Town of Home Hill Flood Study

BURDEKIN SHIRE COUNCIL

Final

10/09/2003
Town of Home Hill Flood Study

BURDEKIN SHIRE COUNCIL

Final
10/09/2003

Sinclair Knight Merz
ABN 37 001 024 095
369 Ann Street, Brisbane 4000
PO Box 246
Spring Hill QLD 4004 Australia
Tel: +61 7 3244 7100
Fax: +61 7 3244 7307
Web: www.skmconsulting.com

COPYRIGHT: The concepts and information contained in this document are the property of Sinclair Knight Merz Pty Ltd. Use or copying of this document in whole or in part without the written permission of Sinclair Knight Merz constitutes an infringement of copyright.
## Contents

**Acknowledgments**  
1

**Executive Summary**  
2

1. **Introduction**  
6

2. **Catchment Description**  
8  
2.1 The Study Area  
8  
2.2 Description of Flooding  
10  
2.2.1 Regional River Flooding  
10  
2.2.2 Local Catchment Flooding  
12  
2.2.3 Historical Floods  
13  
2.3 Existing Mitigation Measures  
15  
2.3.1 Burdekin River Improvement Trust  
15  
2.3.2 Flood Warning System  
15

3. **Hydrologic Review and Modelling**  
16  
3.1 Regional River Hydrology  
16  
3.1.1 Introduction  
16  
3.1.2 Flood Frequency Analysis  
16  
3.2 Local Catchment Hydrology  
19  
3.2.1 Overview  
19  
3.2.2 Model Description  
20  
3.2.3 Model Setup and Parameters  
20  
3.2.4 Critical Duration Analysis  
25  
3.2.5 Model Calibration and Validation  
25

4. **Hydraulic Analysis**  
26  
4.1 Overview  
26  
4.2 Hydraulic Model Description  
26  
4.3 Model Setup and Parameters  
26  
4.3.1 Survey and Topographic Data  
27  
4.3.2 Manning’s Roughness  
28  
4.3.3 Boundary Conditions  
29  
4.4 Regional Flood Model  
30  
4.4.1 Regional Model Establishment  
30  
4.4.2 Model Calibration  
31  
4.4.3 Regional Flood Model Results  
33  
4.5 Local Flood Model  
34  
4.5.1 Local Model Establishment  
34
4.5.2 Model Calibration 37
4.5.3 Local Flood Model Results 37
4.6 Impacts of the Bruce Highway and Brisbane-Cairns Rail Line on Flood Passage 39
4.7 Home Hill Stormwater Drainage Analysis 40

5. Flood Risk Management 42
5.1 Risk Management Principles 42
5.2 Regional Flooding Mitigation Measures 43
5.2.1 Home Hill Levee 43
5.2.2 Flood Warning System and Emergency Response Plan 44
5.2.3 Land Planning 45
5.2.4 Sand Deposit and Vegetation Clearing 46
5.2.5 Bank stabilisation 48
5.3 Local Flooding Mitigation Measures 48
5.3.1 Option A: Pump to River 48
5.3.2 Option B: Floodways 51
5.3.3 Option C: Aquatic Weed Reduction 55
5.3.4 Waterway management 58
5.3.5 Flood Easements 58
5.4 Flood Risk Management Strategy Plan 59

6. References 63

Appendix A Burdekin River Catchment 64
Appendix B Historical Flood Inundation Maps 65
Appendix C Regional Model Calibration 66
Appendix D Regional Flood Model Results 67
Appendix E Local Flood Model Results 68
Appendix F Hydraulic Structures Analysis 69
Appendix G Stormwater Drainage Analysis 70
Figures

- Figure 2-1: Study Area 8
- Figure 2-2: Major Creeks and Irrigation Channels 9
- Figure 2-3: Home Hill Floodplain Longitudinal Profile 12
- Figure 3-1: Design Events Hydrographs 19
- Figure 3-2: Subcatchment Boundaries 21
- Figure 3-3: RAFTS Model Layout 23
- Figure 4-1: Survey Data Extent 28
- Figure 4-2: Regional Model Bathymetry (50m) 30
- Figure 4-3: Calibration Results at Inkerman Bridge 32
- Figure 4-4: Regional Flooding Inundation Map (20 year ARI) 33
- Figure 4-5: Local Model Bathymetry (20m) 35
- Figure 4-6: Roughness Map for Local Flood Model (20m) 36
- Figure 4-7: Local Flooding Inundation Map (20 year ARI) 37
- Figure 5-1: Proposed Levee 44
- Figure 5-2: Example of Designated Flood Corridors 46
- Figure 5-3: Sand deposit and vegetation clearing – Afflux Plot 47
- Figure 5-4: Option A - Pump Location 49
- Figure 5-5: Option A - Afflux Plot 50
- Figure 5-6: Option A - Discharge Hydrographs 51
- Figure 5-7: Option B - Proposed Floodways 52
- Figure 5-8: Option B - Afflux Plot 53
- Figure 5-9: Option B - Discharge Hydrographs 54
- Figure 5-10: Option C - Channel Vegetation Reduction 55
- Figure 5-11: Option C - 5 Year ARI Afflux Plot 56
- Figure 5-12: Option C - 20 Year ARI Afflux Plot 57
## Tables

- Table 3-1: Design flood peak discharges (Kinhill, 2000)  
- Table 3-2: Calculated Pervious / Impervious areas  
- Table 3-3: IFD Data for Home Hill  
- Table 3-4: Hydrologic Model Parameters Summary  
- Table 3-5: Peak Discharges and Critical Storm Durations  
- Table 4-1: Roughness Values  
- Table 4-2: Adopted Manning's Coefficients  
- Table 5-1: Risks and Consequences of Flooding  
- Table 5-2: Summary of Risks and Mitigation Options
Document history and status

<table>
<thead>
<tr>
<th>Revision</th>
<th>Date issued</th>
<th>Reviewed by</th>
<th>Approved by</th>
<th>Date approved</th>
<th>Revision type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Draft</td>
<td>08/07/2003</td>
<td>FV</td>
<td>RNB</td>
<td></td>
<td>Draft report for comment</td>
</tr>
</tbody>
</table>

Distribution of copies

<table>
<thead>
<tr>
<th>Revision</th>
<th>Copy no</th>
<th>Quantity</th>
<th>Issued to</th>
</tr>
</thead>
<tbody>
<tr>
<td>Draft</td>
<td>1</td>
<td>1</td>
<td>Burdekin Shire Council</td>
</tr>
<tr>
<td>Draft</td>
<td>1</td>
<td>1</td>
<td>SKM Internal</td>
</tr>
<tr>
<td>Final</td>
<td>1</td>
<td>3 hard</td>
<td>Burdekin Shire Council</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1 digital</td>
<td>Burdekin Shire Council</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>SKM Internal</td>
</tr>
</tbody>
</table>
Acknowledgments

Sinclair Knight Merz would like to acknowledge the inputs and support of the following people and organisations during this project:

- Burdekin Shire Council (Trevor Williams, Gary Keane, Gary Pappalardo, Gary Bowtell, Wayne Odorica)
- South Burdekin Water Board (Bill Lowis)
- Bureau of Meteorology Brisbane (Terry Malone, Anna Moloy)
- AAM Surveys (Jamie Hansen)
- Department of Main Roads Brisbane (Garry Burton)
Executive Summary

An assessment of the flood risks that exist for the Home Hill area has been undertaken as part of the National Disaster Risk Management Studies Program. The investigation has been based on the following strategy:

- Establish flooding characteristics, issues, and risks;
- Establish computer models to simulate flooding behaviour;
- Identify potential risk mitigation options;
- Evaluate the impacts of the mitigation options using the computer models;
- Evaluate and prioritise mitigation options based on cost and benefits;
- Integrate options into a Flood Risk Management Plan.

The investigation involved considerable consultation with local landholders, the South Burdekin Water Board and Burdekin Shire Council officers regarding historical flood behaviour.

Two main flood processes were identified for the study area; **regional flooding** caused by overflows from the Burdekin River, and **local flooding** resulting from local catchment rainfall runoff.

River Flooding

River floods result in widespread inundation throughout the Home Hill area, including Home Hill Township. Flood flows are very large, and widespread damage to crops and regional infrastructure often occurs. There are several known areas along the river where breakouts occur, although experience has shown that there is a significant risk of new breakouts occurring during large floods.

There have been significant river floods in the Home Hill area on four occasions in recent history, the most recent of which occurred in 1991. Flood frequency analysis indicates that the worst flood in recent history (1958) was about a 1 in 20 year event.

Local Catchment Flooding

The Home Hill area can be subjected to very heavy and intense tropical rainfall, particularly as a result of cyclone activity. Significant flooding can occur without river overflows occurring. The landscape is generally very flat, and has been highly developed for sugar cane production. Loss of natural watercourses due to development, agricultural drainage, and aquatic weed growth all contribute to flooding problems. The worst affected areas are located in downstream areas, where the ground is naturally low lying, and has poor natural drainage outfall.
The main impacts of local flooding are usually crop damage and loss of road access for up to several weeks. There are concerns amongst local farmers that the frequency of flooding is increasing.

**Flood Modelling**

To simulate the behaviour of these two processes, two-dimensional flood models were established to simulate both the river flooding and local flooding processes. These models were based on terrain data collected by aerial laser survey, with the main difference between the models being the higher terrain resolution used in the local flood model. Although limited calibration data was available, the models are believed to provide a reasonable description of flooding behaviour.

As a result of discussions with local stakeholders, and through analysis of the modelled flood behaviour, a range of potential risk mitigation options were identified. These included:

- **Home Hill Levee**: Construction of a levee around the west side of Home Hill township to exclude local and some river flooding.

- **Flood Easements**: Procurement of flood easements by Council over the Kidby Gully and Ford’s Gully watercourses downstream of Home Hill to secure the main drainage outfall pathways for the town.

- **Flood Corridor Designation**: Designation of Flood Corridors over the major flood pathways through the area, to ensure Council can control future development in these areas.

- **Burdekin River Sand and Vegetation Removal**: Excavation of sand and removal on vegetation from within the bed of the River to lower flood levels.

- **Floodways to River**: Construction of floodways to the River to provide relief from flooding in downstream areas.

- **Pumping to River**: Construction of flood relief pumps to pump water from downstream areas to the River, as a means of reducing the period of residual flooding.

- **Vegetation Management In Downstream Creeks**: Removal of aquatic weeds from the main creek systems draining the downstream parts of the study area.

- **Improved Water Management**: Removal of irrigation supply water from the natural watercourses in the study area, as a means of reducing aquatic weed growth and hence improving flood passage.

- **Groper Creek Helicopter Landing Pad**: Construction of a helicopter landing pad in Groper Creek township to enable evacuation of residents during large floods.

These options were evaluated in conjunction with existing risk management initiatives, such as Council’s Flood Warning System, and the activities of the Burdekin River Improvement Trust.
A Risk Management Plan has been developed that is a combination of continuation of existing measures, and implementation of a number of new strategies mainly in the form of non-structural measures. In general, structural options appear to have little potential to provide cost-effective improvements to either river or local flooding. The elements of the Plan include:

1) Development and implementation of a Vegetation Management Plan for the lower Burdekin River and the main watercourses draining the Home Hill catchment area. For the Burdekin River, this Plan will aim to identify areas where vegetation is impinging on the flood capacity of the River, and identify appropriate means of reducing vegetation growth in these areas. Such areas will be limited to vegetation growth within the River channel, and will not include the river banks.

For the watercourses draining the Home Hill area the Plan will, in the short term, aim to establish a “flood corridor” by reducing weed growth through the appropriate use of herbicides and mechanical weed harvesting techniques. Strategies for the long term management of aquatic weed growth will also need to be developed. These strategies will need to be integrated with improvements in water management outlined in 2) below. **Estimated Capital Cost: $25,000 (plan development only).**

2) An ongoing program of removal of irrigation supply water from natural flood watercourses should be jointly developed by Council, the South Burdekin Water Board, and regulatory agencies. Whilst it is recognised that practical difficulties and costs will limit the capacity to undertake works in some areas, further detailed examination of this issue is required to identify areas where works can proceed. **Estimated Capital Cost: $50,000.**

3) Designation of “Flood Pathways” under Council’s new Planning Scheme, to provide a means for Council to control significant development in these areas. This is important for the preservation of waterway capacity, and the ability for river floods to pass through the Home Hill area with minimum impact. **Estimated Capital Cost: nil.**

4) Acquisition of easements along unsecured reaches of Kidby and Ford’s Gullies, extending downstream to the junction of those watercourses. This will provide ongoing secure drainage pathways for the Township of Home Hill. **Estimated Capital Cost: $50,000.**

5) Continued support and development of the existing Flood Warning System established by Council, including ongoing review of the performance of the system, and a program of community awareness of flood risks and response arrangements. **Estimated Capital Cost: nil.**

6) Establishment of improved evacuation arrangements for the Groper Creek community during times of large river flooding. This should include the establishment of an elevated helicopter landing pad in Groper Creek for the emergency evacuation of residents during severe floods. **Estimated Capital Cost: $50,000.**
7) Continued support for the activities of the Burdekin River Improvement Trust in identifying areas for potential river breakout, and undertaking works to reduce the risks of new breakouts occurring. Estimated Capital Cost: nil.
1. **Introduction**

During the last century severe flooding problems have threatened and affected the floodplain surrounding the township of Home Hill, causing extensive damage to properties and significant agricultural losses.

As part of the “National Disaster Risk Management Studies Program”, Sinclair Knight Merz was commissioned by the Burdekin Shire Council to undertake a detailed investigation of the flooding and drainage characteristics over the Burdekin River floodplain, in the Home Hill area.

The primary aims of this flood study were to:

- improve the understanding of flooding and drainage behaviour over the study area;
- identify the main risks to human life and property damage due to major flooding in the township of Home Hill;
- establish flood mitigation measures and strategies that address and reduce the risks of flooding, and
- recommend and develop a Flood Risk Management Strategy Plan.

This document details the methodology developed for the detailed investigations of the flooding and drainage characteristics, and outlines the results of the analysis.

An Advisory Group supervised the completion of the project and provided valuable input during the formulation of the proposed mitigation strategies, through an on-going process of consultation with Sinclair Knight Merz during the study. The Advisory Group included:

Chief Executive Officer – Mr. G. Webb
Shire Mayor – Cr. J. Woods
Chairman of Works – Cr. M.E. Parison
Shire Councillors – Crs D.R. Jackson, L. Loizou and L.R. Davies
Director of Works – Mr. T. Williams
Technical Supervisor – Mr. G. Keane
Manager South Burdekin Water Board – Mr. Bill Lowis
Design Office Manager – Mr. G. Pappalardo
Counter Disaster Rescue Services – Mr. M. Fleming and Mr. P. McAlonan

**SINCLAIR KNIGHT MERZ**
C.S.R. Limited – Mr. Ian Ballard
Canegrowers Association – Mr. K. Mann and Mr. C. Canavan
SunWater – Mr. P. Wheeler
Department of Natural Resources and Mines – Mr. R. Shaw

Computer modelling outcomes, discussions with stakeholders and the local community, and a general assessment of costs and benefits resulted in the identification of the most effective flood mitigation measures. These have been combined into an integrated flood risk management plan.
2. Catchment Description

2.1 The Study Area

The region of interest extends eastwards from “The Rocks” to the coast of Upstart Bay and covers approximately 450 km$^2$, as shown on Figure 2-1.

The town of Home Hill is the major urban centre located on the right bank of the River delta, with a population of approximately 3,000 urban and 1,600 rural residents. The area also includes the small coastal settlement of Groper Creek, home to several permanent residents and “weekenders”.

The section of the Burdekin River included in this area is part of the Burdekin River delta, and extends from a locality called “The Rocks”, where the river is confined on both sides by Mt. Kelly and Stokes Range, to the coast downstream of the Groper Creek area.

The Burdekin River appears to have been reasonably stable over recent history with no evidence of recent changes to its course. The river alignment is fairly straight between the major bend at The Rocks and the next bend downstream of the Burdekin River Bridge. Below this last bend the river is broader and less well confined.
Old courses of the river are still represented in the Burdekin River floodplain and continue to act as distributary channels during the more extreme floods. The main distributaries are located on the left bank (Sheepstation Creek, Hughes Creek, Kalamia Creek, Plantation Creek and the Anabranch). The Stokes Creek breakout is located on the right bank upstream The Rocks and flows through the Warren’s Gully system, past Inkerman through Yellow Gin Creek into Upstart Bay.

The main area of interests for this study is the floodplain on the right bank of the Burdekin River. This country is predominantly cane land, with the exception of the urbanised centre of Home Hill, and is relatively flat and low-lying. The study area is drained by a series of natural watercourses, consisting of several intermittent creeks / lagoon systems, artificial irrigation channels, waterholes and swamps. In recent years, these systems have become overgrown with aquatic weeds, particularly cumbungi and water hyacinth.

Three major drainage systems constitute the main floodwater carriers through the study area, as shown on Figure 2-2.

Figure 2-2: Major Creeks and Irrigation Channels

The first system consists of two major natural depressions (Kidby and Ford’s Gully) collecting separately a large proportion of Home Hill’s stormwater outflows before joining into a low
capacity channel running in a south-east direction, parallel to the Burdekin River. This drainage course is separated from the river by a natural levee system.

The central area of the floodplain is characterised by several interconnected natural and artificial drains (Central Diversion, Iyah Creek, Mather’s Lagoon Diversion, Charlie’s Creek Diversion, etc.) flowing through very low country in an easterly direction. This creek system carries a large amount of the total floodwaters crossing the floodplain and it ensures the distribution of water for irrigation purposes to the adjacent cane farms. This system eventually discharges to Upstart Bay via Merryplain and MacDonald Creeks, upstream the Groper Creek area.

A third main creek system extends from the Stokes Creek breakout to the Warren’s Gully irrigation channel on the west of the Bruce Highway. It links with the Alma Creek – Yellow Gin Creek system on the east-side of the Highway around Mount Inkerman and Mount Alma. This is a major corridor for the passage of breakout flows from the Burdekin River.

2.2 Description of Flooding

The Burdekin River has a very large catchment of about 130,000 km$^2$. Because of this large catchment area, storm events in different parts of the catchment can produce floods at the outlet, and the distribution of rainfall over the catchment can affect the flood hydrograph, causing localised or widespread outflows from the river onto the floodplain. Appendix A shows the Burdekin River catchment.

Flooding characteristics in the study area during major flood events are quite complex, with a number of different interacting flood processes. The main flood patterns of the study area are:

- Regional River Flooding
- Local Catchment Flooding

Both types of flooding are of concern and have been assessed independently, as part of this project.

Storm surge flooding is being assessed under a separate risk management study being coordinated by Burdekin Shire Council.

2.2.1 Regional River Flooding

Due to its proximity to the Burdekin River, a large proportion of the study area is inundated during major floods, with floodwaters generally overtopping and breaking the riverbanks and flowing over the floodplain.
Floods with an average annual recurrence interval up to about ten years are totally contained within the banks of the main river channel, with only larger events breaking out of the banks of the river. Some areas of relatively high ground remain above flood levels, but these are quite limited.

Several locations were identified as past and/or future potential breakouts from the river causing widespread inundation on the right floodplain.

**Stokes Creek**

Stokes Creek outlet, located just upstream The Rocks, is a natural low-point in the riverbank. Floodwaters flow over the existing river bank, run parallel to the river along Warren’s Gully irrigation drain, reach Fowlers Lagoon at Home Hill Aero Club and spread to the south-east of the study area over a natural swampland depression. Floodwaters subsequently separate into two main flowpaths:

- The first flowpath re-enters the South Burdekin Water Board Area through Alma Creek west of the Bruce Highway, then overtops the Bruce Highway flowing north of Mount Inkerman along Alma Creek. Finally, floodwaters split between Sandy Creek (flowing north towards the Groper Creek area) and Alma Creek (flowing east through Mount Alma and Mount Inkerman towards the Upstart Bay).
- The second flowpath follows the Munro’s Lagoon system near Inkerman, and flows south of Mount Inkerman along Saltwater Creek and finally reaches the Upstart Bay.

**Lago’s Breakout**

Approximately 2 km upstream the Inkerman Bridge, floodwaters overtop the existing low right bank of the Burdekin River (Lago’s Breakout), cross the Home Hill - Kirknie Road following a natural depression and then separate into two main flowpaths, just upstream the township, west of the Bruce Highway:

- The first flowpath splits between Kidby’s Gully towards the east-side of the township (near the TAFE college), and Ford’s Gully through Barton’s Caravan Park, flowing south-east towards the State High School Oval. This flowpath causes widespread inundation throughout the entire township.
- The second flowpath follows a southern direction towards the Water Tower and the natural deep depression called Gardner’s Lagoon, then flows parallel to the Railway line and reaches the Porter’s Lagoon Irrigation Drain overtopping the Highway and Railway lines. Floodwaters then run east along Porter’s Lagoon and Mather’s Lagoon Diversions, to McDonald Creek and Groper Creek area.
Tapiola’s Breakout
Approximately 2km downstream the Inkerman Bridge floodwaters spill over the right river bank, inundating local farms and flowing in an easterly direction along the Kidby and Ford’s Gully system.

Except for some localised minor man-made levees protecting local cane farms, the entire downstream section of the Burdekin River right bank is generally quite low. In addition, the main river channel is naturally elevated with respect to the adjacent broader floodplain. During major floods, waters can easily overtop the river banks and cause widespread inundation over large areas of the adjacent floodplain. The relative levels of the river and the floodplain are represented in Figure 2-3. The cross-section shown follows the main highway from the river across the study area.

Figure 2-3: Home Hill Floodplain Longitudinal Profile

2.2.2 Local Catchment Flooding
In addition to the major Burdekin River floods and breakouts, significant local flooding also occurs regularly in the Home Hill area, caused by local rainfall events.

The local catchment flooding is generally less significant than the river outflows, both in terms of flow rates and flood levels. However, local flooding has been identified by local landholders as an issue for concern, who believe this flooding to be occurring more frequently than in the past.

In particular, the areas that have proven to be susceptible to regular local flooding are:

- On the eastern side of the study area, Groper Creek is subject to regular flooding due to the coincidence of local catchment flooding, storm surge and or king tides. However, according to the local community, the frequency of flooding produced by local rainfall has increased during the recent years, causing property damage and cutting off road access. Increased local
flooding is mostly attributed to the uncontrolled growth of aquatic weeds in natural watercourses that drain the downstream part of the study area.

- At the confluence of Kidby and Ford’s Gully, significant farmland is subject to flooding even during small local rainfall events. It is believed that the existing downstream drainage channel capacity is not adequate to convey the combined flows from both gullies.

- The junction of Mt Alma Road, Groper Creek Road and Fry’s Road represents a crucial intersection where the three major creek and drainage systems join together, conveying the total combined waters from the entire floodplain. Inadequate waterway capacity, irrigation and crop embankments, conflicting use of waterways and vegetation growth are all factors that contribute to increased local flooding.

2.2.3 Historical Floods

The Burdekin River has a long history of flooding, and there are a number of recent floods that have had substantial local impact. Significant floods have occurred in the Lower Burdekin floodplain in 1927, 1940, 1946, 1958, 1974, 1988 and 1991. Only four of these have resulted in significant inundation of the right floodplain area (1940, 1946, 1958, 1991).

The historical floods selected and analysed in detail are discussed below. Very few records are available on the nature and damage caused by those past floods. Farm damages records are non-existent. Recorded peak discharges at particular locations (river gauging stations) and some flood level data and maps are limited.

1940 Flood

The 1940 flood occurred in April, with a recorded peak discharge of 38,300 m$^3$/s at Home Hill.

At that time, the 1940 flood was described as the worst in living memory, with over 340 acres of farming land severely eroded or silted. Riparian farms were most severely affected, with more than 250 acres of assigned cane land permanently damaged in the surroundings of the Inkerman Mill area.

No flood levels or anecdotal flood maps are available for this flood event.

1946 Flood

The 1946 flood is the largest event for which recorded data were available, with a peak discharge of 40,400 m$^3$/s recorded in March at the river gauge at Home Hill.

Some information on flood damage in the Inkerman Mill area is available in a report compiled by the Mill Suppliers Committee. The flood was generally as high as the 1940 flood, but it was more prolonged. A significant aspect of this flood is that floodwaters broke the left bank of the Burdekin
River at Gladys’ Lagoon (upstream of Clare) and joined up with floodwaters from the Haughton River, causing widespread flooding in very large areas in the north side of the study area.

Flood information was reported and summarised into a plan showing flood contours and flood levels at three locations. This represents limited information only and it does not extend over the whole study area.

**1958 Flood**

Two floods occurred in 1958, the first one at the end of February with a peak discharge of approximately 25,000 m$^3$/s, and the second one recorded at Clare in early April, with a peak discharge of 36,000 m$^3$/s.

While this flood was slightly smaller than the 1946 event, it can be considered as another major event for the area of concern (the third largest event in the combined records of Clare / Home Hill), producing extensive areas of inundation in and around Home Hill.

The significant feature of the April 1958 flood was that it rose and fell relatively quickly, and consequently substantial areas of Home Hill and Ayr were flooded and affected by heavy deposition of sand and debris. Cane farm erosion was particularly severe as the flood coincided with the commencement of the cultivation period.

A number of flood levels were recorded in the river and in the southern floodplain area and crude inundation maps were prepared from those observations. Although these maps do not correspond to an accurate representation of the area, they do provide useful information for model calibration purposes.

Additional data from the Burdekin Shire Council was summarised into inundation maps with some observed levels in the urban centres of Ayr and Home Hill. The April 1958 flood inundation map for the township of Home Hill is provided in Appendix B, Figure B 1 and is discussed more in detail in Section 4.4.2.

**1991 Flood**

The February 1991 flood is the most recent major flood event with a peak discharge of 29,800 m$^3$/s recorded on 3 February 1991 at the Clare stream gauging station.

More extensive data on flood patterns and levels is available for this flood event, due to its recent occurrence. Flood inundation maps for the townships of Ayr, Brandon and Home Hill were prepared by the Burdekin Shire Council. From the Home Hill 1991 flood map, presented in Appendix B, Figure B 2, it can be observed that this flood was sufficiently large to cause
extensive overflows from the river. Although quite different in terms of peak discharges, the 1958 and 1991 floods resulted in similar areas of inundation in and around Home Hill.

2.3 Existing Mitigation Measures

2.3.1 Burdekin River Improvement Trust
The Burdekin River Improvement Trust is responsible for river maintenance works along the lower Burdekin River. This includes, from time to time, works on the river banks to repair flood damage. These works are regarded as an important part of ensuring that new river breakout points are not allowed to occur, and to maintain the condition of existing breakouts locations. In terms of managing the risks of flooding, particularly for small to medium sized river floods, the activities of the River Trust play an important part in maintaining local awareness of the risks of river flooding.

2.3.2 Flood Warning System
Over recent years, Burdekin Shire Council has established an extensive flood warning system for the Burdekin River catchment. The system has been developed in conjunction with the Bureau of Meteorology (BoM) and is based on the BoM URBS real time forecasting hydrologic models.

Monitoring of rainfall in the catchment is undertaken at a number of locations. Real-time flood modelling and flood height prediction is possible and procedures have been established with BoM to ensure coordinated monitoring and flood prediction during flood events.

Outputs from the flood prediction system are integrated with response protocols established for the local Counter Disaster Committee, to ensure appropriate response actions are implemented during times of flood.

The flood warning system is a vital part of Council’s flood risk management strategy.
3. Hydrologic Review and Modelling

Two different hydrological approaches were selected to investigate the two types of flooding of the study area (river flooding and local flooding). A detailed investigation of historical data and a flood frequency analysis review were conducted for the regional river flooding. A hydrologic modelling approach was selected for the generation of design events for the local catchment flooding.

3.1 Regional River Hydrology

3.1.1 Introduction

Several studies have been previously undertaken to investigate flooding and associated issues in the area of interest and the surrounding region, in particular:

- Ayr – Home Hill Bypass Flooding Investigation, Kinhill, 2000;
- Strategic Plan for Management and Improvement of the Lower Burdekin River, Kinhill Cameron McNamara, 1996;

Relevant information concerning the main characteristics of the river hydrology was extracted from the above reports.

3.1.2 Flood Frequency Analysis

The hydrology of the Burdekin River has been analysed on a number of occasions over the years, but the records available are only a small portion of the hydrological history of the region. Various estimates of the recurrence intervals, especially of the larger floods, can show significant uncertainties in the results. This can be due to a number of factors, such as the nature of the recorded data, the different sources of information, datum conversions, and changes in the river morphology and different estimate methods.

Due to the very large catchment area of the Burdekin River (130,000 km²) and the long records of historical stream flow data, a flood frequency analysis was considered the most appropriate approach for estimating flood flow rates for different recurrence intervals for the lower Burdekin River. This approach is based on standard procedures from “Australian Rainfall and Runoff” (1998).

There are a number of stream gauging stations located in the lower reaches of the Burdekin River which have sufficient data useful for an analysis of flooding and flow duration. In particular, two
long-term stream gauges (Clare and Home Hill) provide flow data for more than 80 years (from 1920 to date) when combined together.

Kinhill (2000) undertook a frequency analysis using this data, whereby the annual series of recorded peak discharges of the two combined stations were fitted with the Log-Pearson III distribution.

The results of the analysis are shown in **Table 3-1**.

<table>
<thead>
<tr>
<th>ARI (years)</th>
<th>Clare / Home Hill (m$^3$/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>15,800</td>
</tr>
<tr>
<td>10</td>
<td>24,800</td>
</tr>
<tr>
<td>20</td>
<td>35,800</td>
</tr>
<tr>
<td>50</td>
<td>53,600</td>
</tr>
<tr>
<td>100</td>
<td>69,600</td>
</tr>
</tbody>
</table>

Several factors influencing the sensitivity of the analysis were analysed before adopting the above results, in particular:

- The two stations of Clare and Home Hill were combined and considered as one single station in the flood frequency analysis. In fact, these stations are approximately 20 km apart. It is important to note, however, that during large floods major breakouts occur between the two stations (Plantation Creek and Stokes Creek), which would reduce peak flows at the downstream gauging station and affect the results of the analysis. Local catchment inflows between the gauging stations could also affect results. Both of these factors, however, are considered insignificant when compared to the peak flow rates in the main river channel, and will have little affect on the flood frequency analysis results.

- Another factor to consider is the potential impact of the Burdekin Falls Dam on the results of the frequency analysis. The dam was completed in 1987 and only one major flood has occurred since its completion (1991). It could be expected that the dam has an effect on the river flows, however, as its capacity is significantly smaller than the total runoff volume of large flood events it is unlikely that the dam could influence the records of large flood. For example, the 1991 flood event had a total volume of $36.9 \times 10^6$ ML, while the Burdekin Falls Dam has a capacity of $1.86 \times 10^6$ ML. As a result, it is expected that the Burdekin Falls Dam will have only small or insignificant impact on the major floods.

- It is commonly recognised that different sources of the same information might generate anomalies or inconsistencies. The Burdekin Shire Council, in collaboration with the Bureau
of Meteorology (BoM), has developed an advanced flood warning system based on recorded peak heights at key gauging stations along the river. Recorded peak heights are converted into peak discharges by using the rating curve (Q/h relationship) for that particular gauging station. Rating curves are available from the BoM at Millaroo, Dalbeg, Clare and Inkerman Bridge (Home Hill). For those same locations DNRM provides slightly different rating curves based on recorded peak discharges of several historical flood events. A slight difference in the rating curve might generate significant differences in the predicted peak discharges for the same location.

For the purposes of this study, the consistency between the recorded peak heights from BoM and the recorded peak discharges from DNRM was analysed. Some anomalies were found due essentially to different extrapolations of the rating curves for larger events.

Notwithstanding the above, the Kinhill (2000) represents a reasonable evaluation of the flood frequency characteristics of the lower Burdekin River and the peak discharge values shown in Table 3-1 were adopted for this study.

This analysis also indicates that:

- the 1958 flood is equivalent to a 20 year ARI design event (36,000 m$^3$/s);
- the 1991 flood can be related to a 10-20 year ARI design event (29,800 m$^3$/s);
- the largest recorded flood (1946) is smaller than the 50 years ARI design event.

An estimate of the Probable Maximum Flood (PMF) peak discharge was derived from a recent review conducted by Sinclair Knight Merz of the hydrology of the largest northern Queensland dams. The aim of this review was to estimate a relationship between the 100 year ARI event and the PMF peak discharges, using the catchment size, the topography of the area and the location as the most representative variables. For the Burdekin River catchment, an average ratio of 7 was adopted, which corresponds to an estimated peak discharge of approximately 487,000 m$^3$/s for the PMF event.

Since the flood frequency analyses provides only peak discharges, the flood hydrographs for different design events were extracted from recorded flood hydrographs of the larger historical floods. The shape adopted for the flood hydrograph was taken from the April 1958 flood, which was selected as the most representative for the Burdekin River. The design floods were scaled to match the shape with the calculated flood peak discharges. Figure 3-1 presents the hydrographs for the 20, 50 and 100 year ARI design events generated for this study.
The flood frequency analysis was also used to confirm some flooding patterns over the study area. Outflows from the Burdekin River were adopted according to the following occurring sequence:

- At 4,200 m$^3$/s approximately, outflows occur via the Anabranch on the left bank, upstream Rita Island;
- At 14,000 m$^3$/s approximately, outflows occur via Stokes Creek and Warren’s Gully (right bank);
- At 25,500 m$^3$/s approximately, outflows occur over the Burdekin River banks downstream from the Rocks (both sides);
- At 28,000 m$^3$/s approximately, outflows occur through Gladys Lagoon towards the Baratta Creek system, upstream Clare (left bank).

These breakout discharges were used as valuable information during the hydraulic model calibration task.

### 3.2 Local Catchment Hydrology

#### 3.2.1 Overview

A hydrological model was established to analyse the local catchment hydrology over the study area. Hydrological modelling provides a robust means of estimating run-off rates for various rainfall events, using local catchment physical characteristics.
3.2.2 Model Description
The objective of the local hydrologic analysis was to develop a reliable model capable of predicting design flood hydrographs from design rainfall events. The hydrologic runoff routing modelling software XP-RAFTS was selected for this study. This program was originally developed by Willing & Partners and the Snowy Mountains Engineering Corporation in 1974 and is widely used throughout Australia.

XP-RAFTS requires the user to subdivide the watershed into a number of subcatchments. For each of these subcatchments, information on slope, roughness and area are entered into the model. Non-linear runoff routing procedures are used to develop a stormwater runoff hydrograph from either a recorded rainfall event or a design storm using Intensity-Frequency-Duration (IFD) information together with storm temporal patterns derived from standard AR&R data. The main outcomes of the model are represented by local or total hydrographs at each selected catchment, which can be exported into a suitable format for further hydraulic modelling analysis.

3.2.3 Model Setup and Parameters
The total area modelled was approximately 350 km$^2$. The subcatchment boundaries were determined using available topographic information, Council plans, site inspections and discussions with Council officers, South Burdekin Water Board and local landholders. The selection of boundaries was influenced by the location of hydraulic structures, drainage lines, changes in catchment landuse and existing infrastructure network (road and railway lines).

Twenty-nine subcatchments were defined. The extent of the model, the subcatchment boundaries and the assumed routing lines are shown in Figure 3-2.
The parameters adopted for the hydrologic modelling of the Home Hill area were consistent with those used in other studies conducted within the region (Ayr Flood Study, SKM 2002).

Physical catchment parameters and additional data required for the hydrological modelling are discussed below:

**Percentage Catchment Impervious**

Most of the subcatchments were modelled as being 100 % pervious, except for the subcatchments 1 and 2 containing the township of Home Hill, which were modelled as partially impervious. Table 3-2 provides the breakdown between impervious and pervious areas for the subcatchments 1 and 2.

**Table 3-2: Calculated Pervious / Impervious areas**

<table>
<thead>
<tr>
<th>Subcatchment</th>
<th>Area Pervious (ha)</th>
<th>Area Impervious (ha)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1273.5</td>
<td>108.8</td>
</tr>
<tr>
<td>2</td>
<td>657.4</td>
<td>90.9</td>
</tr>
</tbody>
</table>
Subcatchment Slope
The slope of each subcatchment was determined using available contour information for the area. Generally, the topography of the area is very flat with many of the subcatchment slopes set at less than 1%.

Due to the extensive sugarcane cropping carried out in the study area, a Manning’s roughness of 0.08 was adopted for all subcatchments, which reflects a relatively high hydraulic roughness of the catchment areas. A value of 0.014 was set for the impervious subcatchments representing the urbanised area within the township.

Link Parameters
Each subcatchment is connected to the other in the model using a link. The model uses these links to lag the flood hydrographs from one node to the next, by knowing the travel time and the velocity of the flows.

Guidance was obtained from previous studies in determining the velocity to adopt for computing the links lag. For overland flow, a general value of 0.5 m/s was adopted to calculate the length of time that it would take for the hydrograph to move from one subcatchment outlet to the next.

Figure 3-3 details the configuration of the RAFTS model developed for the area of interest, showing the nodes and the links network.
Rainfall Data
Design storm hydrographs were generated using local Intensity-Frequency-Duration (IFD) information extracted from Volume 2 of AR&R (1987). IFD data for Home Hill is presented in Table 3-3. The storm temporal patterns are automatically selected by the model from in-built standard temporal pattern tables (AR&R), once the appropriate Zone is specified. The Home Hill floodplain is situated within the Zone 3 pattern.
Rainfall Losses

An Initial/Continuing loss rate method was used to estimate rainfall excess for the catchment. These losses are used to compute the runoff generated from the catchment in response to a rainfall event, and include two main components:

- **Initial Loss**: a fixed loss value (mm) accounting for infiltration effects, deducted from the rainfall data prior to the occurrence of surface runoff.
- **Continuing Loss**: a constant loss rate (mm/h) deducted from the rainfall data over the duration of the storm.
The loss rates adopted for this study were based on the results of several previous studies carried out on catchments within the Burdekin River area. As a consequence, an Initial Loss of 40 mm and a Continuing Loss rate of 2.5 mm/hr were applied, as used in the Ayr Flood Study (SKM, 2002).

The model main parameters used for this study are summarised in Table 3-4.

Table 3-4: Hydrologic Model Parameters Summary

<table>
<thead>
<tr>
<th>Model Parameter</th>
<th>Value Adopted</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roughness</td>
<td>0.08</td>
</tr>
<tr>
<td>Initial Loss</td>
<td>40 mm</td>
</tr>
<tr>
<td>Continuing Loss</td>
<td>2.5 mm/hr</td>
</tr>
<tr>
<td>Link Velocity</td>
<td>0.5 m/s</td>
</tr>
</tbody>
</table>

Based on the above parameters and conditions, four design Average Recurrence Interval (ARI) storms were generated. These were the 5, 20, 50 and 100 year ARI design events.

3.2.4 Critical Duration Analysis

A critical duration analysis was conducted in order to determine the storm duration that causes the greatest peak runoff rate from the catchment. A series of storm events for each ARI and a number of storm durations (12 to 72 hours) were run and compared to find the event which produced the maximum discharge at the system outlet. The results of this analysis are presented in Table 3-5.

Table 3-5: Peak Discharges and Critical Storm Durations

<table>
<thead>
<tr>
<th>ARI (year)</th>
<th>Peak Discharge (m³/s)</th>
<th>Critical Storm Duration (hrs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>1211</td>
<td>24</td>
</tr>
<tr>
<td>50</td>
<td>977</td>
<td>24</td>
</tr>
<tr>
<td>20</td>
<td>757</td>
<td>36</td>
</tr>
<tr>
<td>5</td>
<td>442</td>
<td>36</td>
</tr>
</tbody>
</table>

The critical duration events were used in the subsequent analysis.

3.2.5 Model Calibration and Validation

No historical data was available for calibration purposes within the local catchment area. Validation of the RAFTS results was performed using the basic Rational Method verification process from AR&R. The 100 year ARI design event was used for this comparison. This assessment showed a reasonable match between the predicted peak discharges.
4. Hydraulic Analysis

4.1 Overview
An extensive hydraulic analysis was undertaken to identify the flooding characteristics of the study area and to establish the key issues in relation to flood risk management. Based on an understanding of local flooding characteristics, an hydraulic model was established to simulate flooding patterns. This model was used to assess the impact of mitigation measures.

4.2 Hydraulic Model Description
The topographic characteristics of the Burdekin River floodplain and the complex nature of flooding described in Section 2.2 require a two-dimensional hydraulic modelling to adequately describe the flood processes involved. These advanced hydraulic modelling techniques are capable of simulating simultaneously main channel hydraulics, overland flows, river breakouts, natural drainage flowpaths and complex creek and lagoon system interactions.

MIKE21, a software package developed by the Danish Hydraulic Institute, was selected as the suitable hydraulic modelling software for this study. MIKE21 is a fully hydrodynamic two-dimensional modelling system capable of simulating flood flow behaviour within extensive floodplain areas. It can also be used to account for tidal dynamics and the effects of hydraulic structures such as bridges and culverts.

The hydrodynamic module (MIKE21-HD) uses a finite difference scheme to solve the integrated equations of the St Venant for the conservation of continuity and momentum. This numerical resolution method has two main implications in the model setup and development:

- the model is based on a constant grid spacing in “x” and “y” directions;
- the model area must lie in a rectangular grid, i.e. it must be rectangular in shape.

As a direct consequence, the success of a particular application of MIKE21 is firmly dependent upon the selection of a suitable model area, combined with the choice of the most appropriate grid spacing to represent the data available. These requirements influence directly the setup of the model parameters and are discussed in detail in the following section.

4.3 Model Setup and Parameters
Setting up a hydraulic model requires the combination of topographical surveys and several hydraulic parameters describing water depths or fluxes at the boundaries of the study area, as well as roughness’s values representing channel and floodplain resistance to flows. The selection of
adequate parameters is the most important task in the modelling process and it influences the model performance and its final results.

4.3.1 Survey and Topographic Data
The Burdekin Shire Council commissioned AAM Surveys for this project to collect accurate terrain data covering the Home Hill floodplain.

Airborne Laser Scanning (ASL) data was acquired in June 2002. GPS base station ground check points were used to assess the accuracy of ASL data. In areas not obscured by vegetation very good agreement existed between ALS data and GPS field information. Several interpolation algorithms were applied over the areas covered by dense vegetation (such as sugar cane), followed by extensive manual checking and editing to improve the quality of the terrain model. Further thinning procedures were used to aggregate points within a 10m horizontal radius and within ±0.10m vertically. The aims of these procedures were to improve data handling by reducing the total number of ground points extracted from ASL techniques, without decreasing the initial data accuracy.

A final Digital Terrain Model (DTM) was supplied to Sinclair Knight Merz in November 2002 and consisted of approximately 4.5 million of ground points on irregular intervals, with an average vertical accuracy of ±0.15 m.

The original extent of the study area selected by the Burdekin Shire Council included only the section of the floodplain on the right bank of the Burdekin River. In order to conduct a comprehensive and correct hydraulic analysis of the region, the area of interest was extended to include the main river channel and all major breakouts both on the left and on the right riverbanks. Additional topographic data was collected from other sources, in particular:

- The area between Ayr and Home Hill, east of Bruce highway including the main river channel, was acquired from AAM Surveys as part of the concurrent Storm Surge Project. Terrain information was supplied in conjunction with the original data for the Home Hill floodplain.
- The area between Ayr and Home Hill, west of Bruce Highway, was documented by the Department of Main Roads for a previous project (Ayr – Home Hill Bypass Flooding Investigation, Kinhill, 2000). This information was generated by photogrammetry from 1:10,000 scale aerial photography captured in November 1996.
- Topographic maps in digital format (1:100,000) were acquired by Sinclair Knight Merz and used for a variety of purposes in this study, including as background for modelling and result preparation. These maps show 5m contours, which were digitised and used as additional source of information to complete the missing terrain data in the area of the Stokes Creek/Warren’s Gully system. As mentioned in Section 2.1 this system represents one of the
main breakouts and it was fundamental for the purposes of this study to include this natural depression in the modelling process.

An additional GPS ground survey was undertaken by Sinclair Knight Merz in December 2002 to acquire cross sectional data of Stokes Creek at the confluence with the Burdekin River. Further field inspections were also undertaken to pick up features such as size and levels of culverts and bridges, road crossings and several drain characteristics.

Figure 4-1 presents the final study area extent, lying in a rectangular grid as required by the hydraulic model.

4.3.2 Manning’s Roughness
River and floodplain roughness values over the study area were primarily based on cadastre data and land use classification supplied by the Burdekin Shire Council. An aggregation procedure was
applied to the original land use data and a final classification into six categories was adopted. A range of Manning’s coefficients was assigned to each land use category based on site visits, past experience from previous flood studies and extensive consultation with Council and South Burdekin Water Board.

The range of roughness values used in the hydraulic modelling process is listed in **Table 4-1**

<table>
<thead>
<tr>
<th>Category</th>
<th>Description</th>
<th>Min Value</th>
<th>Max Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Urban Areas / Roads</td>
<td>Concrete, asphalt, gravel</td>
<td>0.015</td>
<td>0.020</td>
</tr>
<tr>
<td>Main River</td>
<td>Regular cross section, no major obstructions</td>
<td>0.025</td>
<td>0.030</td>
</tr>
<tr>
<td>Natural Creeks and Depression Systems</td>
<td>Winding, trees obstruction, thick grass, dense vegetation</td>
<td>0.040</td>
<td>0.150</td>
</tr>
<tr>
<td>Land</td>
<td>Grass, scattered vegetation</td>
<td>0.025</td>
<td>0.030</td>
</tr>
<tr>
<td>Grazing Land</td>
<td>Pasture, scattered bush</td>
<td>0.030</td>
<td>0.040</td>
</tr>
<tr>
<td>Agricultural Land</td>
<td>Cane farm, dense vegetation</td>
<td>0.080</td>
<td>0.160</td>
</tr>
</tbody>
</table>

For the agricultural land category (cane farms), one broad range of roughness values was selected and applied to the entire floodplain. This range, which varies from 0.080 to 0.160, accounts for local differences in farm practice, and variable cane growth due to crop rotation.

This range of values was used to assess the sensitivity of the water depths predicted by the hydraulic model to the different vegetation cover resistances, and to optimise model results in the calibration process.

### 4.3.3 Boundary Conditions

The description of the water levels and flows at the open boundaries of the model area is another fundamental task in the model establishment. The better the boundary conditions the better the results, and the fewer the instability problems during the computational process.

Generally, the most common combination of boundary conditions consists of:

- specifying discharge hydrographs representing flow conditions at the upstream ends of the hydraulic model;
assuming water levels showing the hydrodynamic conditions at the bottom ends of the model.

Boundaries express the model input and output conditions and during the calibration process they are the link between the hydraulic model and the recorded historical flow data. Similarly, during hydrological model calibration, they are the points at which modelled and observed events are compared.

4.4 Regional Flood Model

4.4.1 Regional Model Establishment

MIKE21 was used to model overflows from the Burdekin River (“Regional River Flooding”).

The Digital Terrain Model covering the study area shown in Figure 4-1, was imported into a GIS platform and converted into a regular grid to form the basis of the hydraulic model.

The grid resolution selected for this application was based on a constant 50m spacing in “x” and “y” directions, which corresponds to approximately 350,000 computational points over the entire study area. Figure 4-2 presents the terrain model of the regional river flooding model. This level of DTM resolution was considered appropriate, given the broad scale of regional flood events.
Roughness values were imported into MIKE21 by specifying a map similar to the model bathymetry, with a resistance value for each grid point in the model area. The cadastre layout classified into six landuse categories was converted into a 50 m grid, where each grid point within the same landuse class was assigned to the same roughness value. The final Manning’s coefficients adopted for the regional flooding modelling are reported in Table 4-2. This is the set of parameters that provided the best model results during the calibration process.

Table 4-2: Adopted Manning’s Coefficients

<table>
<thead>
<tr>
<th>Category</th>
<th>Description</th>
<th>Manning’s ‘n’</th>
</tr>
</thead>
<tbody>
<tr>
<td>Urban Areas / Roads</td>
<td>Concrete, asphalt, gravel</td>
<td>0.020</td>
</tr>
<tr>
<td>Main River</td>
<td>Regular cross section, no major obstructions</td>
<td>0.030</td>
</tr>
<tr>
<td>Natural Creeks and Depression Systems</td>
<td>Winding, trees obstruction, thick grass, dense vegetation</td>
<td>0.150</td>
</tr>
<tr>
<td>Land</td>
<td>Grass, scattered vegetation</td>
<td>0.030</td>
</tr>
<tr>
<td>Grazing Land</td>
<td>Pasture, scattered bush</td>
<td>0.040</td>
</tr>
<tr>
<td>Agricultural Land</td>
<td>Cane farm, dense vegetation</td>
<td>0.160</td>
</tr>
</tbody>
</table>

Discharge hydrographs of selected historical floods and of the generated design events were used as boundary conditions at the upstream ends of the model, located approximately 2 km upstream of The Rocks. A constant tailwater level set at Mean Sea Level (0.044m AHD) was initially applied to the downstream ends to simulate a static tidal boundary, so that the boundary condition has no influence over the model results.

A timestep of 5 seconds was adopted during the hydraulic computation process. The simulation period, which is directly dependent on the duration of the input flood hydrographs, was set at six days for this application. The total model runtime for each simulation was approximately 24 hours.

4.4.2 Model Calibration

Model calibration for regional flooding was based on two major historical events; the 1958 and the 1991 flood. For both events, recorded peak flows and river heights at Clare and Home Hill gauging stations were available, as well as some anecdotal information on flowpaths and indicative flood maps over the township precinct (refer to Appendix B). Very limited discrete flood level information was recorded for the 1958 flood and the reliability of these records is unclear. Observed flood levels are available for the 1991 event, but they cover only the area of the Home Hill township. A few breakout locations were confirmed with local landholders during a site visit as part of this study. No other data was available for the general floodplain.
Calibration of 2D hydraulic models is usually undertaken by variation of the roughness values so that modelled flood levels approximate observed or historical levels. By adopting the final set of Manning’s coefficients shown in Table 4-2, good calibration was achieved at the Inkerman Bridge gauging station, as shown on Figure 4-3. The maximum difference between recorded and predicted river heights was approximately only +0.20 m at the time of the peak.

The location of the major breakouts was correctly simulated by the model, confirming the consistency of the selected parameters and the robustness of the results.

A good match was also obtained between modelled and observed flood levels within and around the township of Home Hill. A comparison between the 1958 and 1991 observed flood levels and the model outputs at several key locations shows an average difference of approximately 0.1m – 0.2m in the urban area, as presented in the maps in Appendix C.
Generally, within the constraints imposed by the paucity of historical flood level information, it can be considered that the MIKE21 model provides a reasonable representation of the historical flooding patterns throughout the modelled area.

### 4.4.3 Regional Flood Model Results

The inflow hydrographs generated from the Flood Frequency Analysis (refer to Section 3.1.2) for the 20, 50 and 100 year ARI design events were run through the calibrated MIKE21 model to simulate flooding under these events.

Flood inundation maps showing peak water depths over the floodplain were produced for each design event modelled and are presented in Appendix D. The modelling indicates that any major flood of a magnitude equivalent to a 20 year ARI event or larger, causes widespread inundation over the entire floodplain, leaving only very few isolated dry areas. Figure 4-4 shows extent of flooding and peak water depths corresponding to a 20 year ARI flood event.

![Figure 4-4: Regional Flooding Inundation Map (20 year ARI)](image)

The flooding progression characteristics across the floodplain, described in Section 2.2.1, were reproduced by the MIKE21 model. When flows in the river reach about 14,000 m$^3$/s floodwaters start flowing through the Stokes Creek / Warren’s Gully system and move towards Inkerman
following the existing depression system. As river flows reach approximately 25,000 m³/s, breakouts occur at several locations along the right bank (Lago’s and Tapiola’s breakouts). Waters cross the floodplain in a south-east direction, flowing through the township of Home Hill, across the downstream floodplain and finally to the ocean, inundating the Groper Creek area and surrounding regions. Maximum modelled water depth in Groper Creek was approximately 3.0 m for the 20 year event, and 4.0 m for the 100 year event. These levels are based on a mean sea level, and as such, do not account for any storm surge effects.

Modelling indicates the township of Home Hill is subject to partial flooding during a 20 year river flood, with water depths raising up to a maximum of 0.8 m-1.0 m above ground. Floodwaters from the river flow through the urbanised area following the two existing natural depressions (Kidby and Ford) and leave some areas of relatively high ground above flood levels (west of Bruce Highway, in between the two gullies). For larger floods (i.e. 50, 100 year ARI) the entire township is inundated, with floodwaters crossing and overtopping the two main infrastructure lines (Bruce Highway and Railway) at several locations and inundating the west side of the town, where waters depths reach a maximum depth of 1.8m-2.0m above ground.

Given the relatively flat topography of the floodplain and the presence of several large natural swamps and depressions, the flood recession, after the peak, is relatively slow, with 2-3 days of residual inundation.

4.5 Local Flood Model
A second more detailed MIKE21 model was developed to investigate local flooding issues and to assess, in particular, the hydraulic performance of the existing drainage system in some localised areas within and downstream of Home Hill. The design events generated by the RAFTS hydrological model were run through the local MIKE21 hydraulic model to identify the areas at major risk and to provide a means of evaluating future improvement strategies.

4.5.1 Local Model Establishment
Given the more accurate level of detail required for this second hydraulic investigation, a finer grid resolution was selected for the local flood model. A constant 20 m grid spacing was considered appropriate for the purposes of this analysis. To limit the computational time involved, the model area was reduced to approximately 200 km², extending from 2-3 km east of Bruce Highway downstream to Groper Creek area, as shown in Figure 4-5.
The total number of grid points for the detailed hydraulic assessment was nearly 600,000 over the reduced modelled area.

The same landuse classification and roughness values used for modelling the regional flooding issues were applied to the local catchment hydraulic analysis. **Figure 4-6** presents the 20 m roughness map generated for this application.
For the purposes of this investigation, local flooding was analysed assuming no river overflows occur. This is considered a reasonable assumption, given the quicker response time of the local catchment compared to the river. It is acknowledged however that there is a possibility that both extreme local flooding and river overflows could coincide, however the probability of this is considered to be small.

As a consequence, the input boundary conditions selected for this analysis are based on a constant Burdekin River flow rate of 8,000 m$^3$/s, which is less than 1 in 2 year ARI flow. Downstream boundary conditions were set at Mean Sea Level (0.044 m AHD), as for the regional flooding model.

The discharge hydrographs predicted by the RAFTS model were considered as internal inflow conditions (“source points”) representing local and total subcatchment inflows over the modelled area. The centroids of each subcatchment located within the model area were selected as source points (refer to Figure 3-2).
A rainfall/runoff event duration of 4 days was considered appropriate for this analysis. A timestep of 5 seconds was used during the computational process. The total model runtime for each design event was approximately 60 hours.

4.5.2 Model Calibration

No historical flood data was available for calibration purposes within the local catchment area. Validation of the MIKE21 results was based on consultation with Council and the officers from the South Burdekin Water Board.

Without the availability of calibration data to provide further evidence to support significant variations in the model setup, it is considered that the local MIKE21 model provides an acceptable basis for prediction of peak water levels and flows throughout the modelled area.

4.5.3 Local Flood Model Results

The range of design events calculated by the RAFTS model was run through the local MIKE21 model. A series of flood inundation maps and velocity maps were produced for the 5, 20, 50 and 100 year ARI local design floods and are presented in Appendix E. An example of a flood inundation map for the 20 year ARI local event is presented in Figure 4-7.

Figure 4-7: Local Flooding Inundation Map (20 year ARI)
Peak water depths and peak velocities shown on these maps demonstrate that the local MIKE21 model is capable of reproducing the broad scale observed flooding characteristics caused by local rainfall events, as described in Section 2.2.2. In particular, it is possible to identify several major areas subject to regular flooding, as discussed below.

### Confluence of Kidby and Ford’s Gully

The combined Kidby and Ford’s Gully system appears capable of conveying floodwaters without causing major flooding problems up to a 50 year ARI design local event. The 100 year ARI event appears to cause several localised problems, especially along Kidby Gully.

The confluence of the two gullies is susceptible to flooding even for small local rainfall events. This reflects the poor drainage conditions downstream the confluence due to the combination of different factors, such as the natural topography, land development, and inadequate capacity of the drainage channel conveying the combined flows from both gullies.

### Downstream Area

As shown in Figure 4-7, the downstream section of the floodplain, from the junction of Mt Alma Road and Fry’s Road to the Groper Creek area, receives floodwaters from the three combined major creek and channel systems, which drain the entire floodplain. Drainage conditions in this area are poor, greatly hindering the passage of floodwaters. These conditions are a result of:

- loss of natural flood waterway due to land development, particularly sugar cane;
- excessive growth of aquatic weeds. This is a result of increased runoff and associated nutrients from the upstream catchment. In some areas, increased ponding of water due to the use of depression systems to carry irrigation water is also a factor.

Notwithstanding the poor condition of the waterways in the area, discussions with local landholders indicates the impacts of flooding are limited to sugar cane productivity reductions or losses, and periodic loss of road access.

The township of Home Hill does not appear, in general, to be severely inundated during local rainfall events, suggesting reasonable performance of the existing drainage system. There are concerns related to the section located west of Bruce Highway, in particular around Gardner’s Lagoon. Once the maximum capacity of this deep natural depression is reached, floodwaters flow through the urban area in a south-east direction and join the drainage channel running parallel to the railway line. This existing channel seems unable to convey waters without causing localised flooding in this area, suggesting an eventual inadequate capacity and undersized culvert dimensions.
4.6 Impacts of the Bruce Highway and Brisbane-Cairns Rail Line on Flood Passage

A detailed hydraulic analysis was also conducted on all major structures located along the sections of the Bruce Highway and Brisbane-Cairns Rail Line within the study area. These pieces of infrastructure are the main physical features in the study area that have the potential to limit the passage of floodwaters. Both of these systems are elevated above the surrounding floodplain with cross-drainage provided by culvert structures.

To determine the extent to which these systems hinder flood passage, six major composite structures were analysed using a detailed pipe and structure hydraulic modelling software (CulvertW). Information on culvert numbers and sizes and on bridge opening dimensions were obtained from Department of Main Road and QRAIL and confirmed during field inspections. Relevant terrain data, such as culvert invert and top levels were extracted from the available original Digital Terrain Model, and peak discharges for the range of the modelled design rainfall events were extracted from the MIKE21 flood surface results at each selected location. Each structure was entered in the CulvertW software on a stand-alone basis and maximum discharges and headlosses through the structure were computed based on the above information.

By comparing the maximum potential capacity of each structure with the relative predicted peak discharges it was possible to assess the maximum degree of obstruction caused by each structure, in the worst hydraulic situation.

The details of the results of this investigation are summarised in Appendix F. Generally, all the selected structures appear capable of conveying the predicted peak discharges up to the 100 year ARI design rainfall event without causing impediment to natural flows. The existing sizes and dimensions of culverts and openings ensure an acceptable performance of each crossing, with a computed highest headloss of approximately 0.40-0.50 m for the 100 year ARI local event. During regional river flooding, the volume of floodwaters involved in the flooding process is of a higher order of magnitude when compared to the maximum hydraulic capacities of the above structures, thus causing regular overtopping both of the railway and the highway. The maximum affluxes are reached when floodwaters start spilling over the top of each structure, creating maximum headlosses of approximately 0.6-0.7m. The impacts of each crossing reduce as floodwaters rise over the top of each structure, generating submerged conditions downstream.

The results of this analysis demonstrate that the existing railway line and the Bruce Highway have only minor impacts on the peak flood levels experienced upstream during both local and regional flooding.
4.7 Home Hill Stormwater Drainage Analysis

An additional detailed analysis of the Home Hill stormwater drainage system was conducted as part of this study, to ensure the adequacy of the design proposed by Burdekin Shire Council.

Eight different stormwater drainage systems were investigated separately, in particular:

1) Twelfth Avenue leading to Kidby Gully;
2) Thirteenth Avenue leading to Kidby Gully;
3) Fourteenth Avenue between Sixth Street and Tenth Street;
4) Second Street between Ninth Avenue and Tenth Avenue;
5) Underground drainage path from Barton’s Service Station to Ford’s Gully;
6) Third Avenue to Gardner’s Lagoon;
7) Fifth Avenue from Twelfth Street to First Street;
8) Downstream of Home Hill State High School.

A Rational Method analysis was undertaken for each system to predict its performance under a Q5 discharge and under a Q100 peak discharge, as in accordance with the Queensland Urban Drainage Manual (QUDM, 1992) and with Council requirements.

In light of the data provided by Council, a basic Rational Method analysis was used to predict the discharge at the outlet of each systems catchment area. This discharge was then used to assess the adequacy of the pipework in the lower half of each catchment.

For systems number 5 and 7, a simplistic DRAINS hydraulic model of the stormwater systems currently in place was established. The Rational Method was again employed for analysing these two systems

The details of the results of this investigation are presented in Appendix G. For each system, upgrade works were also modelled and recommendations for each of the individual systems were proposed, if necessary.

The only drainage system found to be not adequate to contain the Q5 is system No.5 (underground drainage path from Barton’s Service Station to Ford’s Gully), with the overland flowpath overtopping in this event. The 1200x600 RCBC from 10th Street to 7th Street is also under-capacity in a 5 year ARI event with the Hydraulic Grade Line extending approximately 5.5m above the surface level. The current arrangement of pipes and overland flowpaths is also not ideal due to the proximity of adjacent residential properties.
The required upgrades were modelled using DRAINS hydraulic model, based on piping of the Q5 event between Tenth and Thirteenth Avenue. The modelling indicates that three 1800x1200 RCBCs are required. The size of the culverts is governed by the need to maintain the HGL at or beneath the surface (i.e., not pressurise the pipe) and can be attributed to the flat hydraulic gradient through this area. Between Tenth Avenue and Eighth Avenue, the required pipe size is reduced to 2 of 1200x900 RCBCs. At the top of the system, two 750 dia RCPs are required.

Preliminary estimates of the potential cost involved in the construction of the proposed upgrades indicate the capital cost would be approximately $3,800,000 for civil works only, with a total cost of more than $5,200,000 including preliminaries, service relocation, survey and design, project management and contingencies. A breakdown of these estimates is shown in Appendix G.

Generally, all the other remaining designed stormwater drainage systems appear capable of conveying the Q5 discharge safely. The existing or proposed design sizes and dimensions of culverts and drains ensure a Q5 discharge immunity for the Home Hill township. In a 100 year ARI flood event, a few systems (3, 4 and 6) can cause localised overland flows, with waters inundating the adjacent road reserve.
5. Flood Risk Management

5.1 Risk Management Principles
Flood risk management, or how we deal with the consequences of flooding, is a new and formal statement of an old concept that has been practised informally since communities began to appreciate and deal with the consequences of flooding.

Risk management has now become a structured approach and it is based on four fundamental procedures:

- Identify the risks: What can go wrong? What can happen?
- Analyse the consequences: What does the risk involve if it happens?
- Set the criteria: What are the principles to account for?
- Mitigate the risk: What can be done?

This methodology formed the basis for the identification of key mitigation measures aimed at improving the overall floodplain situation under flooding conditions. Risks and consequences were determined separately for both the regional and the local flooding, and are summarised in a matrix, as shown in Table 5-1:

<table>
<thead>
<tr>
<th>Risks</th>
<th>Consequences</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Regional River Flooding</strong></td>
<td></td>
</tr>
<tr>
<td>General large scale flooding over the entire floodplain;</td>
<td>Extensive damage to properties and infrastructure;</td>
</tr>
<tr>
<td>River breakouts at new locations.</td>
<td>Extensive agricultural losses;</td>
</tr>
<tr>
<td>Re-routing of the main river channel</td>
<td>Prolonged loss of important road/rail access due to damage;</td>
</tr>
<tr>
<td></td>
<td>Potential loss of life.</td>
</tr>
<tr>
<td><strong>Local Rainfall Flooding</strong></td>
<td></td>
</tr>
<tr>
<td>Extensive rural flooding particularly in downstream areas;</td>
<td>Property and agricultural losses;</td>
</tr>
<tr>
<td>Channel erosion / soil degradation.</td>
<td>Short term loss of road access due to inundation;</td>
</tr>
<tr>
<td></td>
<td>Social disruption.</td>
</tr>
</tbody>
</table>
There are a number of important considerations in relation to potential flood mitigation measures. These include:

- Under large events, flooding behaviour in the floodplain is complicated by overflows from the Burdekin River. Generally, regional river flooding risks are extremely difficult to avoid, due to the magnitude of water volumes involved;
- A general sense of awareness and acceptance of the risks of living in a flood-prone area is present throughout the community (“very little can be done”);
- When implementing flood mitigation measures, extreme caution should be adopted to ensure flooding is not, in fact, exacerbated by the proposed actions;
- The natural (as distinct from “existing”) flood and drainage characteristics of the landscape should be considered and if possible maintained, preserved or restored;
- Whenever possible, the flood management should try to facilitate flows through existing waterways and to minimise the period of residual inundation;
- Improvement strategies should be as equitable as possible (“sharing the risks”);
- Improvement strategies should give consideration to capital and ongoing costs, environmental impacts, practicability, likely degree of stakeholders acceptance, and residual risks.

As a result of the initial assessment of the modelling, and following discussion with local stakeholders, a number of potential improvement options were identified for further investigation. A combination of structural measures and non-structural strategies were analysed using the hydraulic model to evaluate their impacts. Concept-level cost estimates were prepared to assess the feasibility and effectiveness of each option.

### 5.2 Regional Flooding Mitigation Measures

#### 5.2.1 Home Hill Levee

Home Hill township is particularly susceptible to river flooding, and has suffered significant damage in recent history from relatively low recurrence interval river floods. Local flooding has also caused some problems, particularly on the western side of town.

During discussions with local landholders, the possibility of constructing a levee along the western boundary of the town was suggested. It was proposed that such a levee would protect the western part of the town from local flooding, and provide some protection from river floods. Figure 5-1 shows the potential location of the levee.

The proposed levee would provide a barrier to river overland flow (from Lago’s breakout). Floodwaters approaching the town from the western side would initially be collected in Gardner’s...
Lagoon. Overflows from the lagoon would be diverted to a south-east direction, parallel to the Railway line, to join an existing natural channel system (Porters’ Lagoon diversion).

* Figure 5-1: Proposed Levee

On-site assessment and further discussions with local stakeholders indicated that this option would not be feasible due to:

- potential negative impacts on the township by blocking floodwaters flowing from north and eventually increasing water levels throughout the urban area on the east side of the highway;
- the option would attract strong opposition from individual landholders, across whose land the levee would pass;
- the generally flat nature of the local terrain means that it would be difficult to “tie in” the levee with natural high ground, leaving the system exposed to out-flanking during flood events.

As a consequence, the levee option was not considered further.

5.2.2 Flood Warning System and Emergency Response Plan
Burdekin Shire Council has recently conducted a major upgrade and expansion of its flood warning system for the Burdekin River. ALERT monitoring systems were installed in conjunction with the
Bureau of Meteorology, to provide better information on rainfall in the Burdekin River catchment. Predictive modelling using this data and the BoM URBS models are used to assess flood risk.

Burdekin Shire Council is also the coordinating authority for the local Counter Disaster Committee. This Committee is chaired by the Mayor of Burdekin Shire Council, and is responsible for implementing emergency response actions. Council offices are used to coordinate emergency activities.

The adequacy and performance of the flood warning system is regularly re-assessed by Council, in consultation with the Bureau of Meteorology. A general review of the system as part of this study indicated that the system provides a very good basis for predicting flooding behaviour in the lower Burdekin River.

With respect to a flood emergency, Council already has the procedures in place to advise the local community using local media and emergency services such as the Fire Brigade and SES. Flood-proof evacuation centres have been nominated for the congregation and housing of the effected community. It therefore appears that, whilst the risk posed by flood overflows from the Burdekin River is significant, appropriate measures are in place (or are being implemented) to ensure the risks to human life are minimised.

An integral part of Council’s flood risk management will be the ongoing maintenance and improvement of this system, and the associated response emergency procedures. In particular, ongoing community education and awareness programs will be required to maintain awareness, and to ensure response procedures can be effectively implemented when needed.

Establishment of improved evacuation arrangements for the Groper Creek community during times of large river flooding is recommended. This should include the development of an elevated helicopter landing pad in Groper Creek for the emergency evacuation of residents during severe floods.

5.2.3 Land Planning
Burdekin Shire Council is currently revising its Planning Scheme, including the study area. This provides an opportunity to designate major natural flowpaths in the study area to ensure future development is consistent with natural flood carrying functions of those areas. Designation of “Flood Pathways” will not allow Council to control works on private lands when within the designated use, however it will provide a mechanism for controlling works on public infrastructure, or major developments requiring referral to Council.

It is proposed that “Flood Pathway” zones be defined as shown in Figure 5-2: Example of Designated Flood Corridors.
5.2.4 Sand Deposit and Vegetation Clearing

Changes in river bed levels due to the deposition of sand and vegetation growth within the Burdekin River were suggested as a potential cause of increased risks of river flooding. According to local landholders, the river bed morphology has changed during the last decades, with extensive sand build-up in the river bed, raising flood levels. Vegetation growth on sand bars in the river is believed to exacerbate this problem by obstructing natural flood flows.

Bed levels survey data recorded at the old Railway Bridge over recent decades (Kinhill, 2000), do not support this claim. Despite considerable changes in bed level between 1910 and 1992, with significant bed level modifications up to 4-6m in the 1980’s, the mean bed level in 1992 was only about 0.16m higher than it was in 1910. Currently, average bed levels (extracted form the laser survey) are essentially the same as in 1958 and are lower by about 0.54m than in 1982.

Notwithstanding the above, an analysis was undertaken to assess the impacts of large scale sand and vegetation removal from the Lower Burdekin River on flood levels. This involved:

- an adjustment of the current river Digital Terrain Model to simulate the “improved” river bed conditions (removal of sand deposits). A general reduction of 1m was applied to a section of the river bed invert, from The Rocks to approximately 2-3 km downstream the Inkerman Bridge;
a reduction of the existing roughness parameters, to reproduce vegetation clearing operations within the river that would be undertaken as part of the sand extraction. The value of the Manning’s coefficients adopted was reduced from 0.03 to 0.025 to simulate lower channel roughness.

The 1991 flood event was run through the modified MIKE21 model to evaluate the impacts of these modifications over the flooding behaviour. An afflux plot was produced to show the differences in water levels between the modified and the existing conditions. Results are presented in Figure 5-3.

Figure 5-3: Sand deposit and vegetation clearing – Afflux Plot

Clearly, the extraction of a 1m deep sand layer from the river bed would have the benefits of reducing flooding over the right floodplain upstream of the Bruce Highway, with a maximum decrease of 0.3-0.4m.

However the modelling indicated that more water would also be carried towards the downstream section of the river, causing increased flooding immediately downstream of Burdekin River Bridge. Water levels would be raised by up to 0.3m in these areas. These impacts would be considered unacceptable by landholders in these areas.
These results suggest that if any major modification of the river bed were to be countenanced, they would need to commence at the downstream end of the river and progress upstream. This would potentially include removal of the South Burdekin Water Board sand dams. Considerable expense would be also be involved, and there would also be significant environmental issues to address. As such large scale sand removal to improve flood passage down the river would be difficult to justify.

Visual inspection does suggest, however, that vegetation growth on sand bars in the lower river could have the potential to exacerbate flooding. Targeted vegetation removal could improve flood capacity of the river channel.

The possibility of controlled removal of this vegetation should be included in a general Vegetation Management Plan for the study area. This Plan would need to be developed in conjunction with Council, the Burdekin River Improvement Trust, South Burdekin Water Board, responsible authorities and local landholders.

5.2.5 Bank stabilisation

The Burdekin River Improvement Trust is currently responsible for undertaking bank erosion protection works along the lower Burdekin River. These works are carried out on an ongoing basis, subject to the degree of erosion risk and available funding.

The risk of the river breaking out at new locations is a significant concern, with potential to cause substantial agricultural damages and loss of rural infrastructure. Breakouts have occurred at new locations on a number of occasions in recent history. The River Trust monitors river bank condition, and where feasible undertake improvement or erosion protection works. Their activities are an important part of minimising, as far as possible, the risks of river overflows and consequent flooding. The activities of the River Trust should continue to be supported as part of the Flood Management Strategy for the Home Hill area.

5.3 Local Flooding Mitigation Measures

5.3.1 Option A: Pump to River

As described in Section 4.5.3, one of the main locations subject to regular inundation during local rainfall flood events is the area around the confluence of Kidby and Ford’s Gully. The possibility of reducing water levels and duration of inundation in this area by constructing a flood relief pumping system was investigated.

The pump would transfer floodwaters to the river, and would operate only during flood events. Peak flows through this area are relatively high and removal of all of this flow, or even a significant proportion of it, is not feasible due to the high costs involved. The main benefit of a flood relief system would be the reduction in the duration of residual flooding at the end of the
flood event, thereby potentially avoiding crop damage due to prolonged inundation. As such smaller capacity pumping systems could provide some benefits.

To assess the likely performance of a flood pumping system, a pumping capacity of 100 ML/d was modelled. Figure 5-4 shows a possible location of the pump, approximately 1 km downstream the confluence of the two gullies.

*Figure 5-4: Option A - Pump Location*

The structure of the local MIKE21 flood model was modified by introducing a “sink” at the selected location, which enables the simulation of a pump extracting waters from the model area when activated by floodwaters. The 5 year ARI design local flood event was run through the model to assess local flooding impacts. An afflux plot, which shows the difference in water levels between the modified and the existing conditions, is presented in Figure 5-5.
A general reduction of peak water levels can be observed within a radius of 2-3 km from the pump location. However, this reduction varies only between 0.01 to 0.02 m.

Discharge hydrographs for the 5 year ARI local flood event were also extracted to assess the impacts of the pumping system on the duration of residual flooding over the floodplain. Results are presented in Figure 5-6.
A reduction of the inundation period due to the implementation of the proposed pump system can be observed. However, the duration of residual flooding generally decreases only of approximately 3 to 4 hours, which represents a limited benefit in terms of crop damage.

A preliminary estimate of the potential costs involved in the construction of the proposed pump and pipeline system indicates the capital cost would be approximately $550,000.

The modelling results suggest that even for a pump station with a 100 ML/d capacity, the benefits in terms of reduced peak flows or reduced inundation period are limited. The high capital costs associated with the infrastructure would be hard to justify given these benefits.

5.3.2 **Option B: Floodways**

A second potential mitigation option was proposed and assessed for the downstream area of the floodplain. This area suffers from limited capacity to pass flood flows due mainly to land development and weed growth in waterways. As a consequence, the main flood risks relate to loss of road access, and to a lesser extent potential agricultural losses.

The hydraulic model was modified to reflect excavation of 60m wide artificial floodways in the vicinity of Groper Creek and Fry’s roads. These potential sites were selected after consultation.
with Council and South Burdekin Water Board, and are shown in Figure 5-7. No consultation was undertaken with those landholders on whose land the floodways may potentially be located, or with the other surrounding landholders. As such, the floodway locations modelled are indicative only.

Non-return flood gates would be required at the downstream ends of the floodways to eliminate any potential for river water to flow backwards along the floodways during high river flow events.

**Figure 5-7: Option B - Proposed Floodways**

The 20 year ARI local design event was simulated and the results were compared with the existing conditions, as shown in Figure 5-8.
The afflux plot presented above demonstrates that the two combined floodways cause a positive impact on the area, by decreasing water levels up to 0.20m over an extended section of the floodplain.

The impacts of the combined floodways on the duration of flooding, for the 20 year ARI local flood event, over the downstream areas are presented in Figure 5-9.
The plot presented above shows that the implementation of the proposed floodways has the potential of reducing up by 40% the residual flooding over the downstream areas during a 20 year ARI local flood event. However, it is anticipated that the reduction of the duration of submersion could rapidly diminish during major local rainfall events.

Whilst the option of floodways appears to potentially improve downstream flooding conditions, it has a number of significant drawbacks, including:

- The need for non-return gates at the downstream ends makes the floodways relatively expensive, with estimated capital costs of approximately $600,000.
- Ongoing maintenance of the non-return gates would be required to ensure that they operate properly during times of flood. Even with good maintenance there would remain risk that the gates may not operate effectively when required, causing flooding of agricultural land. This would expose the Council to possible damage claims.
- The floodways would require relatively deep sections of earthwork excavation through the natural levee system adjacent to the river. This would require ongoing maintenance to avoid soil erosion problems.
- The outlet of each floodway to the river would be subject to erosion on an ongoing basis, and as such would require ongoing maintenance.
There is likely to be significant concerns raised by local landholders adjacent to the floodways about loss of land and possible increased flooding.

Given these factors, the option of floodways appears difficult to justify when considered in terms of the agricultural benefits likely to result.

5.3.3 Option C: Aquatic Weed Reduction

As mentioned in Section 4.5.3, extensive aquatic weed growth has been identified as one of the key issues for the study area, particularly for the downstream sections where prolonged flooding occurs. It is clear that this weed proliferation has largely been caused by ineffective waterway management and it has caused extensive reduction in waterway capacity and culvert blockages during flooding events.

The possibility of eliminating or reducing aquatic weeds along the main channels of the two most affected creeks, MacDonald and Mosquito Creeks, was considered as a mitigation measure for this area. Figure 5-10 shows the selected channel corridors where aquatic weed clearing would be most beneficial.

Figure 5-10: Option C - Channel Vegetation Reduction
The Manning’s coefficients in the roughness map of the MIKE21 local flood model was changed to simulate improved smooth conditions along the two proposed corridors, reflecting improved weed management in these areas. The modelling assumed that weed removal would be feasible in a 50m wide corridor through these systems, reflecting the combined use of targeted aerial spraying, spraying from boats, and mechanical removal of weed growth. The existing roughness values set at 0.150, representing very dense vegetation and tree obstructions, were altered and set at 0.040, corresponding to relatively clean flood pathways, with no major obstructions.

The impacts of these modified waterway conditions were assessed by running both the 5 year and the 20 year ARI design local flood events and by comparing the results with the existing situation. Afflux plots were produced for both events and are presented in **Figure 5-11** and **Figure 5-12**.

**Figure 5-11: Option C - 5 Year ARI Afflux Plot**

![Option C: Reduction of Aquatic Weeds in Groom Creek Area (MacDonald & Mosquito Creek)](image)
The modelling indicates that improved waterway conveyance conditions can produce significant positive effects in the downstream areas. A general improvement can be observed within a radius of 3-4 km, with water levels decreasing in average by 0.3 m, up to a maximum of 0.5 m in some areas along Mosquito Creek. In the Groper Creek area, water level reduction was approximately 0.3 m.

The implementation of this option would require the development of an appropriate Vegetation Management Plan, with approvals from responsible authorities and agencies. Preliminary consultation with the Environmental Protection Agency (EPA), indicates that large-scale excavations works, or removal of existing natural vegetation (particularly trees) would not be permitted, however, spraying with environmentally compatible herbicide in a targeted manner would be acceptable. Further discussions with Council, South Burdekin Water Board and responsible agencies are required to confirm the most appropriate procedures to be applied. More detailed assessment of the weed growth and waterway capacity would also be required to determine the best weed control methods.

Cost estimates for this type of work are very difficult to define in the absence of a clearer understanding of the extent of creek systems to be cleared, and the various techniques that could be
used. It could be expected that initial costs could be approximately $200,000 over the first two years, with annual ongoing costs of $30,000 to $50,000.

5.3.4 Waterway management
Aquatic weed growth is partly caused by the extended presence of water in the natural depressions, due to use of depressions as water carriers for irrigation, and from agricultural runoff. As a consequence, the natural hydrological cycle of these systems has changed from ephemeral to perennial, allowing the colonisation of the waterways by opportunistic aquatic weed species such as cumbungi and hyacinth.

Recent efforts to isolate irrigation channels from waterways in upstream areas of the system have been successful. Opportunities for further works need to be identified in conjunction with the South Burdekin Water Board, both in downstream and upstream areas. These improvements, in conjunction with targeted aquatic weed removal would greatly improve the flood carrying capacity of the natural depression systems.

5.3.5 Flood Easements
Home Hill township is drained by two main drainage depressions – Kidby and Ford’s Gully. At present these systems appear to have adequate capacity to drain the township under local flood events. Preservation of this capacity is important, to ensure the ongoing performance of the town drainage system.

In recognition of this, Burdekin Shire Council has commenced the acquisition of flood easements along these waterways. This process has largely been undertaken on an ad-hoc basis, in conjunction with land subdivision applications by the landholders concerned.

A coordinated acquisition program to secure easements is required, to ensure the easement process be completed within the next few years. The existing watercourses are relatively well defined, and generally have not been developed for agriculture. Acquisition should be limited to the defined waterway area, and should extend, in the short term, as far downstream as the junction of the two gully systems. These easements will provide a legal framework to restrict future development within the two watercourses to ensure flood capacity is not diminished.

Downstream of this location, flooding becomes more extensive as a consequence of a broadening of the flood pathways. These areas are serviced by moderately low capacity rural drains, which are relatively ineffective under large flood events. As a consequence, acquisition of flood easements in this area is less justifiable, given the extensive areas of land involved, and the reduced probability of any particular localised works having a significant impact on the overall flood behaviour. In recognition of this, the most appropriate form of control in these areas would be via appropriate zoning and planning controls, as discussed in Section 5.2.3.
5.4 Flood Risk Management Strategy Plan

Based on the above assessment of flood management options, a Management Plan has been developed that is a combination of continuation of existing measures, and implementation of a number of new strategies mainly in the form of non-structural measures. In general, structural options appear to have little potential to provide cost-effective improvements to either river or local flooding. The elements of the Plan include:

1) Development and implementation of a Vegetation Management Plan for the lower Burdekin River and the main watercourses draining the Home Hill catchment area. For the Burdekin River, this Plan will aim to identify areas where vegetation is impinging on the flood capacity of the River, and identify appropriate means of reducing vegetation growth in these areas. Such areas will be limited to vegetation growth within the River channel, and will not include the river banks.

   For the watercourses draining the Home Hill area the Plan will, in the short term, aim to establish a “flood corridor” by reducing weed growth through the appropriate use of herbicides and mechanical weed harvesting techniques. Strategies for the long term management of aquatic weed growth will also need to be developed. These strategies will need to be integrated with improvements in water management outlined in 2) below.

2) An ongoing program of removal of irrigation supply water from natural flood watercourses should be jointly developed by Council, the South Burdekin Water Board, and regulatory agencies. Whilst it is recognised that practical difficulties and costs will limit the capacity to undertake works in some areas, further detailed examination of this issue is required to identify areas where works can proceed.

3) Designation of “Flood Pathways” under Council’s new Planning Scheme, to provide a means for Council to control significant development in these areas. This is important for the preservation of waterway capacity, and the ability for river floods to pass through the Home Hill area with minimum impact.

4) Acquisition of easements along unsecured reaches of Kidby and Ford’s Gullies, extending downstream to the junction of those watercourses. This will provide ongoing secure drainage pathways for the Township of Home Hill.

5) Continued support and development of the existing Flood Warning System established by Council, including ongoing review of the performance of the system, and a program of community awareness of flood risks and response arrangements.

6) Establishment of improved evacuation arrangements for the Groper Creek community during times of large river flooding. This should include the establishment of an elevated helicopter landing pad in Groper Creek for the emergency evacuation of residents during severe floods.
7) Continued support for the activities of the Burdekin River Improvement Trust in identifying areas for potential river breakout, and undertaking works to reduce the risks of new breakouts occurring.

Table 5-2 summarises the main flooding risks for the study area, and outlines how each of the above options address these risks.
### Table 5-2: Summary of Risks and Mitigation Options

<table>
<thead>
<tr>
<th>Process</th>
<th>Probability</th>
<th>Consequence</th>
<th>Risk</th>
<th>Risk Management Option</th>
<th>Comments</th>
</tr>
</thead>
</table>
| Widespread flooding from Burdekin River | High | High | High | - Flood Warning System  
- Emergency Response Plan  
- Land Planning  
- Removal of irrigation water from natural watercourses.  
- Vegetation Management Plan (Burdekin River and Downstream Creek systems) | Non-structural measures proposed in recognition of difficulty in dealing with large regional flooding events. To be undertaken in conjunction with vegetation management |
| New Burdekin River breakouts | Medium | High | Medium / High | - Ongoing Bank Stabilisation Works  
- Vegetation Management Plan | River Trust works to be supported |
| Flooding of isolated Groper Creek community | Medium | High | Medium | - Flood Warning System  
- Air Rescue Services | Flood risk due to coincidence of local flooding followed by regional floods, endangering lives in Groper Creek |
<table>
<thead>
<tr>
<th>Local Rainfall Flooding</th>
<th>Process</th>
<th>Probability</th>
<th>Consequence</th>
<th>Risk</th>
<th>Risk Management Option</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Flooding in downstream areas</td>
<td>Medium / High</td>
<td>Medium / Low</td>
<td>Medium</td>
<td>- Vegetation Management Plan</td>
<td>Non-structural measures reflect poor cost/benefit of structural options</td>
</tr>
<tr>
<td></td>
<td>Local flooding in Home Hill township</td>
<td>Medium</td>
<td>Medium</td>
<td>Medium</td>
<td>- Flood Easements</td>
<td>Priority on maintaining drainage outfall for the town</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Process</th>
<th>Probability</th>
<th>Consequence</th>
<th>Risk</th>
<th>Risk Management Option</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flooding in downstream areas</td>
<td>Medium / High</td>
<td>Medium / Low</td>
<td>Medium</td>
<td>- Vegetation Management Plan</td>
<td>Non-structural measures reflect poor cost/benefit of structural options</td>
</tr>
<tr>
<td>Local flooding in Home Hill township</td>
<td>Medium</td>
<td>Medium</td>
<td>Medium</td>
<td>- Flood Easements</td>
<td>Priority on maintaining drainage outfall for the town</td>
</tr>
</tbody>
</table>
6. References


Appendix A  Burdekin River Catchment
Appendix B  Historical Flood Inundation Maps

- Figure B 1: 1958 Flood Inundation Map
- Figure B 2: 1991 Flood Inundation Map
Appendix C  Regional Model Calibration

- Figure C 1: 1958 Flood Calibration
- Figure C 2: 1991 Flood Calibration
Appendix D  Regional Flood Model Results

- Figure D 1: 20 year ARI Regional Flood Inundation Map
- Figure D 2: 50 year ARI Regional Flood Inundation Map
- Figure D 3: 100 year ARI Regional Flood Inundation Map
Appendix E  Local Flood Model Results

- Figure E 1: 5 year ARI Local Flood Inundation Map
- Figure E 2: 20 year ARI Local Flood Inundation Map
- Figure E 3: 50 year ARI Local Flood Inundation Map
- Figure E 4: 100 year ARI Local Flood Inundation Map
- Figure E 5: 5 year ARI Local Flood Velocity Map
- Figure E 6: 20 year ARI Local Flood Velocity Map
- Figure E 7: 50 year ARI Local Flood Velocity Map
- Figure E 8: 100 year ARI Local Flood Velocity Map
Appendix F  Hydraulic Structures Analysis
Predicted Peak Discharges (m³/s) from MIKE21

<table>
<thead>
<tr>
<th>Local Flooding</th>
<th>ARI</th>
<th>5</th>
<th>20</th>
<th>50</th>
<th>100</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Kidby</td>
<td>4.2</td>
<td>7.3</td>
<td>9.7</td>
<td>12.4</td>
</tr>
<tr>
<td>2</td>
<td>Drain</td>
<td>1.6</td>
<td>3.7</td>
<td>4.5</td>
<td>5.4</td>
</tr>
<tr>
<td>3</td>
<td>Porters</td>
<td>8.8</td>
<td>16.6</td>
<td>21.6</td>
<td>30.3</td>
</tr>
<tr>
<td>4</td>
<td>Mather</td>
<td>9.3</td>
<td>23.3</td>
<td>35.3</td>
<td>45.1</td>
</tr>
<tr>
<td>5</td>
<td>Lyah</td>
<td>23.8</td>
<td>35.8</td>
<td>43.4</td>
<td>49.7</td>
</tr>
<tr>
<td>6</td>
<td>Alma</td>
<td>97.8</td>
<td>166.2</td>
<td>190.5</td>
<td>243.1</td>
</tr>
</tbody>
</table>

Regional Flooding

<table>
<thead>
<tr>
<th>ARI</th>
<th>20</th>
<th>50</th>
<th>100</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kidby</td>
<td>141.8</td>
<td>284.7</td>
<td>507.4</td>
</tr>
<tr>
<td>Drain</td>
<td>532.5</td>
<td>745.5</td>
<td>1046.9</td>
</tr>
<tr>
<td>Porters</td>
<td>1711.9</td>
<td>2351.9</td>
<td>3166.4</td>
</tr>
<tr>
<td>Mather</td>
<td>398.3</td>
<td>688.8</td>
<td>1079.3</td>
</tr>
<tr>
<td>Lyah</td>
<td>379.8</td>
<td>572.3</td>
<td>835.2</td>
</tr>
<tr>
<td>Alma</td>
<td>223.7</td>
<td>432.4</td>
<td>623.1</td>
</tr>
</tbody>
</table>

Computed Peak Discharges (m³/s) from CulvertW

Highway Crossing
4 Rect. Culvert 2100x1800 RCBC
RL_in | 10.7 m
RL-out | 10.6 m
Slope | 0.001
n | 0.013

<table>
<thead>
<tr>
<th>Local Flooding</th>
<th>Discharge</th>
<th>Headlosses</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q100</td>
<td>12.40</td>
<td>0.12</td>
</tr>
<tr>
<td>Q50</td>
<td>9.70</td>
<td>0.10</td>
</tr>
<tr>
<td>Q20</td>
<td>7.30</td>
<td>0.09</td>
</tr>
<tr>
<td>Q5</td>
<td>4.20</td>
<td>0.03</td>
</tr>
</tbody>
</table>

Regional Flooding
Max Q in Culvert | 42 | 0.6 |

Railway Crossing
7 Span Bridge
Trapezoidal Xsec
Width_base | 30
Batter Slope | 2
Slope | 0.001
n | 0.03 | 0.04
Depth_max | 1.5 | 1.5
Qmax | 64 | 48
### Location 2

**Highway / Railway Crossing**

4 Rect. Culvert 1200x600 RCBC

<table>
<thead>
<tr>
<th>RL_in</th>
<th>9.8 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>RL_out</td>
<td>9.7 m</td>
</tr>
<tr>
<td>Slope</td>
<td>0.001</td>
</tr>
<tr>
<td>n</td>
<td>0.013</td>
</tr>
</tbody>
</table>

**Local Flooding**

<table>
<thead>
<tr>
<th>Discharge</th>
<th>Headlosses</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q100</td>
<td>5.40</td>
</tr>
<tr>
<td>Q50</td>
<td>4.50</td>
</tr>
<tr>
<td>Q20</td>
<td>3.70</td>
</tr>
<tr>
<td>Q5</td>
<td>1.60</td>
</tr>
</tbody>
</table>

**Regional Flooding**

Max Q in Culvert | 7.6 | 0.45

### Location 3

**Porters Lagoon Crossing**

**Highway / Railway Crossing**

10 Rect. Culvert 1200x600 RCBC

<table>
<thead>
<tr>
<th>RL_in</th>
<th>7.3 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>RL_out</td>
<td>7.2 m</td>
</tr>
<tr>
<td>Slope</td>
<td>0.001</td>
</tr>
<tr>
<td>n</td>
<td>0.013</td>
</tr>
</tbody>
</table>

**Local Flooding**

<table>
<thead>
<tr>
<th>Discharge</th>
<th>Headlosses</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q100</td>
<td>30.30</td>
</tr>
<tr>
<td>Q50</td>
<td>21.60</td>
</tr>
<tr>
<td>Q20</td>
<td>16.60</td>
</tr>
<tr>
<td>Q5</td>
<td>8.80</td>
</tr>
</tbody>
</table>

**Regional Flooding**

Max Q in Culvert | 21.8 | 0.6

### Location 4

**Mathers Lagoon Crossing**

**Highway Crossing**

5 Rect. Culvert 2100x1800 RCBC
6 Rect. Culvert 2100x1200 RCBC

<table>
<thead>
<tr>
<th>RL_in</th>
<th>6.8 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>RL_out</td>
<td>6.6 m</td>
</tr>
<tr>
<td>Slope</td>
<td>0.002</td>
</tr>
<tr>
<td>n</td>
<td>0.013</td>
</tr>
</tbody>
</table>

**Local Flooding**

<table>
<thead>
<tr>
<th>Discharge</th>
<th>Headlosses</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q100</td>
<td>45.10</td>
</tr>
<tr>
<td>Q50</td>
<td>35.30</td>
</tr>
<tr>
<td>Q20</td>
<td>23.30</td>
</tr>
<tr>
<td>Q5</td>
<td>9.30</td>
</tr>
</tbody>
</table>

**Regional Flooding**

Max Q in Culvert | 96.5 | 0.7
### Railway Crossing
7 Span Bridge  
**Trapezoidal Xsec**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value 1</th>
<th>Value 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width_base</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>Batter Slope</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>Slope</td>
<td>0.001</td>
<td></td>
</tr>
<tr>
<td>n</td>
<td>0.03</td>
<td>0.04</td>
</tr>
<tr>
<td>Depth_max</td>
<td>1.5</td>
<td>1.5</td>
</tr>
<tr>
<td>Qmax</td>
<td>64</td>
<td>48</td>
</tr>
</tbody>
</table>

6 Span Bridge  
**Trapezoidal Xsec**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value 1</th>
<th>Value 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width_base</td>
<td>25</td>
<td></td>
</tr>
<tr>
<td>Batter Slope</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>Slope</td>
<td>0.001</td>
<td></td>
</tr>
<tr>
<td>n</td>
<td>0.03</td>
<td>0.04</td>
</tr>
<tr>
<td>Depth_max</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Qmax</td>
<td>120</td>
<td>90</td>
</tr>
</tbody>
</table>

### Location 5  
**Iyah Creek Crossing**

#### Highway Crossing
6 Rect. Culvert 2100x1800 RCBC

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value 1</th>
<th>Value 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>RL_in</td>
<td>6.4 m</td>
<td></td>
</tr>
<tr>
<td>RL-out</td>
<td>6.2 m</td>
<td></td>
</tr>
<tr>
<td>Slope</td>
<td>0.002</td>
<td></td>
</tr>
<tr>
<td>n</td>
<td>0.013</td>
<td></td>
</tr>
</tbody>
</table>

**Local Flooding**

<table>
<thead>
<tr>
<th>Discharge</th>
<th>Headlosses</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q100</td>
<td>49.70</td>
</tr>
<tr>
<td>Q50</td>
<td>43.40</td>
</tr>
<tr>
<td>Q20</td>
<td>35.80</td>
</tr>
<tr>
<td>Q5</td>
<td>23.80</td>
</tr>
</tbody>
</table>

**Regional Flooding**

<table>
<thead>
<tr>
<th>Max Q in Culvert</th>
<th>Value 1</th>
<th>Value 2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>73</td>
<td>0.77</td>
</tr>
</tbody>
</table>

### Railway Crossing
10 Span Bridge  
**Trapezoidal Xsec**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value 1</th>
<th>Value 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width_base</td>
<td>45</td>
<td></td>
</tr>
<tr>
<td>Batter Slope</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>Slope</td>
<td>0.001</td>
<td></td>
</tr>
<tr>
<td>n</td>
<td>0.03</td>
<td>0.04</td>
</tr>
<tr>
<td>Depth_max</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Qmax</td>
<td>154</td>
<td>115</td>
</tr>
</tbody>
</table>
Appendix G  Stormwater Drainage Analysis

Figure G 1: Stormwater Drainage System - Catchment Boundaries
Home Hill Stormwater Drainage System – Details

Eight Stormwater Drainage Systems within the Home Hill DCP area were analysed for adequacy of design. These areas are given below:

1) Twelfth Avenue leading to Kidby Gully
2) Thirteenth Avenue leading to Kidby Gully
3) Fourteenth Avenue between Sixth St and Tenth St
4) Second St between Ninth Ave and Tenth Ave
5) Underground drainage path from Barton’s Service Station to Ford’s Gully
6) Third Ave to Gardner’s Lagoon
7) Fifth Ave from Twelfth St to First St
8) Downstream of Home Hill State High School

A Rational method analysis was undertaken for each system to predict its performance under a Q5 discharge (minor system design in accordance with BSC requirements) and also under a Q100 peak discharge. In light of the data provided by BSC, a basic Rational method analysis was used to predict the discharge at the outlet of each systems catchment area. This discharge was then used to assess the adequacy of the pipework in the lower half of the catchment. Although this is a conservative approach, it will be indicative of the overall systems capacity, without performing a detailed analysis.

For systems number 5 and 7, a simplistic DRAINS hydraulic model of the stormwater systems currently in place was established. The Rational method was again employed for analysing these two systems.

In both cases, upgrade works were also modelled and recommendations for each of the individual systems are given below.

1) Twelfth Avenue leading to Kidby Gully

This system was designed to divert runoff from part of the catchment draining to the existing system in Thirteenth Avenue and to pick up additional flows from Tenth Street to alleviate drainage problems in these areas. It was noted in the BSC report that the system has not been designed to cater for river flooding or major storm events. As such, this system has been analysed for the minor storm event of a 5 year ARI.
Catchment areas and pipe sizes have been based on data provided by BSC in their planning report. Further details of pipe grades and sizes on plans provided by BSC for this system indicate that a general pipe grade of 0.11% has been adopted.

The Twelfth Avenue system caters for a total catchment area of 21 Ha, resulting in a predicted Q5 discharge of 3.45 m³/s. The proposed drainage system consists of two 1350 dia RCPs (at a slope of 0.11%) and has a theoretical capacity of 3.8 m³/s. As such, this system will safely convey the 5 year ARI peak discharge.

2) Thirteenth Avenue leading to Kidby Gully

This existing piped system consists of one 1350 dia RCP at its outlet, connected to the upstream system by a 1200 dia RCP (both at an average grade of 0.11% as discussed in Catchment #1). This system was analysed in conjunction with Catchment #1 and was again based on the report provided by BSC. With the Twelfth Avenue system in place, the catchment area draining to this system will be approximately 11 Ha of low density residential development. Using the Rational Method, a peak 5 year ARI discharge of 1.8 m³/s was predicted.

A Manning's check of the existing pipework indicates that the 1350 dia RCP has a theoretical capacity of 1.9 m³/s and the 1200 dia RCP has a capacity of 1.5 m³/s. As such, this system will have adequate capacity to convey the 5 year ARI peak discharge with the Twelfth Avenue system in place.

3) Fourteenth Avenue between Sixth St and Tenth St

This system has been recently designed by BSC and consists of a proposed new length of underground drainage extending from Tenth Street to Fourth Street and eventually discharging into Ford’s Gully. Kerbing and Channelling of Fourteenth Ave is also proposed.

Based on the catchment areas provided by BSC, it was estimated that approx 15 Ha of residential land is serviced by this system. At the outlet of the catchment a Q5 of 2.2 m³/s was predicted using the Rational method. The designed system of two 1050 dia RCPs was found to be adequate to convey the Q5 discharge.

In a 100 year ARI event, a peak discharge of 5.2 m³/s was predicted. An analysis of the proposed road profile indicates that an overland flow of 3 m³/s (2.2 m³/s conveyed in piped system) would not be contained in the road reserve. This has not been considered to be a major issue as, based on the Flood Study results, in a Q100 event widespread flooding throughout the town occurs from adjacent creeks and river.
4) Second St between Ninth Ave and Tenth Ave
This system has also been recently designed by BSC and comprises two areas of underground drainage which outlet into an open channel through Ford Park. Council has referred to the two systems as follows:
Subcatchment A – Second St (Eighth Ave to Ninth Ave)
Subcatchment B – Second St (Ninth Avenue to Tenth Ave)
Detailed information was only provided for Subcatchment B, so only the pipework in this system was analysed. The open channel through Ford Park and the racecourse was assessed based on discharge from both Subcatchments A & B.
The report indicates that one 750 dia RCP will be installed through the drainage corridor between First Street and Second Street. The Rational method analysis indicates that this will be insufficient to convey the predicted Q5 of approx 1.2 m$^3$/s. It would be recommended that at least two 750 dia RCPs are provided through this corridor to meet a Q5 immunity.

The 1.5m deep open channel through Ford Park shows sufficient capacity to convey the Q100 discharge from Subcatchment B (from Chainages 0 to 270 on Plan # D/139 Sheet 5). However, from Chainage 270 onwards (ie at junction with drain from subcatchment A), the channel is shown in the cross section details to become wider with flatter side slopes. This shallower drain is insufficient to convey the Q100 from both subcatchment A and B. This may not pose any major problems as there does not appear to be any buildings in the vicinity of the drain. The only area of concern would be the railway line adjacent to the drainage path, with confirmation of ground levels and potential flooding being required on site.

5) Underground drainage path from Barton’s Service Station to Ford’s Gully
A DRAINS hydraulic model of this system was established to represent the existing pipework and overland flowpaths from Barton’s Service Station to the outlet at Ford’s Gully downstream of the Home Hill High School. Surveyed data provided by BSC was used to establish a simplified hydraulic model of the main drainage line (see attached data for System 5). This system was analysed for both the minor (5 year ARI) and major (100 year ARI) storm events.

It was found that the current system cannot adequately contain the Q5 with the overland flowpath overtopping in this event. The 1200x600 RCBC from 10th Street to 7th Street is also under capacity in a 5 year event with the Hydraulic Grade Line extending approximately 5.5m above the surface level. The current arrangement of pipes and overland flowpaths is also not ideal due to the proximity of adjacent residential properties.
The hydraulic model was used to predict the upgraded pipe sizes for this system. In order to underground the open channel Tenth Avenue to Thirteenth Avenue, three 1800x1200 RCBCs are required. This is governed by the need to maintain the HGL at or beneath the surface (ie not pressurise the pipe) and can be attributed to the flat grade of the pipes. Between Tenth Avenue and Eighth Avenue, the required pipe size is reduced to 2 of 1200x900 RCBCs. At the top of the system, two 750 dia RCPs are required. These pipe sizes are illustrated on the attached figures.

6) Third Ave to Gardner’s Lagoon

This system is detailed in the BSC report dated 11 December 2000. A simple DRAINS hydraulic model was established based on Option 1, Drawing No D-66 5A as recommended in this report. The catchment areas used in the DRAINS model were also based on the report data and an average pipe grade of 0.5% was adopted based on road profile data also provided by BSC.

The results of the hydraulic modelling indicate that the designed pipe system for Option 1 has sufficient capacity to adequately convey the predicted 5 year ARI peak discharge of 1.5m$^3$/s. It is unlikely that the predicted 100 year ARI peak local discharge of 3.2 m$^3$/s would be contained within the road reserves, however, flooding is also most likely to occur from Gardner’s Lagoon in a Q100.

7) Fifth Ave from Twelfth St to First St

The table drain along Fifth Avenue from Twelfth Street to Sixth Avenue was considered to be the integral component of this drainage system. A DRAINS hydraulic model was established of this reach of drainage, incorporating culvert crossings where data was available. The long section was based on BSC Job WKS14529 Figure D/135, but as no typical cross sections of the drain were supplied, a design has been prepared based on conveying the Q100 safely. This can then be compared on site with the current channel.

From the hydraulic model it was found that the size of drain required to safely convey the Q100 from Twelfth St to Sixth St is a 2m wide base channel, 1m deep and a slope batter 1:4. Any culverts which are required within this channel should be of a minimum size of 1 of 1200 x 900 RCBC. This will give a Q5 immunity to any crossings of the channel.
8) Downstream of Home Hill State High School

In this system an analysis of Ford’s Gully downstream of the Home Hill High School was undertaken, including the culverts beneath Fourteenth Avenue. The catchment area is as shown in System 5 and includes the discharge from the Fourteenth Avenue System previously analysed as System 3.

The road culverts beneath Fourteenth Avenue were previously analysed in System 5 and were found to have adequate capacity to convey the 5 year ARI discharge.

A typical cross section of Ford’s Gully was extracted from the contour data generated from Council’s DTM. This cross section is represented by a typical trapezoidal cross section with a 5m wide base channel, approximately 2.5m deep and a longitudinal slope of 0.01. From the DRAINS model established for System 5, the 100 year ARI discharge at the outlet to Ford’s Gully was predicted to be 24.5 m$^3$/s for local flows. A simple manning’s equation check of the gully cross section indicated that, under a Q100 local discharge, the gully would be approximately flowing half full.

At the downstream end of this drainage system, where Burdekin Road crosses Ford’s Gully, the existing 1200x1200 RCBC box culvert has a capacity of 3.5m$^3$/s which ensures a Q2 immunity to the crossing. A 600 dia RCP 80m long underground pipeline is situated approximately 10m further downstream the Burdekin Road crossing. The existing pipe has a very limited capacity (approx 0.8-1 m$^3$/s), which is insufficient to convey the Q2. In the event of a 2 year event or higher, backwater effects will be likely to occur between the culvert and the pipe, and possibly further upstream, as the constriction caused by the pipe will decrease the culvert capacity.

However, as the configuration of Ford’s Gully between Fourteenth Avenue and Burdekin Road ensures a Q100 immunity, backwaters will be contained within the gully banks and no flooding issues will occur. The following 0.1m contour map shows the existing topographic configuration of the area. No future urban land development should however be permitted in the area upstream of Burdekin Road and the existing channel should be preserved as natural flowpath.
Upgraded pipe network - Catchment #5.
HOME HILL FLOOD STUDY
IMPROVEMENT WORKS
Tenth Ave to 13th Ave
RCBC Culvert

COST ESTIMATE

| Location: | Tenth Ave to 13th Ave |
| ID Number: | RCBC Culvert |
| Description: | 3 No. 1.8 x 1.2 Culverts 1350m long |
| Total Cost | $3,406,973 |

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
<th>Qty</th>
<th>Unit</th>
<th>Rate</th>
<th>Amount</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>Preliminaries</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.1</td>
<td>Establishment</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.2</td>
<td>Temporary works (barriers etc)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td>Civil Works</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.1</td>
<td>Culvert Installation</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.1.1</td>
<td>Excavation</td>
<td>1350</td>
<td>m</td>
<td>$700</td>
<td>$945,000</td>
</tr>
<tr>
<td>2.1.2</td>
<td>Purchase Culverts</td>
<td>1350</td>
<td>m</td>
<td>$660</td>
<td>$891,000</td>
</tr>
<tr>
<td>2.1.3</td>
<td>Concrete Foundation Slab</td>
<td>8019</td>
<td>sq.m</td>
<td>$100</td>
<td>$801,900</td>
</tr>
<tr>
<td>2.2</td>
<td>Miscellaneous</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.2.1</td>
<td>Services Relocation</td>
<td>Item</td>
<td></td>
<td>$100,000</td>
<td>$100,000</td>
</tr>
</tbody>
</table>

| Base Cost | 7 % | $2,765,400 |
| Prime Cost | 5 % | $138,270 |
| Contingencies | 10 % | $309,725 |

Total Cost | $3,406,973 |
HOME HILL FLOOD STUDY
IMPROVEMENT WORKS
COST ESTIMATE
Eighth Ave to Tenth Ave
RCBC Culvert

Location: Eighth Ave to Tenth Ave
ID Number: 2 No. 1.2 x 0.9 Culverts 900m long

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
<th>Qty</th>
<th>Unit</th>
<th>Rate</th>
<th>Amount</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>Preliminaries</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.1</td>
<td>Establishment</td>
<td></td>
<td></td>
<td>$7,500</td>
<td>$7,500</td>
</tr>
<tr>
<td>1.2</td>
<td>Temporary works (barriers etc)</td>
<td></td>
<td></td>
<td>$20,000</td>
<td>$20,000</td>
</tr>
<tr>
<td>2.0</td>
<td>Civil Works</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.1</td>
<td>Culvert Installation</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.1.1</td>
<td>Excavation</td>
<td>900</td>
<td>m</td>
<td>$500</td>
<td>$450,000</td>
</tr>
<tr>
<td>2.1.2</td>
<td>Purchase Culverts</td>
<td>900</td>
<td>m</td>
<td>$400</td>
<td>$360,000</td>
</tr>
<tr>
<td>2.1.3</td>
<td>Concrete Foundation Slab</td>
<td>2376</td>
<td>sq.m</td>
<td>$100</td>
<td>$237,600</td>
</tr>
<tr>
<td>2.2</td>
<td>Miscellaneous</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.2.1</td>
<td>Services Relocation</td>
<td></td>
<td></td>
<td>$100,000</td>
<td>$100,000</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.0</td>
<td>Survey and Design</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.0</td>
<td>Project Management</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.0</td>
<td>Contingencies</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Base Cost
7%

Prime Cost
5%
10%

Total Cost
$1,447,723
HOME HILL FLOOD STUDY
IMPROVEMENT WORKS
COST ESTIMATE

RCP Culverts

Location:
ID Number:
Description: 2 No. dia. 750mm RCP Culverts 380m long

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
<th>Qty</th>
<th>Unit</th>
<th>Rate</th>
<th>Amount</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>Preliminaries</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.1</td>
<td>Establishment</td>
<td></td>
<td></td>
<td>$7,500</td>
<td>$7,500</td>
</tr>
<tr>
<td>1.2</td>
<td>Temporary works (barriers etc)</td>
<td></td>
<td></td>
<td>$20,000</td>
<td>$20,000</td>
</tr>
<tr>
<td>2.0</td>
<td>Civil Works</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.1</td>
<td>Culvert Installation</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.1.1</td>
<td>Excavation</td>
<td>380</td>
<td>m</td>
<td>$300</td>
<td>$114,000</td>
</tr>
<tr>
<td>2.1.2</td>
<td>Purchase Culverts</td>
<td>380</td>
<td>m</td>
<td>$160</td>
<td>$60,800</td>
</tr>
<tr>
<td>2.2</td>
<td>Miscellaneous</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.2.1</td>
<td>Services Relocation</td>
<td></td>
<td></td>
<td>$100,000</td>
<td>$100,000</td>
</tr>
<tr>
<td>3.0</td>
<td>Survey and Design</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.0</td>
<td>Project Management</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.0</td>
<td>Contingencies</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Base Cost:

- Survey and Design: 7%
- Project Management: 5%
- Contingencies: 10%

Prime Cost: $338,576

Total Cost: $372,434