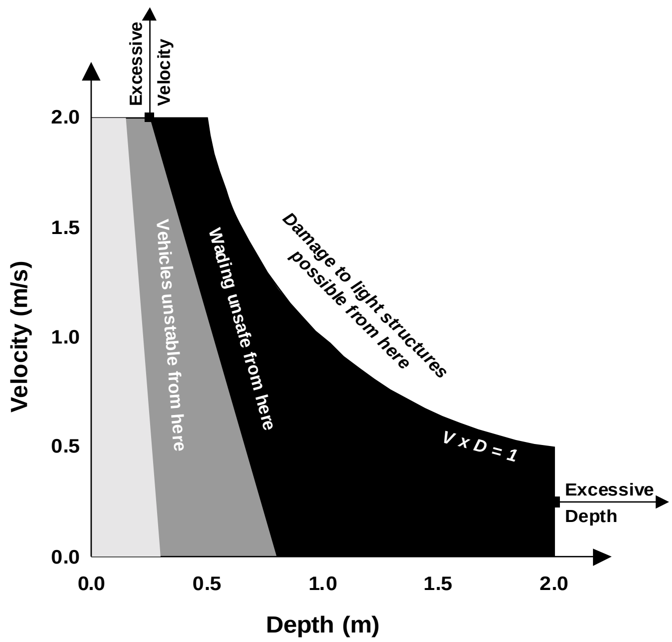


Figure 11-2 Velocity and Depth Relationships

11.1.3 Recommended Approach

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

- duration of flooding is universally long (in the order of days) across the floodplain;
- warning times are long;

- rates of floodwater rise are slow; and
- flood awareness is generally high and does not vary significantly across the floodplain.

The above four parameters are not significantly variable to warrant specific treatment and are therefore not used to define variations in the flood hazard, but will be built into the 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 11-2 be adopted for the Herbert River floodplain and that they be defined in accordance with the criteria in Figure 11-3 which combines Figure 11-1 and Figure 11-2.

Table 11-2 Flood Hazard Categories for Herbert Floodplain

Hazard Category	Characteristics
Low	adult can wade
High – Wading Unsafe	wading not possible, risk of drowning
High – Depth	damage only to building contents, large trucks able to evacuate
High – Floodway	truck evacuation not possible, structural damage to light framed houses, high risk to life
Extreme	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) that is presented in Section 11.4.

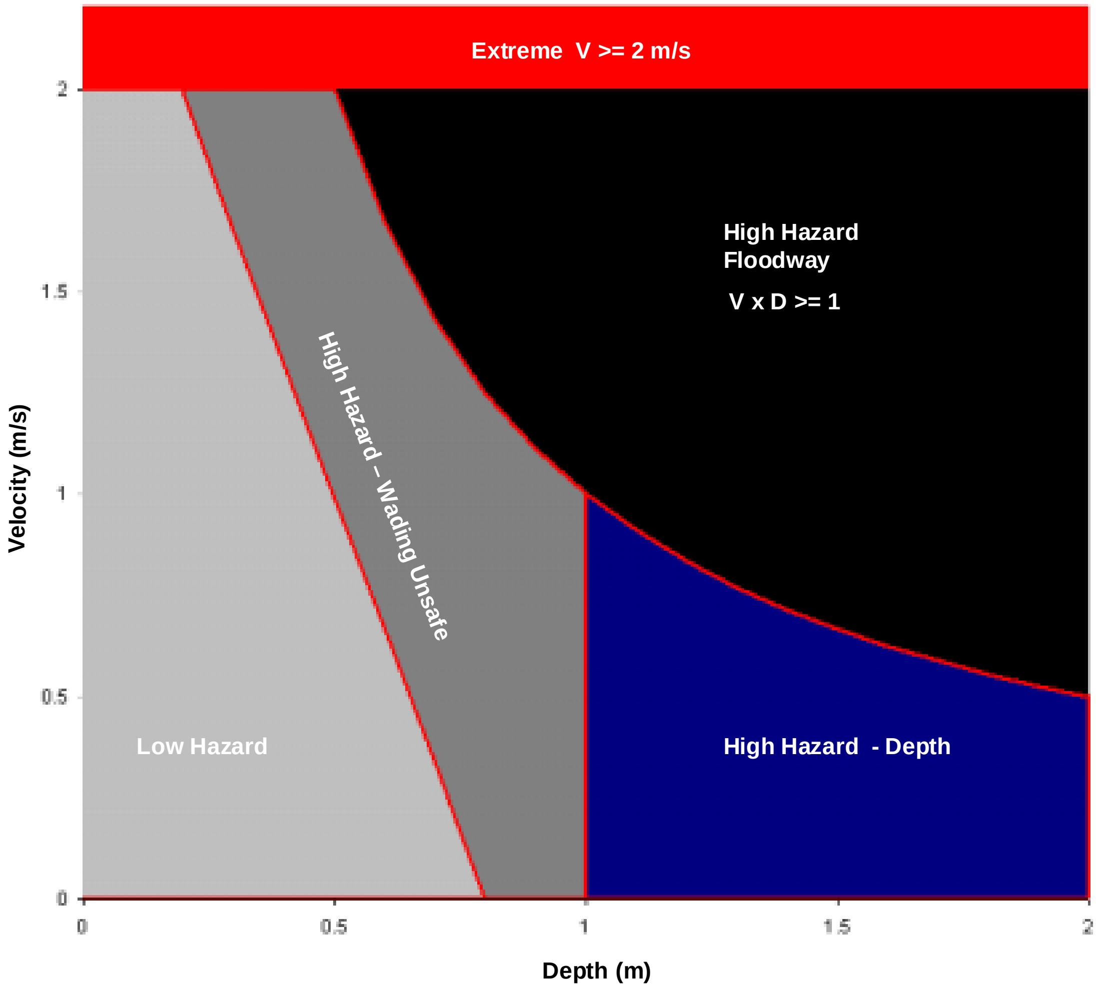


Figure 11-3 Definition of Recommended Flood Hazard Categories

11.1.4 Flood Hazard Maps

Using the Flood Hazard categorisation described in the previous section, flood hazard was determined for the entire floodplain using the 100 year ARI design flood and is presented in Drawing 11-1. Also shown on this drawing are properties located within the floodway categories. These are referred to in Section 11.2.

11.1.5 Flood Access to Major Townships

Access to townships during flooding is an important aspect of emergency management. An assessment of major routes was undertaken to determine the size of the flood in which the route becomes untrafficable. The criteria adopted for the assessment was that used by the Queensland Department of Main Roads and specified for National Highways, ie, a road is deemed not trafficable when the total head (H) exceeds 0.3 m where the total head is defined by Equation 11.1.

$$H = d + \frac{v^2}{2g} \qquad \text{Equation 11.1}$$

where

d is the depth of water over the road (m)

v is the velocity (m/s)

g is the gravitational acceleration (m/s^2)

To undertake this assessment in detail would require survey of the road network and assessment at each location, which is beyond the scope of the study. However, an approximation has been undertaken which is sufficient for the purposes of emergency management planning. Experience has shown that when H is 0.3 m, d is normally in the range 0.2 m to 0.22 m where there is crossflow. Therefore, flood depths were assessed based on the road grade information in the DEM to determine the size of flood in which the depth exceeded 0.2 m. The results of this assessment over the study area are presented in Table 11-3. It was found that all routes except the Forrest Beach Road would be untrafficable in a 5 year ARI event, which was the smallest flood modelled for this study. Some of these routes may be cut in smaller flood events.

Greater detail of the route trafficability analysis is presented in the emergency management maps prepared as part of this study. In these maps, the sections of all major access roads that are not trafficable are identified for each of the design floods modelled.

Table 11-3 Approximate Route Immunity

Route Sector	Flood event for which d > 0.2 m^{+}
Timrith to Trebonne	5 year ARI
Trebonne to Ingham	5 year ARI
South of Toobanna	5 year ARI
Toobanna to Ingham	5 year ARI
Ingham to Forrest Beach	Just cut in 10 year ARI
Ingham to Cordelia	5 year ARI
Cordelia to Halifax	5 year ARI
Ingham to Bemerside	5 year ARI
Bemerside to Halifax	5 year ARI

$^{+}$ A 5 year ARI flood was the smallest flood modelled for this study. Some of these routes may be cut in smaller flood events.

11.2 Voluntary House Purchase

11.2.1 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. Given that the voluntary purchase of houses is specifically undertaken to protect the lives of the occupants, a benefit-cost analysis to assess this measure based solely on economic ratio will not convey the true benefits. However, a benefit-cost ratio is still determined to indicate the possible tangible economic benefits or otherwise.

Buildings that are currently located within the High Hazard - Floodway category are shown in Drawing 11-1. Many of the properties shown in south-west Ingham as being in the High Hazard Floodway category are on the fringe of the category and would probably not be considered for house purchase because they can still be accessed.

Assessments undertaken in relation to the purchasing of houses were limited to the urban areas because in rural areas the houses are normally associated with a farm and purchase of the property may affect the operation of the farm. However, this should not preclude the rural areas from the scheme. The reduction in damages achieved by purchasing a house is determined using existing property surveys and descriptions, and stage damage relationships as discussed in Section 6.

A basic procedure for calculating reduction in flood damage is similar to that used for the assessment of structural measures and is as follows:

- calculate the existing annual average damages;
- select houses located in the high floodway hazard zone;
- calculate the annual average damages following the theoretical purchasing and removal of those houses;
- estimate the cost of purchasing and removing the houses; and
- determine a monetary benefit-cost ratio for each scenario.

Calculations were performed for the removal of residential properties in Ingham and Cordelia, the only locations at which houses were in the High Hazard - Floodway category; for this analysis, those noted earlier as being on the fringe of the floodway category were included in the analysis. It was assumed that once purchased, these houses were removed thus removing all damages that were previously associated with these houses.

11.2.2 Benefits

The monetary benefits of house purchase arise from the reduction in the level of flood damage incurred through a reduction in house damage and a reduction in property damage (e.g. garden damage).

In addition, there are a number of health, social, and psychological benefits as people are spared the trauma associated with having their homes inundated by flood waters. These are not easily quantifiable in monetary terms, and are not included in the benefit-cost calculations.

11.2.3 Costs and Impacts

Local real estate agent Lawrence Molacahino advised that properties in Ingham in flood prone areas would typically be valued in the range $80 000 to $90 000 and at Cordelia would be in the range $50 000 to $60 000. The upper end of each range was adopted to provide a conservative estimate of the cost of the scheme, but also the purchase price under these circumstances might be above market value to allow for moving costs and inconvenience. Ninety-nine properties in Ingham and thirteen in Cordelia are located in the floodways.

One impact of house purchase 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 may not have a significant impact.

It is understood that HSC has previously considered the introduction of a flood prone land category into its town plan, but was not implemented as a result of concerns from residents in these areas relating to property values and a reduction in the saleability of a property. The recommendation of this study that a High Hazard – Floodway category be introduced would likely be the subject of similar protests. However, these concerns may be allayed if it is implemented in conjunction with a voluntary house purchase program.

11.2.4 Monetary Benefit-Cost Ratios

Table 11-4 summarises the monetary benefit-cost ratios (BCR) of the house purchase. A contributing factor to the low BCR of 0.07 is that 34 of the houses in Ingham and five in Cordelia are <u>not</u> inundated in a 100 year ARI flood, ie, the removal of these houses does not significantly reduce the flood damage because they are inundated in only rare events.

Table 11-4 BCR - House Purchase

Total Benefit	$651,500
Number of Properties Purchased	112 (99 in Ingham) (13 in Cordelia)
Total Cost	$9 690 000
Monetary BCR	0.07

It is reiterated that this scheme is intended to save life and limb and the calculation of the BCR is intended only to give an indication of the cost of the scheme. The true BCR is potentially higher due to the benefits of savings lives and reducing injuries, factors which are not able to be quantified in monetary terms.

11.2.5 Intangible Benefits

A major benefit of voluntary house purchase is that it reduces the number of people that require evacuating and reduces the risk to the lives of both the residents and the emergency service workers. The health problems and psychological trauma experienced by residents is substantially reduced as a result of these factors.

11.2.6 Summary

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

11.3 Voluntary House Raising

11.3.1 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 the urban areas of Ingham, Trebonne, Toobanna, Blackrock, Cordelia, Bemerside, Macknade and Halifax. However, rural buildings may be considered on a case-by-case basis. The reduction in damages achieved by raising a building is determined using an estimate of the building floor level, and stage damage relationships as discussed in Section 6.

A preliminary benefit-cost analysis of a voluntary house raising program was presented to the SAG during the course of the study. For the preliminary analysis, it was assumed that all houses could be raised and that the floor level was either the roof level less 3 m or, where the roof level was not available, the ground level plus 0.5 m; roof levels were only available in Ingham and Halifax. The preliminary analysis indicated that the measure may be viable and so the SAG elected to refine the analysis by establishing which houses could be raised and improving the estimate of the habitable floor level at those houses for which the roof level was not available. This additional data was obtained by HSC surveyors. The estimate of floor level was a visual estimate only. Surveyed floor level information obtained for the revised Trebonne levee analysis was also included in the analysis.

The base case damages were recalculated using the revised building data.

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

- calculate the existing annual average damages;
- define a criteria for selecting those buildings to be considered for house raising - as outlined in Section 11.3.2, three different criteria were defined;
- calculate the annual average 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.

11.3.2 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 society. Therefore, raising of all 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 11-5.

Table 11-5 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 were not considered in this assessment.

In the preliminary analysis, the average cost of raising a house in Ingham was assumed to be $15 000 plus 25% contingencies. It was subsequently found that this price did not allow for electrical or plumbing work. Therefore the average price has been increased to $17,000 for this analysis.

11.3.3 Benefits

The monetary benefits of house raising arise from the reduction in the level of flood damage incurred by the town. By reducing the extent of flood damage, monetary savings can be made by reducing house damage.

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.

11.3.4 Costs and Impacts

The average cost of raising a house in Ingham as provided by two Townsville-based house-raising companies is $17 000. Contingencies of 25% were added to the average prices.

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.

11.3.5 Monetary Benefit-Cost Ratios

The results from the benefit-cost analysis for the house-raising options are presented in Table 11-6. 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. The analysis indicates that option A provides the best economic return, but in only raising houses that have existing floor levels of twenty 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, but by applying funding arrangements as discussed in Section 11.3.6. 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.

Table 11-6 BCR - House Raising

	Option A	Option B	Option C
Total Benefit	$2,460,000	$2,830,000	$2,860,000
Total Number Buildings Inundated in study area	611	736	908
Number of Houses Raised	282	406	503
Total Cost	$5,992,000	$8,627,000	$10,690,000
BCR	0.41	0.33	0.27

11.3.6 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.

In WBM (2002), several assessments were undertaken using different proposed funding arrangements including the “Sliding Rule” and “Band Rule”. The “Band Rule” funding arrangement as explained in Table 11-7 was adopted.

Table 11-7 Example of Funding Arrangement used in NSW for House Raising

Band	First $10,000 of House Raising Cost			Remainder of House Raising Cost		
	Council	DLWC[4]	Owner	Council	DLWC	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)

[4] DLWC - NSW Department of Land and Water Conservation

Funding for this scheme would be available through the Regional Flood Mitigation Program. This program would contribute 2/3 of the funds. Given the limited funds available each year in this fund and the practicalities of raising houses, it is recommended that each year an application is made to the fund on the basis of raising approximately 20 to 30 houses in one year over a ten year period assuming there would be about a 50% take up rate. Discussions with the Regional Flood Mitigation Program manager has indicated that such an application would have been successful last year.

11.3.7 Intangible Benefits

In addition to the reduction in monetary damages, a major benefit of voluntary house raising is that it reduces the number of people that require evacuating. Also, it is less likely that residents need to remove their possessions from the house due to the higher flood immunity. The health problems and psychological trauma experienced by residents is reduced as a result of these factors.

11.3.8 Summary

The SAG's recommendation is that Option C should be included in a Floodplain Management Plan with the 1/3 local contribution being fully funded by the property owner.

11.4 Development Control

11.4.1 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 Herbert 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 11-4 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.

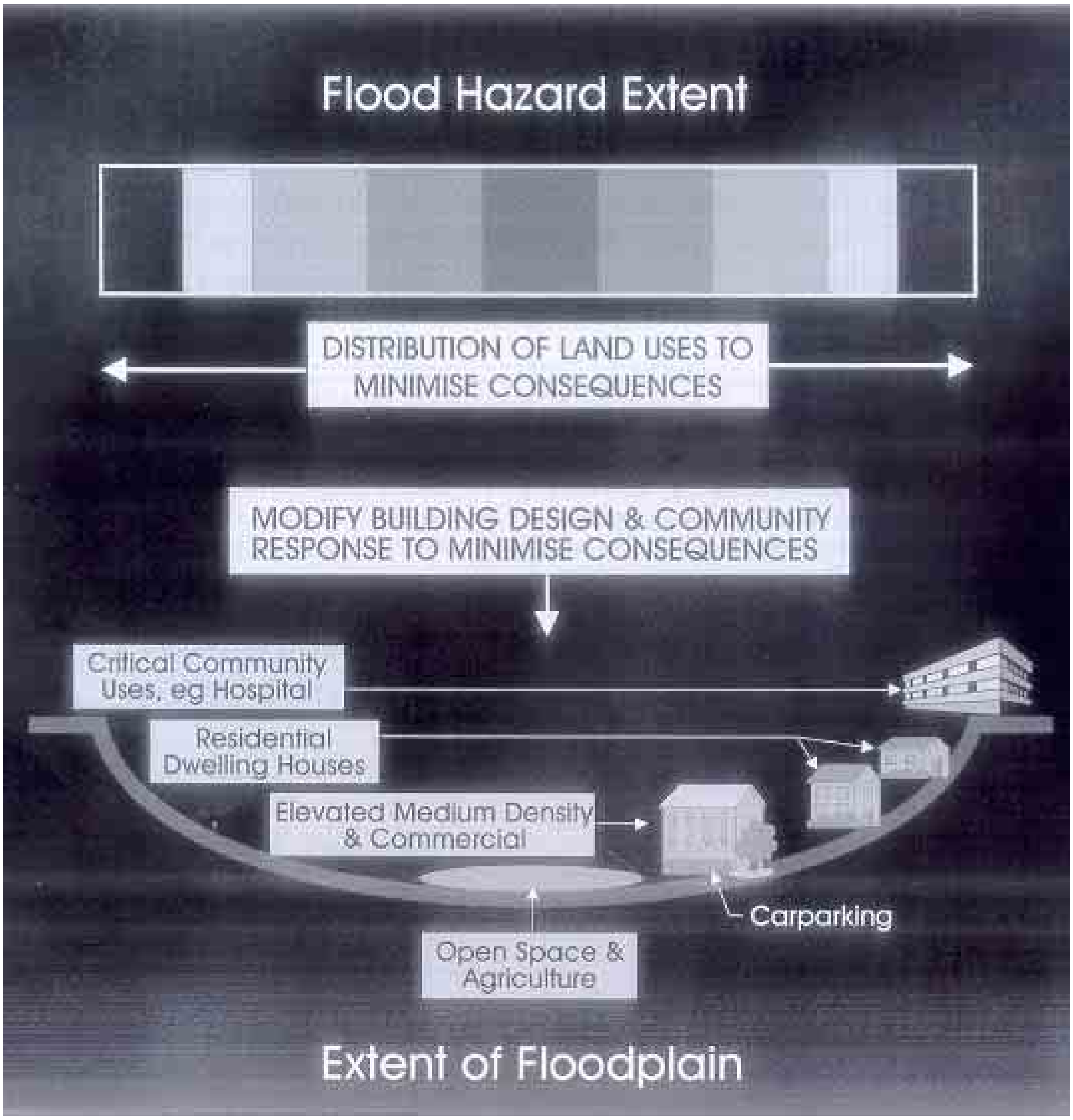


Figure 11-4 Flood Hazard Extent – NSW Floodplain Management Manual (DLWC, 2001)

11.4.2 Current Approach in Hinchinbrook Shire

11.4.3 Flood Policy

Currently, the Hinchinbrook Shire Council has a Planning Approach to floodplain management which is based on the Traditional Approach. A single Flood Planning Level (FPL) of the 1967 flood is used. All development control with flood prone land is related to this FPL. No formal consideration is given to floods larger than the 1967 flood or the associated hazards. HSC is currently revising its planning scheme and it is recommended that this opportunity be taken to incorporate floodplain risk management into the scheme.

11.4.4 Review of Approaches

The following sections provide a summary of the various approaches with a recommended approach outlined in Section 11.4.5. Issues which 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 completed at the commencement of the process using methods approved by the SAG.
- Herbert River 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;
  - there has been uncontrolled development of levees that has altered flooding characteristics resulting in worsening of flooding in some areas of the floodplain;
  - 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.

11.4.4.1 Traditional Approach

The Traditional Approach to floodplain planning has been described and reviewed in Section 11.4.1. 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 the NSW Floodplain Management Manual (DLWC, 2001). WBM Oceanics Australia recommends that this approach is not adopted for planning in Hinchinbrook Shire.

11.4.4.2 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 11-4. 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 11-5 (Bewsher and Grech, 1997). A number of plans showing flood hazard, landuse and flood level information accompanies 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.

11.4.4.3 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 1 in 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.

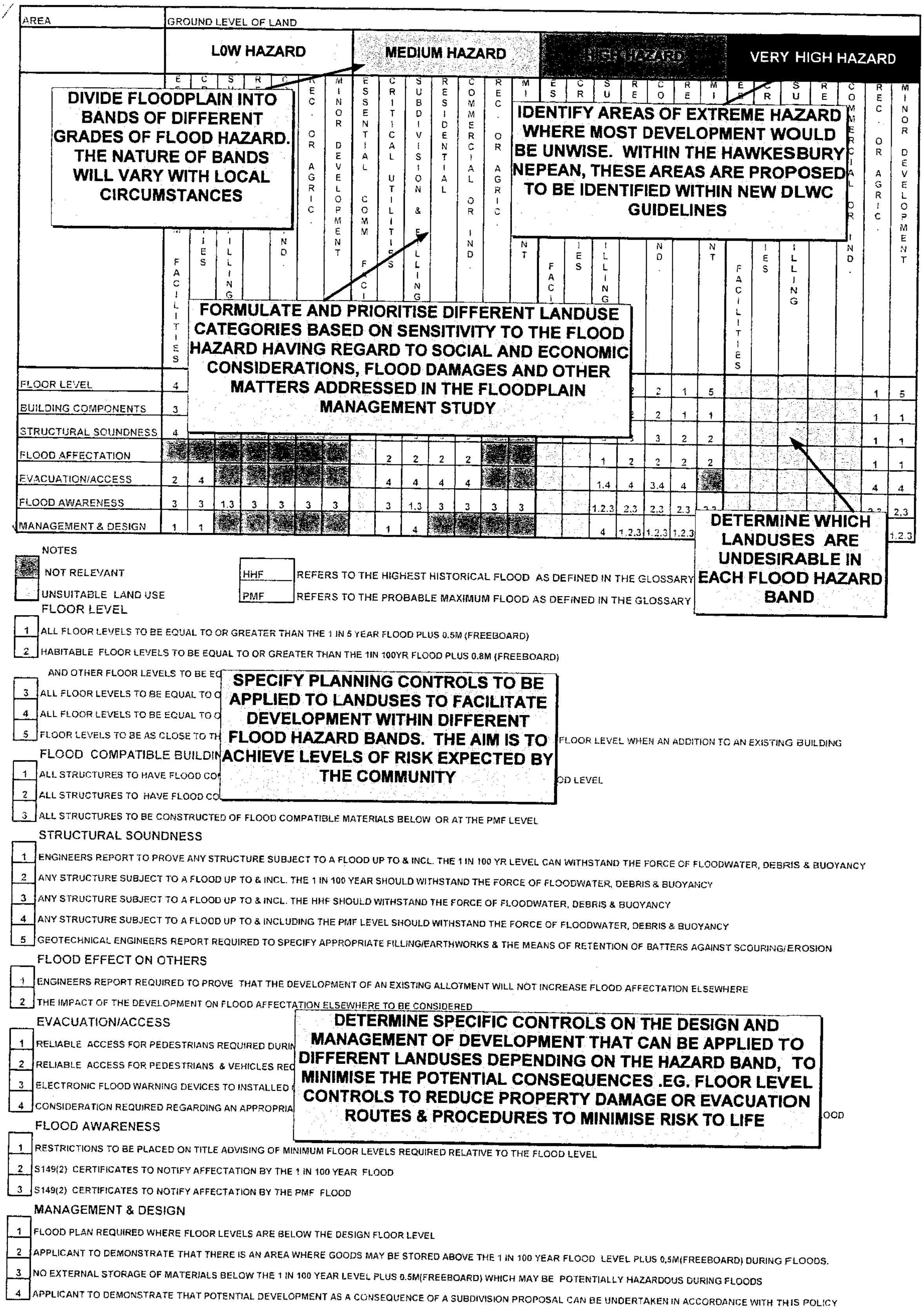


Figure 11-5 Planning Matrix

11.4.5 Recommended Approach

HSC 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 HSC officers, 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 be incorporated into the planning scheme.

11.4.6 Development of HSC Planning Matrices

Example planning matrices are contained within Figure 11-6, Figure 11-7 and Figure 11-8 for each of the discrete land use categories. These matrices would need to be adapted to the particular requirements of HSC.

FLOOD PLANNING MATRIX (Version 2)

TABLE 1: RESIDENTIAL, COMMERCIAL AND INDUSTRIAL DEVELOPMENT WITHIN AN URBAN AREA

Controls	Development / Building Type	No Hazard[1]	Flood Hazard Category: Low Hazard	High Depth Hazard	High Floodway Hazard
Land Use Suitability & Fill Level	Existing Lot - including infill subdivision	N/A	SF1	SF1	
	(this line not used)				
	Subdivision - en globo	N/A	SF2	SF2	
	Emergency Services Site (Hospitals, etc.)	N/A	SF3a		
	Other Community Service Building (School, etc.)	N/A	SF3b		
Floor Level	New Habitable Building	N/A	FL2c	FL2c	
	New Commercial or Industrial Building	N/A	FL2a	FL2a	
	New Emergency Service Building (Hospitals, etc.)	FL3a	FL3a		
	New Other Community Service Building (School, etc.)	FL3b	FL3b		
	New Ancillary Building (eg shed, carport)	N/A	FL1	FL1	
	Building Extension	N/A	FL4a	FL4b	
	(this line not used)				
Building Components		N/A	BC1	BC1	
Structural Soundness	Ancillary Building (eg. shed, carport)	N/A	SS1	SS1	
	Other Building	N/A	SS1	SS2	
Flood Effect	Existing Lot - including infill subdivision	N/A	FE2	FE2	
	Subdivision - en globo	N/A	FE2	FE2	
	New Ancillary Building (eg shed, carport)	N/A	FE2	FE2	
	Building Extension	N/A	FE1	FE2	
	Other Developments (road raising, etc)	N/A	FE2	FE2	FE3
Evacuation & Access	Existing Lot - including infill subdivision	N/A	EA1	EA1	
	Subdivision - en globo	N/A	EA3	EA3	
	Emergency Service Site (Hospitals, etc.)	N/A	EA4a		
	Other Community Service Site (Schools, etc.)	N/A	EA4b		
Flood Awareness, etc		N/A	FA2	FA2	FA2

Note 1: An explanation of the criteria used to define the hazard categories is contained in the Herbert River Flood Study (WBM Oceanics Australia, 2002)

Note 2: Small-scale development implies development on rural land that is small relative to the width of the floodplain and is not part of a planned large-scale development.

Note 3: Weatherproof Area Definition - Enclosed areas excluding garages / carports / open verandahs

N/A	Controls Not Applicable
	Unsuitable Land Use - Not considered suitable for development
	LAND USE SUITABILITY & MINIMUM FILL LEVEL
SF1	Consider for development subject to the controls below. No minimum fill level required.
SF2	Consider for development subject to the controls below. For residential and commercial areas, the minimum fill level to be greater tha or equal to the 100 year flood level. For industrial areas, the minimum fill level to be greater than or equal to the 10 year flood level.
SF3a	Consider for development subject to the controls below. Minimum fill level greater than or equal to the PMF flood level.
SF3b	Consider for development subject to the controls below. Council to give consideration on the benefits of using the development during and after a flood emergency. If the site is to be used for a flood emergency, the minimum fill level should preferably be greater than or equal to the PMF flood leve
	MINIMUM FLOOR LEVEL
FL1	No minimum floor level required (Council to advise developer of flood risk and potential damage to building & contents. Flood levels available on request)
FL2a	All floor levels to be greater than or equal to the 100 year flood level
FL2b	For permissible uses other than residential, it is preferable to have all floor levels greater than or equal to the 100 year flood level subject to industry standards and individual site assessment.
FL2c	All habitable floor levels to be greater than or equal to the 100 year flood level plus 0.3m
FL3a	All floor levels to be greater than or equal to the PMF flood level.
FL3b	If practical, some or all floor levels to be greater than or equal to the PMF flood level, so that these buildings will be available for accommodation / storage during and after a flood emergency.
FL4a	Habitable, commercial or industrial floor levels to be as close to the *minimum floor level* above as practical and not less than the floor level of the existing building being extended if the existing floor level is less than or equal to the minimum floor level. If the extended weatherproof area3 exceeds 50% of the existing weatherproof area, the extension is treated as a new building. The extended weatherproof area is measured as the cumulative area of any previous extensions plus the proposed extension. If building is idenitified as being suitable for voluntary house raising scheme, Council to discuss potential house raising with owner.
FL4b	As for FL4a with the maximum percentage increase in extended weatherproof area[3] to be: (a) 50% if the extension's floor level is less than one (1) metre below the 100 year flood level; (b) 25% if the extension's floor level is greater than two (2) metres below the 100 year flood level; or (c) pro-rata between 50% and 25% for floor levels from one (1) metre to two (2) metres below the 100 year flood level.
	BUILDING COMPONENTS
BC1	Buildings to have flood compatible material below the higher of (a) the minimum floor level or (b) the 1 in 100 year flood level plus 0.3m.
	STRUCTURAL SOUNDNESS
SS1	No structural soundness requirements for the force of floodwater, debris & buoyancy. Must still comply with BCA requirements.
SS2	Engineers report to prove that structures subject to a flood up to the 100 year event can withstand the force of floodwater, debris & buoyancy.
SS3	Engineers report to prove that structures subject to a flood up to the 500 year event can withstand the force of floodwater, debris & buoyancy.
	FLOOD EFFECT
FE1	No action required
FE2	The flood impact of the development to be considered by Council, with Council having the right to request an engineer's report (see FE3 below)
FE3	Engineers report required to prove that the development will not result in adverse flood impact elsewhere
	EVACUATION/ACCESS
EA1	Council to provide information on flood evacuation strategy
EA2	Not used
EA3	Site specific Flood Evacuation Strategy be developed consistent with Council / SES overall Flood Evacuation Strategy.
EA4a	Emergency service site - should have good access up to the PMF and preferably not cut-off from the main residential area(s). Council to evaluate suitablility of site in this respect.
EA4b	If site to be used during and after a flood emergency (see FL3b above), should have good access up to the PMF and preferably not cut-off from the main residential area(s).
	FLOOD AWARENESS
FA1	Not used
FA2	Not used

Figure 11-6 Example Planning Matrix - Residential, Commercial, Industrial Land Use

FLOOD PLANNING MATRIX (Version 2)

TABLE 2: DEVELOPMENT IN RURAL AREAS

Controls	Development / Building Type	No Hazard[1]	Low Hazard	High Depth Hazard	High Floodway Hazard
			Flood Hazard Category		
Land Use Suitability & Fill Level	Habitable Building	N/A	SF1	SF1	
	Ancillary Building (eg. shed)	N/A	SF1	SF1	SF1
	Other Developments (eg. levees, roads, dams, etc)	N/A	SF1	SF1	SF1
	Emergency Services Site (Hospitals, etc.)	N/A	SF3a		
	Other Community Service Building (School, etc.)	N/A	SF3b	SF3b	
Floor Level	New Habitable Building	N/A	FL2c	FL2c	
	(this line not used)				
	New Emergency Service Building (Hospitals, etc.)	FL3a	FL3a		
	New Other Community Service Building (School, etc.)	FL3b	FL3b		
	New Ancillary Building (eg shed, carport)	N/A	FL1	FL1	FL1
	Building Extension	N/A	FL4a	FL4b	
	New Rural Industry	N/A	FL2b	FL2b	
Building Components		N/A	BC1	BC1	BC1
Structural Soundness	Small-scale[2] Development (eg. shed, small dam)	N/A	SS1	SS1	SS2
	Large-scale Development (eg. levee, raised road)	N/A	SS1	SS2	SS2
Flood Effect	Small-scale[2] Development (eg. shed, small dam)	N/A	FE1	FE2	FE2
	Large-scale Development (eg. levee, raised road)	N/A	FE2	FE3	FE3
	(this line not used)				
	(this line not used)				
	(this line not used)				
Evacuation & Access	Habitable Building	N/A	EA1	EA1	
	(this line not used)				
	Emergency Service Site (Hospitals, etc.)	N/A	EA4a		
	Other Community Service Site (Schools, etc.)	N/A	EA4b		
Flood Awareness, etc		N/A	FA2	FA2	FA2

Note 1: An explanation of the criteria used to define the hazard categories is contained in the Herbert River Flood Study (WBM Oceanics Australia, 2002)

Note 2: Small-scale development implies development on rural land that is small relative to the width of the floodplain and is not part of a planned large-scale development.

Note 3: Weatherproof Area Definition - Enclosed areas excluding garages / carports / open verandahs

Control Measures

N/A	Controls Not Applicable
	Unsuitable Land Use - Not considered suitable for development
	LAND USE SUITABILITY & MINIMUM FILL LEVEL
SF1	Consider for development subject to the controls below. No minimum fill level required.
SF2	Consider for development subject to the controls below. For residential and commercial areas, the minimum fill level to be greater than or equal to the 100 year flood level. For industrial areas, the minimum fill level to be greater than or equal to the 10 year flood level.
SF3a	Consider for development subject to the controls below. Minimum fill level greater than or equal to the PMF flood level.
SF3b	Consider for development subject to the controls below. Council to give consideration on the benefits of using the development during and after a flood emergency. If the site is to be used for a flood emergency, the minimum fill level should preferably be greater than or equal to the PMF flood level.
	MINIMUM FLOOR LEVEL
FL1	No minimum floor level required (Council to advise developer of flood risk and potential damage to building & contents. Flood levels available on request)
FL2a	All floor levels to be greater than or equal to the 100 year flood level
FL2b	For permissible uses other than residential, it is preferable to have all floor levels greater than or equal to the 100 year flood level subject to industry standards and individual site assessment.
FL2c	All habitable floor levels to be greater than or equal to the 100 year flood level plus 0.3m
FL3a	All floor levels to be greater than or equal to the PMF flood level.
FL3b	If practical, some or all floor levels to be greater than or equal to the PMF flood level, so that these buildings will be available for accommodation / storage during and after a flood emergency.
FL4a	Habitable, commercial or industrial floor levels to be as close to the *minimum floor level* above as practical and not less than the floor level of the existing building being extended if the existing floor level is less than or equal to the minimum floor level. If the extended weatherproof area[3] exceeds 50% of the existing weatherproof area, the extension is treated as a new building. The extended weatherproof area is measured as the cumulative area of any previous extensions plus the proposed extension. If building is idenitified as being suitable for voluntary house raising scheme, Council to discuss potential house raising with owner.
FL4b	As for FL4a with the maximum percentage increase in extended weatherproof area[3] to be: (a) 50% if the extension's floor level is less than one (1) metre below the 100 year flood level; (b) 25% if the extension's floor level is greater than two (2) metres below the 100 year flood level; or (c) pro-rata between 50% and 25% for floor levels from one (1) metre to two (2) metres below the 100 year flood level.
	BUILDING COMPONENTS
BC1	Buildings to have flood compatible material below the higher of (a) the minimum floor level or (b) the 1 in 100 year flood level plus 0.3m.
	STRUCTURAL SOUNDNESS
SS1	No structural soundness requirements for the force of floodwater, debris & buoyancy. Must still comply with BCA requirements.
SS2	Engineers report to prove that structures subject to a flood up to the 100 year event can withstand the force of floodwater, debris & buoyancy.
SS3	Engineers report to prove that structures subject to a flood up to the 500 year event can withstand the force of floodwater, debris & buoyancy.
	FLOOD EFFECT
FE1	No action required
FE2	The flood impact of the development to be considered by Council, with Council having the right to request an engineer's report (see FE3 below)
FE3	Engineers report required to prove that the development will not result in adverse flood impact elsewhere
	EVACUATION/ACCESS
EA1	Council to provide information on flood evacuation strategy
EA2	Not used
EA3	Site specific Flood Evacuation Strategy be developed consistent with Council / SES overall Flood Evacuation Strategy.
EA4a	Emergency service site - should have good access up to the PMF and preferably not cut-off from the main residential area(s). Council to evaluate suitablility of site in this respect.
EA4b	If site to be used during and after a flood emergency (see FL3b above), should have good access up to the PMF and preferably not cut-off from the main residential area(s).
	FLOOD AWARENESS
FA1	Not used
FA2	Not used

Figure 11-7 Example Planning Matrix - Rural Land Uses

FLOOD PLANNING MATRIX (Version 2)

TABLE 3: OTHER						
			Flood Hazard Category			
Controls	**Development / Building Type**	**No Hazard[1]**	**Low Hazard**	**High Depth Hazard**	**High Floodway Hazard**
Land Use Suitability & Fill Level	Non-Habitable Building (shed, toilets, shelter, etc)	N/A	SF1	SF1	SF1
	(this line not used)				
	Other Developments (eg. levees, roads, dams, etc)	N/A	SF1	SF1	SF1
	(this line not used)				
	(this line not used)				
Floor Level	(this line not used)	N/A	N/A	N/A	N/A
	(this line not used)				
	(this line not used)				
	(this line not used)				
	(this line not used)				
	(this line not used)				
	(this line not used)				
Building Components		N/A	BC1	BC1	BC1
Structural Soundness	Small-scale[2] Development (eg. shed, small dam)	N/A	SS1	SS1	SS2
	Large-scale Development (eg. levee, raised road)	N/A	SS1	SS2	SS2
Flood Effect	Small-scale Development (eg. shed, small dam)	N/A	FE1	FE2	FE2
	Large-scale Development (eg. levee, raised road)	N/A	FE2	FE3	FE3
	(this line not used)				
	(this line not used)				
	(this line not used)				
Evacuation & Access	Not Applicable				
	(this line not used)				
	(this line not used)				
	(this line not used)				
Flood Awareness, etc	Not Applicable				

Note 1: An explanation of the criteria used to define the hazard categories is contained in the Herbert River Flood Study (WBM Oceanics Australia, 2002)

Note 2: Small-scale development implies development on rural land that is small relative to the width of the floodplain and is not part of a planned large-scale development.

Note 3: Weatherproof Area Definition - Enclosed areas excluding garages / carports / open verandahs

Control Measures

N/A	Controls Not Applicable
(hatched)	Unsuitable Land Use - Not considered suitable for development
	LAND USE SUITABILITY & MINIMUM FILL LEVEL
SF1	Consider for development subject to the controls below. No minimum fill level required.
SF2	Consider for development subject to the controls below. For residential and commercial areas, the minimum fill level to be greater than or equal to the 100 year flood level. For industrial areas, the minimum fill level to be greater than or equal to the 10 year flood level.
SF3a	Consider for development subject to the controls below. Minimum fill level greater than or equal to the PMF flood level.
SF3b	Consider for development subject to the controls below. Council to give consideration on the benefits of using the development during and after a flood emergency. If the site is to be used for a flood emergency, the minimum fill level should preferably be greater than or equal to the PMF flood level.
	MINIMUM FLOOR LEVEL
FL1	No minimum floor level required (Council to advise developer of flood risk and potential damage to building & contents. Flood levels available on request)
FL2a	All floor levels to be greater than or equal to the 100 year flood level.
FL2b	For permissible uses other than residential, it is preferable to have all floor levels greater than or equal to the 100 year flood level subject to industry standards and individual site assessment.
FL2c	All habitable floor levels to be greater than or equal to the 100 year flood level plus 0.3m
FL3a	All floor levels to be greater than or equal to the PMF flood level.
FL3b	If practical, some or all floor levels to be greater than or equal to the PMF flood level, so that these buildings will be available for accommodation / storage during and after a flood emergency.
FL4a	Habitable, commercial or industrial floor levels to be as close to the *minimum floor level* above as practical and not less than the floor level of the existing building being extended if the existing floor level is less than or equal to the minimum floor level. If the extended weatherproof area[3] exceeds 50% of the existing weatherproof area, the extension is treated as a new building. The extended weatherproof area is measured as the cumulative area of any previous extensions plus the proposed extension. If building is idenitified as being suitable for voluntary house raising scheme, Council to discuss potential house raising with owner.
FL4b	As for FL4a with the maximum percentage increase in extended weatherproof area[3] to be: (a) 50% if the extension's floor level is less than one (1) metre below the 100 year flood level; (b) 25% if the extension's floor level is greater than two (2) metres below the 100 year flood level; or (c) pro-rata between 50% and 25% for floor levels from one (1) metre to two (2) metres below the 100 year flood level.
	BUILDING COMPONENTS
BC1	Buildings to have flood compatible material below the higher of (a) the minimum floor level or (b) the 1 in 100 year flood level plus 0.3m.
	STRUCTURAL SOUNDNESS
SS1	No structural soundness requirements for the force of floodwater, debris & buoyancy. Must still comply with BCA requirements.
SS2	Engineers report to prove that structures subject to a flood up to the 100 year event can withstand the force of floodwater, debris & buoyancy.
SS3	Engineers report to prove that structures subject to a flood up to the 500 year event can withstand the force of floodwater, debris & buoyancy.
	FLOOD EFFECT
FE1	No action required
FE2	The flood impact of the development to be considered by Council, with Council having the right to request an engineer's report (see FE3 below)
FE3	Engineers report required to prove that the development will not result in adverse flood impact elsewhere
	EVACUATION/ACCESS
EA1	Council to provide information on flood evacuation strategy
EA2	Not used
EA3	Site specific Flood Evacuation Strategy be developed consistent with Council / SES overall Flood Evacuation Strategy.
EA4a	Emergency service site - should have good access up to the PMF and preferably not cut-off from the main residential area(s). Council to evaluate suitablility of site in this respect.
EA4b	If site to be used during and after a flood emergency (see FL3b above), should have good access up to the PMF and preferably not cut-off from the main residential area(s).
	FLOOD AWARENESS
FA1	Not used
FA2	Not used

Figure 11-8 Example Planning Matrix - Other Land Uses

11.4.7 Use of HSC 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 presented in Section 5.3.7 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.

11.4.8 Intangible Benefits

Controls on new development lower the health, social, and psychological trauma associated with flooding. In addition, it is less likely that people residing in new dwellings require evacuation and they may not need to remove their possessions. All of these factors help reduce the impact of flooding.

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

12 RESPONSE MODIFICATION MEASURES

12.1 Background

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. The length of lead time varies from less than a day if the rainfall is predominantly in the lower catchment and up to 2 days if the rainfall is predominantly in the upper catchment. 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 Counter Disaster Committee (CDC) on three occasions and provided opportunities at the community open sessions for members of the public to comment so that the study team could assess community perceptions. In the following sections, further background information is provided along with an assessment of the status quo and recommendations for consideration by the Study Advisory Group.

12.2 Flood Warning & Emergency Planning

12.2.1 Description

The primary responsibility for flood warning and emergency response in the Hinchinbrook 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.

12.2.2 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. Raising community awareness is an important component of this study and is referred to again in Section 12.2.5. It is important to note that community awareness and flood warning are strongly linked.

12.2.2.1 Status Quo

The following assessment of the current level of community awareness is based on information obtained through the community open sessions and meetings with the CDC.

- The community has a high flood awareness because of the regular flooding that has historically occurred in the floodplain.
- The CDC have a high level of flood awareness and have had considerable experience in responding to floods.
- There is some confusion in the community as to what flood information is available, how it can be accessed during a flood and how it can be interpreted (specific community responses are provided in 12.2.5.1.1).
- A full page advertisement is placed in the newspaper once a year to provide the community with information on the flood warning and response system. A pamphlet relating gauge heights to flood impacts is available from the Council office and the Council website provides information on counter disaster operations.

12.2.2.2 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 12.3.

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 CDC, the SAG and the community during open sessions. There has been general support for the concept from the community, but the CDC and the Bureau of Meteorology (BoM) have expressed some reservations as detailed in Section 12.2.5.1.2.

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 SES to effectively communicate the expected peak level of a flood in a way that all can understand and readily apply to their own situation.

The flood totem is discussed in further detail in Section 12.2.5.1.2.

12.2.3 Quality of Flood Information Received by the CDC

12.2.3.1 Status Quo

An 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 is not 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 flood and provides reliable peak flood height predictions in the Lower Herbert.

12.2.3.2 Recommendations

No modification to the system is recommended other than expanding the ALERT system with the installation of a new river height gauge on the Herbert River downstream of the Stone River. This gauge would be used for the colour classification of floods. There is greater variation in the river flood height at this location than occurs at the Ingham Pump Station or at Gairloch thereby giving a better indication of the magnitude of the flood. Being downstream of the Stone River will mean that flooding in the Stone River is incorporated into the flood height prediction as is currently the case with the Ingham Pump Station and Gairloch gauge.

The SAG's recommendation is that a river height gauge be installed on the Herbert River downstream of the Stone River.

12.2.4 Assessment of Flood Information

12.2.4.1 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 up to about only about a 50 year ARI event (1967 flood) and this is now 35 years ago.

The assessment of the flood relies heavily on the predicted river height at Gairloch gauge, as well as less formal procedures based on rainfall in the catchment. As has been noted previously, the height at the Gairloch gauge is relatively insensitive to increasing flood magnitude. Therefore, there can be significant variation in flood depth and extent on the floodplain with only a relatively minor change in the height at the Gairloch gauge. This is not ideal for emergency management. The BoM does provide peak flood height predictions at other gauges, but the CDC and community have a "feel" for the Gairloch gauge height. However, during the open sessions, the study team was frequently asked why there can be significant difference in the flood level on the floodplain from flood to flood even though the Gairloch gauge height is similar.

12.2.4.2 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 previously experienced. Examples of the following were presented to the CDC:

1. Flood inundation extent and depth maps

2. 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 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. Coloured flood totems would be positioned at various locations around Ingham 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.

In some of the lower areas of the floodplain, there is little variation in flood level above a particular level. For example, in Halifax there is little variation between the peak flood height in a 5 year and 100 year ARI floods. At locations such as this, classification of floods using colours is not feasible.

The SAG's recommendation is that colour classification of floods and emergency maps are included in a Floodplain Management Plan.

12.2.5 CDC Response

12.2.5.1 Warnings

12.2.5.1.1 Status Quo

Once the CDC has assessed the data received, their primary role is then to inform the community. Flood information is distributed to the community in the following ways:

- Hourly facsimile updates to agencies, shops, individuals and media of rainfall, current river height and peak river height predictions;
- Flood information telephone lines;
- Rainfall, current river height and peak river height predictions posted outside Council office in Ingham;
- Radio broadcasts.

The community can also access this information through the BoM web site

During the community open sessions the community was asked to provide their perceptions of the flood warning system. The comments received are summarised below into three categories; flood warning efficiency; flood warning medium; general comments.

- Flood Warning Efficiency
  - Improved flood warning required – including outside towns
  - Improved reports from further up catchment, especially rainfall
  - Existing flood warning over radio are adequate – opposite viewpoint also expressed
  - Flood awareness info in papers is more than adequate
  - Improve weather forecasting in Nth QLD on weekends
  - Information 12 hours out of date
- Flood Warning Medium
  - Public awareness of local radio transmitter in Ingham that could give immediate updated river levels
  - Flood board info should be updated hourly in all areas of the shire not just in Ingham
  - Radio reports
  - Nominate one radio station that will broadcast flood information
  - Broadcast information on ABC or AM
  - Fix up flood boards at Halifax
  - Use Italian radio transmitter
- General Comments
  - Identify problem areas and have self-help committees
  - Flood reporting wardens for each area, eg Cordelia, Macknade etc
  - Flood evacuation planning required
  - Totem poles good idea
  - Identification of highest point in areas that have been cut-off
  - Household package with flood warning information, eg, location of gauges
  - Improve community awareness

Although the people surveyed represent only a small percentage of the community, it could be assumed that there is a general perception that the flood warning system could be improved. This is largely because people are not aware of the flood warning system implemented by the CDC, indicating that the community education program needs to be expanded. For example, many people were not aware of the flood information telephone lines (or believed the information is old) and were not aware that flood warnings were issued 7 days a week, 24 hours a day by local radio station 4JC during recent past floods. It is understood that an arrangement for flood warning broadcasts is being made with 4KZ, which has greater coverage in the Shire.

12.2.5.1.2 Recommendations

Some of the ideas put forward could be considered by the CDC. For example, establishing flood bulletin boards at other locations in the district and extending the use of flood wardens (the system is currently established in Ardrossan Estate).

It is recommended that flood totems be implemented to improve the community's awareness of flooding and to improve their appreciation of the implication of the flood in their local area. The system would work as follows:

1. The CDC will receive a predicted peak flood height at the proposed gauge at Trebonne (downstream of Stone River);
2. Using the flood classification chart in Figure 12-1, 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 through the normal procedures (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 for Rotaract Park in Ingham is given in Figure 12-2 - note that the actual totem would only show colours, not flood levels or flood ARI.).

The colour scheme presented here is intended to be indicative only. The final colour scheme should be determined in consultation with the SES at both a local and state level.

Flood totems are not recommended in the following areas because there is insufficient variation in flood level between small and large floods (these areas will still benefit from the range of other response and property modification measures proposed):

- Toobanna;
- Macknade;
- Cordelia;
- Halifax.

The BoM has informally expressed some reservation about colour classification of floods because they believe it may cause confusion with their current flood classification system which also is colour coded. However, the BoM colour classification system is not well recognised nor actively promoted.

The CDC has expressed the following reservations in relation to the colour classification of floods and the totem system:

- Cost – some thought the money would be better spent in the disaster room;
- Legal liability if they got the flood prediction wrong;
- Fading of colours;
- People with colour blindness;
- Interference with the totems (vandalism);
- Large height range of some bands.

Some general comments are provided on these concerns. The scheme could be fully funded through the Regional Flood Mitigation Program, so cost should not be an issue. In relation to legal liability, the CDC seemed to be indicating that it is better to keep the public uninformed than to keep them informed and possibly get it wrong sometimes. Maintenance would overcome fading problems. Vandalism may be a problem, but this should not be allowed to dictate what we do and do not do as a community. The height ranges presented to the CDC were indicative only and could be reduced when the system is designed. However, a reasonable range in the bands is desirable to allow for some tolerance in the predictions.

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 Ingham, 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 Ingham at 3am tomorrow morning. Repeating... A blue flood is expected to peak in Ingham at 3am tomorrow morning. For more information please tune to Radio Station 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.

12.2.5.2 Community Support During Floods

12.2.5.2.1 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. For example, a record is kept in each flood that relates areas that require evacuations and furniture lifting to river gauge height. However, as noted previously, the procedures are based on experience of floods up to about a 50 year ARI event.

12.2.5.2.2 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 12-3.

This re-assessment should include the probable maximum flood. 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.

12.2.6 Intangible Benefits

An effective flood warning system is invaluable in minimising the flood damages and trauma associated with flooding. An accurate, prompt warning system ensures that residents/business are given the best opportunity to remove possessions and themselves from the dangers of floodwaters. Effective response during a flood can help minimise the flood damages and trauma associated with flooding. Being able to promptly warn those at risk and assign resources appropriately ensures that residents are given the best opportunity to remove possessions and themselves from the dangers of floodwaters.

12.2.7 Summary

The Committee's recommendation is that the Floodplain Management Scheme should require:

- ***That a new river height gauge be installed on the Herbert River downstream of the StoneRiver***
- ***that the CDC undertake a review of the data from the study to ensure that planning is adequate for larger flood events;***
- ***that flood totems be implemented;***
- ***that a flood warden system be implemented;***
- ***that flood warnings be posted in other parts of the Shire, not just outside the Council building.***

12.3 Raising Community Awareness

12.3.1 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 Herbert 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, should be the target of a flood information campaign.

12.3.2 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).

12.3.3 General Messages

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

- many areas of the Herbert 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 Herbert 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.
- **Totem Poles** - showing the colour-coded flood bands. 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. 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 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** – Part of HSC site could be devoted to flood awareness and flood warning with links to the BoM web site.

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 Herbert River region.

12.3.4 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, 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 1967 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).

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. 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 12-4, 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.

12.3.5 Intangible Benefits

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

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

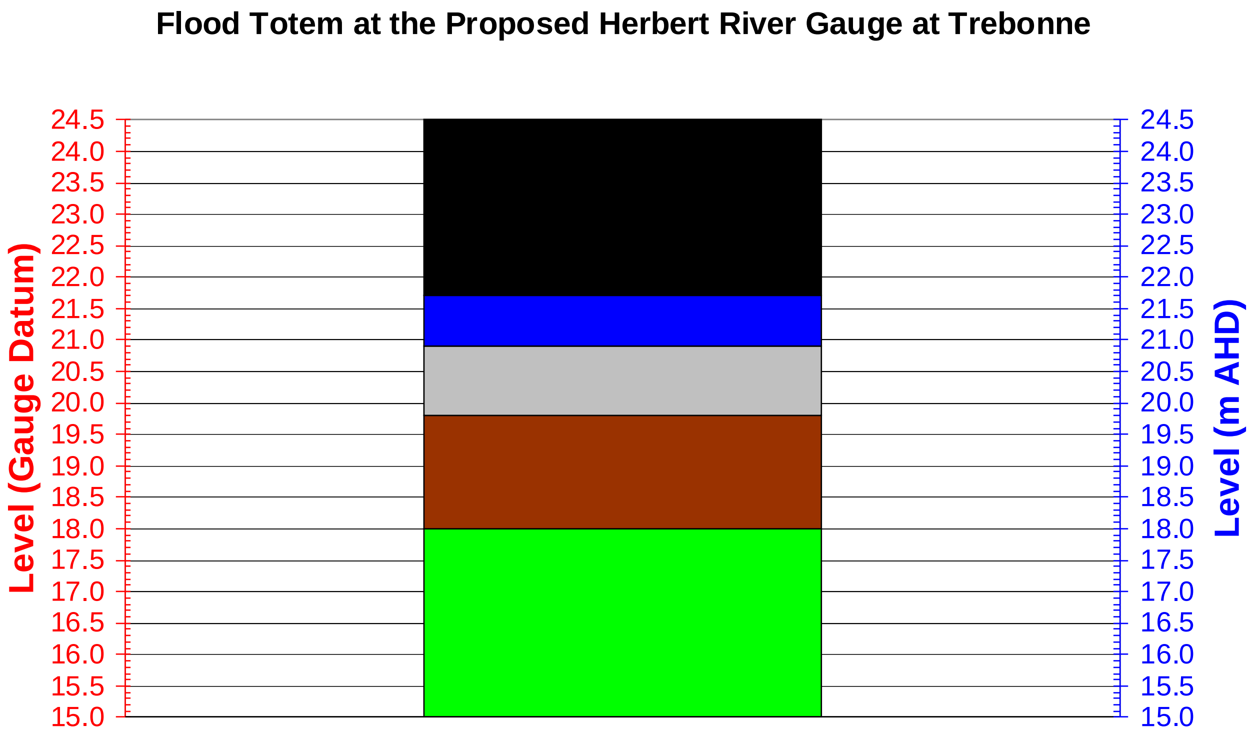


Figure 12-1 Flood Classification at Herbert River Gauge at Trebonne

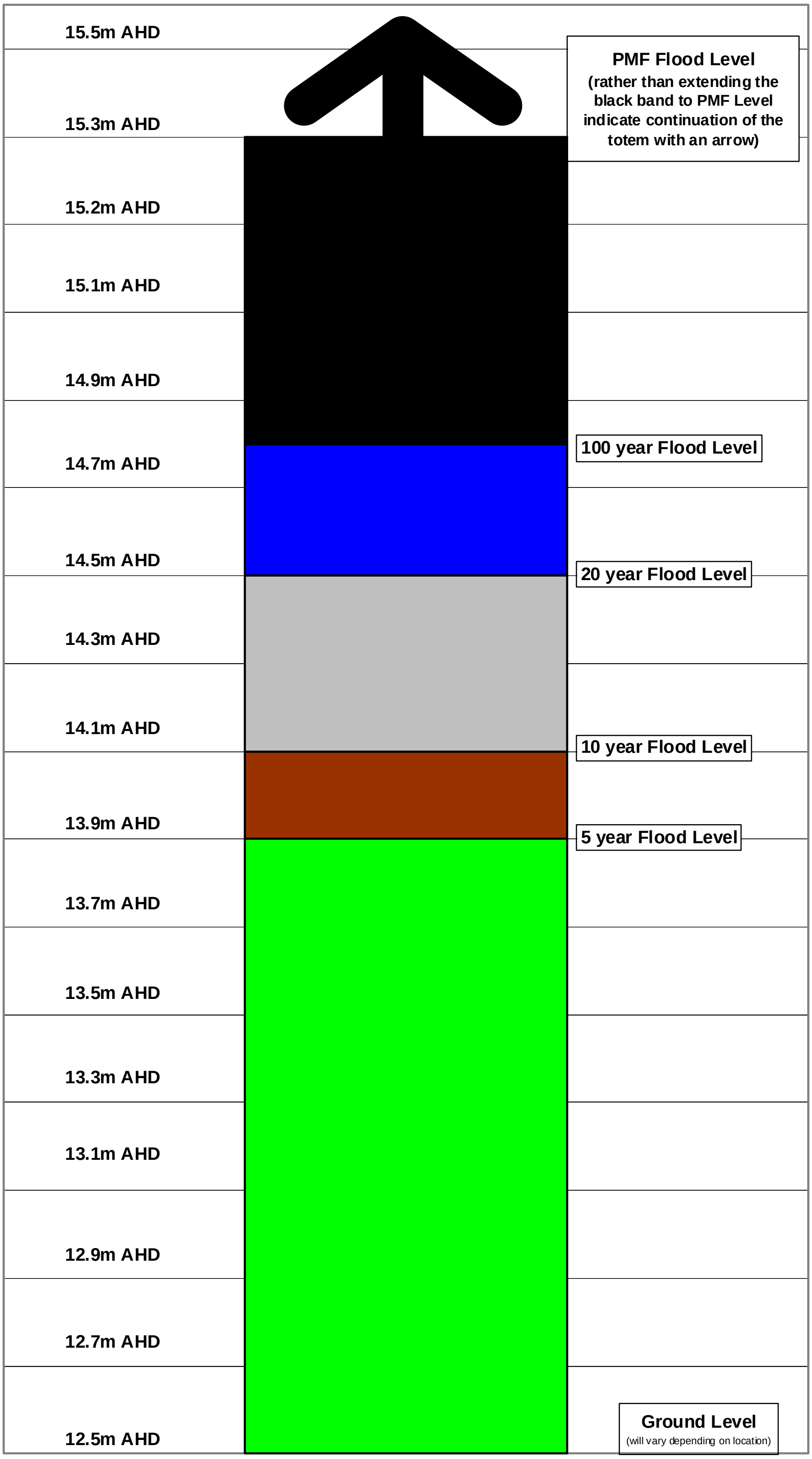


Figure 12-2 Flood Totem at Rotaract Park – Ingham

Note that the actual totem would only show colours, not flood levels or flood ARI.

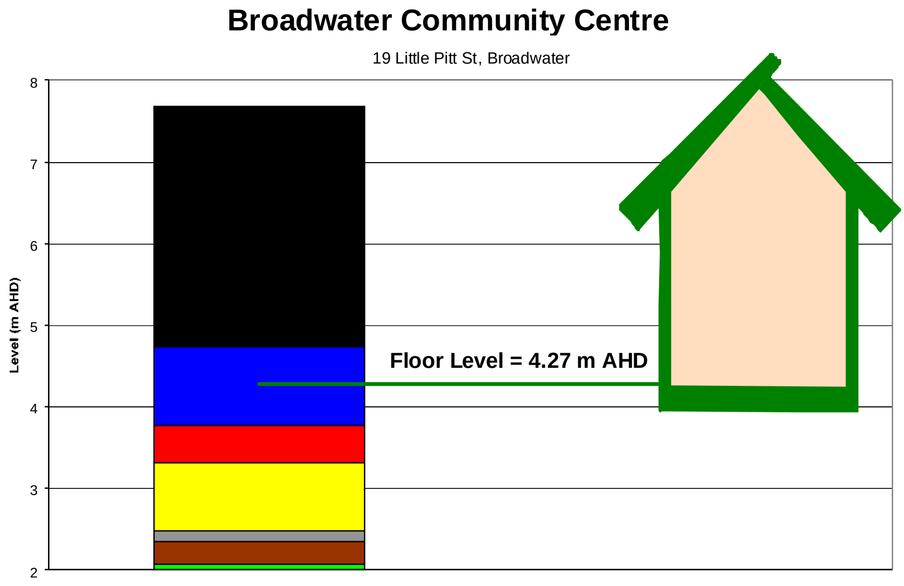


Figure 12-3 Example of Classification of Evacuation Centre

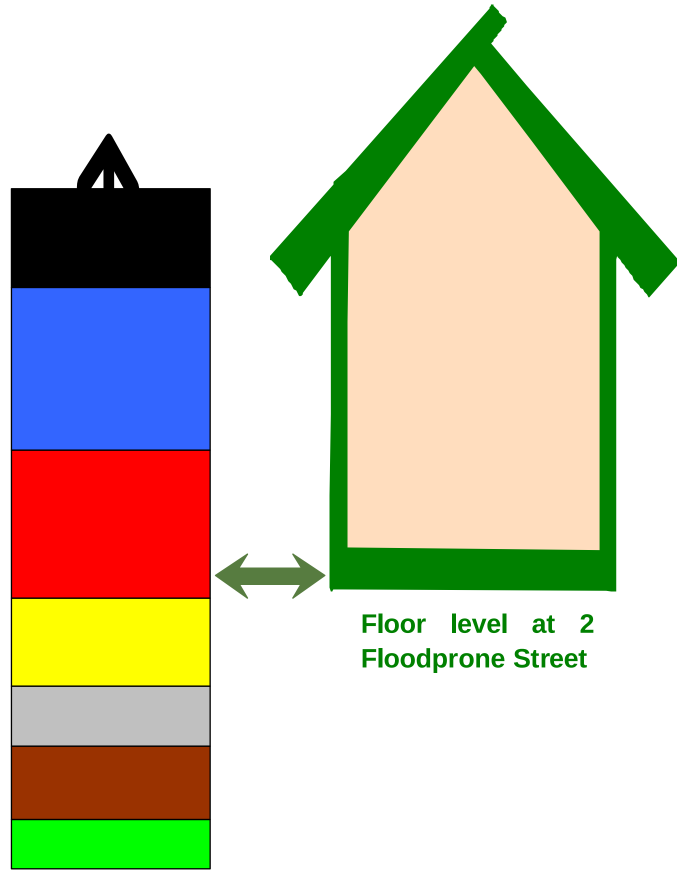


Figure 12-4 Example of a Household Flood Diagram

13 COMBINATION OF MEASURES

Sections 10 to 12 presented structural and non-structural flood management options. Hydraulic impacts, economic benefits, intangible benefits and environmental considerations were considered. Of the structural measures considered, the SAG selected the Halifax levee and Kingsbury Creek floodgate for inclusion in the Floodplain Management Plan.

The previous analyses considered the measures in isolation in terms of the hydraulic impacts and economic benefit. Therefore, it is necessary to determine the benefits and impacts of the measures in combination before they are recommended in the floodplain management scheme.

One of the non-structural measures selected by the SAG was voluntary house raising. Although it is categorised as a non-structural management measure, the implementation or otherwise of it will impact on the economic viability of the structural measures. Therefore, it is included in the economic analysis of the combined structural measures.

This section presents the analysis of the measures in combination.

13.1 Hydraulic Impacts

The hydraulic model representing the existing floodplain was modified to incorporate both the 5 year ARI Kingsbury Creek floodgate and the 100 year ARI Halifax levee and run using the 5 year, 10 year, 20 year, 50 year and 100 year ARI floods. The change in peak flood level for the 5 year, 20 year and 100 year ARI floods are presented in Drawing 13-1 to Drawing 13-3 respectively. A comparison of these drawings with those presented in Section 10 (refer Drawing 10-15 to Drawing 10-17 and Drawing 10-19 to Drawing 10-22) demonstrates that there is little interaction between the measures, ie the combination of the two measures has not increased the impacts of the individual measures.

Similarly, a review of the impact of the measures on velocity indicated that combining the measures has not altered the impacts presented in Section 10.

13.2 Benefit-Cost Analysis

The benefit-cost analysis was undertaken using the same methodology as described in Section 10 for the analysis of the individual measures. The benefits from both these measures are largely a result of a reduction in above floor flooding in residential and commercial buildings. Therefore, the implementation of a voluntary house raising scheme is in conflict in some parts of the floodplain with the proposed levee and floodgates because the benefits derived from the construction of the floodgate and levee will be reduced if houses are raised. To determine the sensitivity of the proposed floodgate and levee to the house raising scheme, the following two schemes were analysed:

1. Kingsbury Creek floodgate and Halifax levee with no house raising;

2. Kingsbury Creek floodgate and Halifax levee with “raisable” residential houses in urban areas lifted to above the 100 year ARI flood level.

The flood damages for each of the flood events and the annual average flood damages was calculated for both schemes. The cost-basis for the analysis is presented in Table 13-1 and Table 13-2. In Section 10, two costs were provided to provide an upper and lower bound for the Halifax Levee. The lower bound used the Maunsell McIntyre construction costs combined with the on-going costs as estimated by WBM and the upper bound used the earthworks quantities approximated by WBM using the ground elevations information in the DEM. Maunsell McIntyre calculated quantities using survey of the existing levee and it is considered to be a better estimate of the earthworks quantities. The construction costs presented in Table 13-2 are minor adjustments to Maunsell's costs. Another difference between these costs and those presented in Section 10 relates to community education. In Section 10, $55,000 was included in the costs for both Kingsbury Creek Floodgate and the Halifax levee because the analysis assumed that the measure was be implemented independently. The measures being considered in combination in this Section and the $55,000 is only included once and it is incorporated into Table 13-3 and Table 13-4.

As shown in Table 13-2, $179,000 has been allowed for mowing and gardening over a 50 year period. A considerable portion of the levee will be behind private properties in currently vegetated sections of the river bank. It is likely that rather than mowing the levee, it would be left to become heavily vegetated. If this is the case, the allowance for mowing could be considerably reduced.

Results from the economic analysis for the two schemes are presented in Table 13-3 and Table 13-4. The addition of the house raising to the scheme reduces the BCR from 2.9 to 0.45, assuming all houses identified for the scheme are raised. The house raising scheme has a significant influence on the BCR because its capital cost is significantly more than the other two measures combined. However, as was suggested in Section 11.3, it is likely that the take up rate for the house rasing scheme would be considerably less than 100 %, and so the BCR for the full scheme would be somewhere in the range presented here.

Table 13-1 Cost Breakdown - Kingsbury Ck Floodgate

Item	Cost
Levee Construction	$ 100,000
Floodgate	$ 75,000
Landscaping	$ 10,000
Sub-Total 1	$ 185,000
Contingencies (25%)	$ 46,000
Sub-Total 2	$ 231,000
Consultation, Survey, Admin	$ 20,000
Engineering	$ 20,000
CONSTRUCTION TOTAL	**$ 271,000**
Mowing and Gardening	$ 5,000
MAINTENANCE TOTAL	**$ 5,000**
Levee Monitoring, Gate Operation	$ 5,000
OPERATION TOTAL	**$ 5,000**
TOTAL	**$ 281,000**

Table 13-2 Levee Construction Costs, Halifax Levee

Item	Cost
Levee Construction	$ 100,000
Landscaping	$ 10,000
Sub-Total 1	$ 110,000
Contingencies (25%)	$ 28,000
Sub-Total 2	$ 138,000
Consultation, Survey, Resumptions, Admin	$ 20,000
Engineering & Geotech	$ 35,000
CONSTRUCTION TOTAL	**$ 193,000**
Mowing and Gardening	$ 179,000
MAINTENANCE TOTAL	**$ 179,000**
Levee Monitoring incl. Annual Survey	$ 112,000
Sand Bags for Road Crossings	$ 20,000
OPERATION TOTAL	**$ 132,000**
TOTAL	**$ 504,000**

Table 13-3 BCR Analysis of Scheme 1

Average Annual Benefit	Total Benefit over 50 Years	Construction Costs	On-Going Costs over 50 Years*	Total Cost	BCR
$176,000	$2,430,000	$464,000	$376,000	$840,000	2.9

*Includes $55,000 for on-going community education

Table 13-4 BCR Analysis of Scheme 2

Average Annual Benefit	Total Benefit over 50 Years	Construction Costs	On-Going Costs over 50 Years*	Total Cost	BCR
$375,000	$5,180,000	$11,154,000	$376,000	$11,530,000	0.45

*Includes $55,000 for on-going community education

13.3 Environmental Considerations

The environmental constraints for the two structural measures were detailed in Section 10.4.

13.4 Summary

It is the SAG's recommendation that both the Kingsbury Creek Floodgate and the Halifax Levee be incorporated into the Floodplain Management Plan and that a voluntary house raising program be implemented. The SAG has recommended that funding should be sought to raise 20 houses in the first year and if successful, call expressions of interest from residents on the basis of 1/3 funding from Federal and State Governments with 1/3 contribution from the owner.

14 FUNDING OPTIONS

Funding for floodplain management works and flooding warning systems in Queensland is available through the Regional Flood Mitigation Program, which is administered by the Department of Natural Resources and Mines, and the Queensland Department of Local Government.

14.1 Regional Flood Mitigation Programme

The Regional Flood Mitigation Programme is a Federal Government initiative working in partnership with State and Local Governments in the implementation of priority, cost effective flood mitigation works and measures in rural, regional and outer metropolitan Australia. The Programme is designed to integrate with the Federal Government's approach to natural disaster mitigation throughout Australia. Projects funded by the Regional Flood Mitigation Programme are those that address flooding issues as part of regional floodplain management.

The Regional Flood Mitigation Programme provides funding to Local Government in Queensland for on-the-ground flood mitigation works or for flood warning systems. Funding is 1/3 from each level of Govt, except for the capital costs of flood warning systems, which are fully funded through the scheme. The exception is flood warning systems, which can be fully funded by the DNRM. The annual pool of money from Federal and State is currently $6.72 M (3.36 each) and is expected to remain at that for the next few years.

The scheme funds capital costs, but not maintenance. Capital cost includes design fees, although the design fees cannot be claimed until the first construction progress payment, ie, the Council usually carries these fees. Funding is available for existing development only, not new developments.

If a project is expected to go over more than one year (eg house raising), the total funds required are applied for, but with a staged program. However, in subsequent years the funding is not guaranteed, but the project will be a front-runner in the assessment for that year and is likely to receive on-going funding. If a project does not receive funding in the second year for example, future applications can still be made for the same project.

Initially there is an expression of interest (EOI) stage to filter out the applications that would not be successful; this often relates to the detail of the background study and the dollars requested. For example, an application for $1M to build a levee based on a $10,000 HEC-RAS assessment is unlikely to be successful, but an application for $5,000 to put some flood markers would not require a background study. Application for funds that has come from recommendations out of the Herbert River Flood Study should make it through the EOI stage.

Applications are assessed using the following criteria, with 1 and 2 having the highest weighting.

1. Effectiveness - the % reduction in AAD is used. If it is a house raising scheme, the AAD to residential properties only is used

2. Economic - BCR is used - no scheme is eliminated because of a low BCR, but if it is less than 0.5 it may not be successful depending on the other applications.

3. Qualitative (Social/Intangibles) - no firm rules, but the % of the community that would benefit is considered.

4. Environmental benefits.

Schemes are ranked using the above criteria and funds allocated down the list until they run out. Historically there has not been a big mis-match between level of funding sought and that available through the scheme, although that this may change with the DES risk studies programme now in place.

14.2 Department of Local Government & Planning

The Queensland Department of Local Government & Planning administers the Local Governing Bodies Capital Works Scheme. Funding for flood mitigation works and studies is available through the scheme to Local Councils for existing urban developments. The scheme offers 20% subsidy for works programs. Funding is not available for the following:

- flood warning systems;
- property modification measures;
- urban drainage works associated with the drainage of catchments, the purpose of which is to prevent localised flooding;
- works directed towards opening up flood prone land for future development

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