Connell Wagner

HINCHINBROOK SHIRE COUNCIL

Hinchinbrook Storm Surge Study

Final Report

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List of Abbreviations

ABS	Australian Bureau of Statistics
AHD	Australian Height Datum
ARI	Average Recurrence Interval
BoM	Bureau of Meteorology
BPA	Beach Protection Authority
CL	Confidence Limits
CP	Central Pressure (hPa)
CSIRO	Commonwealth Scientific and Industrial Research Organisation
DCDB	Digital Cadastral DataBase
DoE	Department of Environment (now the Environmental Protection Agency)
DES	Department of Emergency Services
DNRM	Department of Natural Resources and Mines
EPA	Environmental Protection Authority
EXV1	Extreme Value Type 1
GIS	Geographic Information System
GPD	Generalised Pareto Distribution
HAT	Highest Astronomical Tide
HSC	Hinchinbrook Shire Council
HSSSS	Hinchinbrook Shire Storm Surge Study
HRIC	Hinchinbrook Resource Information Centre
HRIT	Herbert River Improvement Trust
H_s	Significant Wave Height
L&T	Lawson and Treloar
NDRMSP	Natural Disaster Risk Management Studies Program
PAR	Population at Risk
PSM	Permanent Survey Mark
RL	Reduced Level
Rm	Radius to Maximum Winds
SAG	Study Advisory Group
PMS	Probable Maximum Surge
Tp	Peak Spectral Period
V_s	Cyclone Forward Speed

Executive Summary

The Hinchinbrook Storm Surge Study has been carried out under the Natural Disaster Risk Management Studies Program administered in Queensland by the Department of Emergency Services. This study was undertaken by Connell Wagner with specialist sub-consultants Lawson and Treloar.

The Storm Surge Study aimed to:

- Identify the true risk of storm surge inundation along the coastal of Hinchinbrook Shire, and the south and western coastal regions of Great Palm Island;
- Identify critical areas of concern arising from climatic changes;
- Identify storm surge inundation by preparing storm surge hazard mapping; and
- Develop appropriate planning measures and response plans.

Using the Monte Carlo analysis procedure the study developed detailed storm tide statistics at the following sites within the Council's areas of responsibility;

- Cassady Beach;
- Allingham;
- Taylors Beach South;
- Taylors Beach North;
- Lucinda;
- Dungeness;
- Farm (Great Palm Island);
- Mission (Great Palm Island); and
- Butler Bay (Great Palm Island).

Table E1 presents the predicted water levels at the selected locations.

Table E1 – Peak Storm Tide at Selected Locations (excl. Wave Setup and Greenhouse Related Climate Change)

Location	Peak Storm Tide (m AHD)				
	50yr ARI	100yr ARI	500yr ARI	1000yr ARI	10000yr ARI
Allingham	2.32	2.63	3.25	3.51	4.37
Cassady Beach	2.31	2.62	3.25	3.51	4.36
Dungeness	2.13	2.37	2.87	3.07	3.75
Lucinda	2.09	2.32	2.78	2.97	3.60
Taylors Beach North	2.19	2.49	3.11	3.37	4.22
Taylors Beach South	2.21	2.52	3.16	3.43	4.30
Farm	2.03	2.25	2.70	2.88	3.49
Mission	2.07	2.30	2.75	2.94	3.56
Butler Bay	2.15	2.39	2.86	3.06	3.71

Wave setup values are to be added to the predicted storm tide levels. These values are:

- 50 year ARI = 0.2m;
- 100 year ARI = 0.35m; and
- >100 year ARI = 0.45m.

An important outcome of the study is the determination of appropriate design levels for new properties in the coastal townships. These design levels are summarised in Table E2.

Table E2– Property Design Levels for New Properties

Location	Property Design Level (m AHD)	
	Shoreline Area	Inland Area
Allingham	4.93	3.58
Cassady Beach	4.92	3.57
Dungeness	4.67	3.20
Lucinda	4.62	3.27
Taylors Beach North	4.79	3.44
Taylors Beach South	4.82	3.47

Following the prediction of storm tide level and wave setup predictions, the extent of inland inundation was determined using a single two-dimensional hydrodynamic model of the coastal mainland areas. From these predictions of storm tide inundation, an assessment of the hazard to the local community and its infrastructure was undertaken. The expected inundation and hazard posed to each community was then mapped.

The onshore modelling results show that Dungeness and Lucinda may become completely isolated during any storm tide event with an ARI equal to or greater than 50 years, due to inundation of the Lucinda Road. These townships may become completely inundated under an ARI event of 10,000 years and would need to be evacuated before significant sea level rise occurred.

Taylors Beach may become isolated during a storm tide event with an ARI equal to or greater than 100 years. Significant inundation of the township itself is not predicted for events less than the 10,000 year ARI event. Evacuation would need to occur prior to an event of this magnitude.

Allingham should remain accessible during all surge events smaller than the 10,000 year ARI event. Shallow inundation of Ingham Forrest Beach Road is predicted during the 50 year ARI and the 100 year ARI storm tide events, however it should remain trafficable. Cassady Beach and the south-eastern portion of Allingham are predicted to be inundated during the extreme events of 1,000 years and 10,000 years ARI. During these extreme events properties close to the beachfront may need to be evacuated.

Other issues which may arise as a result of inundation include:

- Softening of road pavement structures during extended periods of inundation. It is expected that pavements should return to their pre-inundation strengths once sufficient drying of the subgrades has occurred; and

- Possible flotation of septic tanks, or damage to their treatment systems and release of essentially untreated sewage into saturated absorption trenches.

Detailed emergency management procedures were produced to assist during a storm tide emergency. This included the development of emergency response maps in accordance with the Department of Emergency Services National Storm Tide Mapping Model for Emergency Response.

A public awareness strategy was undertaken to reduce the risk involved with storm tide events. This strategy included:

- The development of public awareness brochures and delivery to all coastal town residents; and
- Holding two community forums to inform residents of the risks involved with storm tide and the emergency management procedures in place.

It is recommended that a continuing public awareness strategy be developed.

A preliminary investigation of the joint probability of storm tide and freshwater flooding was undertaken. In this assessment historical flows in the Herbert River were compared to predicted storm surge and wave setup levels for ten selected historical cyclone events. This showed that although storm surge appears to precede Herbert River flooding no direct relationship between the two events could be inferred from a preliminary study and that a more detailed investigation would need to be undertaken before any conclusions could be made.

1. Introduction

Hinchinbrook Shire is located on the downstream reaches of the Herbert River. The Herbert River catchment is approximately 9000km^2, flowing from its upper reaches in the Atherton Tablelands to the Coral Sea. A large proportion of this catchment is World Heritage Area, Fish Habitat Area or National Park. Cyclones are frequent in the Hinchinbrook Shire area, and usually precede flooding in the lower Herbert River. Landfall of cyclones, coupled with high tides, have the potential to cause significant damage to infrastructure and property, as well as personal injury.

Hinchinbrook Shire Council obtained funding from the Federal and Queensland State Governments to undertake a Storm Surge Study of the Hinchinbrook Shire coastal areas. The funding was made available through the Natural Disaster Risk Management Studies Program, administered in Queensland by the Department of Emergency Services.

The stated aim of the Storm Surge Study was to identify and quantify areas of risk associated with storm surges which impact on the coastal communities of Hinchinbrook Shire, in particular at Lucinda, Dungeness, Taylors Beach, Allingham and Cassady Beach, and also the populated areas of Great Palm Island.

Connell Wagner with specialist sub-consultants Lawson and Treloar was commissioned by Hinchinbrook Shire Council on 15 March 2002 to undertake the Storm Surge Study. The specific objectives of the Study were to:

- Identify the true risk of storm surge inundation along the coastal region extending from Cassady Beach in the south to Dungeness in the north of Hinchinbrook Shire. It also investigated the south and western coastal regions of Great Palm Island, approximately 30km east of Cassady Beach (see Figure 1 and Figure 2). This was undertaken by predicting extreme water levels, and the effects of actual storm surge inundation combined with wave penetration across the specific coastal areas;
- Identify critical areas of concern arising from climatic changes;
- Identify storm surge inundation by preparing storm surge hazard mapping; and
- Develop appropriate planning measures and response plans.

The Study has included the following tasks:

- Predicting extreme water levels for the Hinchinbrook Shire Council coastline jurisdictions, using official cyclone statistics;
- Modelling actual storm surge inundation using state of the art (improved) wind field models and site specific numerical hydrodynamic modelling, which represent the coastlines offshore features and shape of coastline;
- Determining combined wave penetration using estimates of extreme wave conditions to improve the original calculation of wave setup effects across specific coastal areas;
- Identifying critical areas of concern arising from climatic changes, expanding coastal development, increasing population areas and the effects to essential services and Council infrastructure;
- Preliminary analysis of the probability of joint occurrence of storm surge and Herbert River flooding;
- Mapping the storm surge inundation by preparing storm surge and hazard mapping indicating the extreme water levels for all affected areas;
- Developing storm tide emergency response procedures for inclusion into the Hinchinbrook Shire Counter Disaster Plan; and
- Producing emergency response maps in accordance with the Department of Emergency Services National Storm Tide Mapping Model for Emergency Response.

As this study's results are important for counter-disaster planning, it was important to include prediction of 'extreme' water levels. In a freshwater flooding context, predictions usually include up to the so-called "Probable Maximum Flood" (PMF), although in the coastal context, PMF has no meaning. For this Study the extreme event (the "Probable Maximum Surge" event) that has been adopted will be referred to as the 10,000 year ARI event.

This study has also included a preliminary analysis of the issue of freshwater flooding, and its joint occurrence with storm surge. This issue is discussed further in Section 18.

2. Study Area

The Storm Surge Study includes all populated coastal areas within Hinchinbrook Shire Council. The Study Area extends from the mouth of Palm Creek, south of Cassady Beach to the Seymour River in the north. This includes the communities of Cassady Beach, Allingham, Forrest Beach, Taylors Beach, Lucinda and Dungeness (refer to Figure 1 and Figure 2). The study also includes the areas of Farm, Mission and Butler Bay on Great Palm Island.

The Hinchinbrook Shire coastline has a large, interconnected creek and river system including:

- Palm Creek;
- Victoria Creek;
- Gentle Annie Creek;
- Herbert River;
- Seymour River.

The Study Area is predominantly formed of open coast beaches that are provided with some protection by offshore islands and reefs. The beach faces are typically flat with extensive inter-tidal zones that are exposed at low tide. The offshore areas are relatively shallow and provide significant attenuation through bed friction and wave breaking. The back-beach area is generally low and may be protected by a low frontal dune. Where dunes occur, the dune system generally does not extend landward more than the frontal dune itself.

Storm tide penetration into major waterways provides the possibility of significant inflow of ocean water behind the coastal beach front and the opportunity for those high waters to inundate existing development.

3. Historical Flooding

Although cyclones have caused flooding in the region, for example, cyclone Justin in 1997, much of the historical flooding has been caused by rainfall and associated local runoff, rather than storm surge. High ocean water levels may also be caused by east-coast lows, but these systems have not been addressed in this study and affect southern Queensland more frequently than this region. Significant coastal damage with some overtopping of the back-beach area can be caused by the severe waves that occur with many cyclones.

There have been no known major cyclones that have affected the area recently. However, local residents advise that the frontal dune at Allingham was overtopping during cyclones in 1927, 1938, 1948 and 1956. According to Bureau of Meteorology records, the 1927 cyclone crossed the coast near Cairns with a central pressure of 971hPa and caused significant human life loss and property damage, principally from flooding. At Allingham, waves are reported to have overtopped the front dune and were typically knee high in the swale behind the dune.

4. Previous Studies

Experience in Hinchinbrook Shire has been that cyclone surge is generally small, though nuisance flooding occurs on the elevated high tides near Lucinda. For this reason no previous comprehensive study of storm tide has been undertaken for the region. Site specific studies were undertaken by Connell Wagner in 1978 as part of design investigations for the Lucinda Sugar Wharf. On the other hand, significant flooding can occur in the Herbert River during cyclonic events and the effects of those events on coastal communities has been extensively investigated. The most recent of the flooding studies is currently being completed by WBM.

5. Available Data

A range of data items were required to undertake the Storm Surge Study including:

- Data to set up and calibrate to numerical storm surge model;
- Data to establish a numerical wave model; and
- Data to set up the onshore inundation model, prepare inundation maps for affected coastal areas and assess the hazard posed to the local community from storm tide events.

The following sections detail the data collected and used on this project.

5.1 Bathymetric Data

Bathymetric data was required to describe the topography of the seabed and coastline over the area of the proposed numerical modelling systems. Charts AUS 826, 827, 828 and 829 were used. This data was digitised to provide a digital terrain model from which the numerical surge and wave model grids were prepared.

5.2 Tidal Information

Tides in this region are predominantly semi-diurnal (therefore there are two high and two low tides each day). The most representative site is Lucinda (Offshore), for which tidal planes are presented in Table 1.

Table 1 Tidal Information

Tide	Tidal Level (m LAT)	Tidal Level (m AHD)
Highest Astronomical Tide (HAT)	3.89	2.05
Mean High Water Springs (MHWS)	2.91	1.07
Mean High Water Neaps (MHWN)	2.11	0.27
Mean Sea Level (MSL)	1.89	-0.05
Mean Low Water Neaps (MLWN)	1.53	-0.31
Mean Low Water Springs (MLWS)	0.74	-1.10

This information is presented in The Official Tide Tables & Boating Safety Guide (2002), prepared by the Department of Transport, Queensland. All data is to Chart Datum which is Lowest Astronomical Tide (LAT). Datum AHD is 1.844m above Chart Datum. Lucinda tidal constants were used for all mainland sites analysis. However, for sites on Great Palm Island, tidal constants for Albino Rock were adopted.

5.3 Wave Data

The Beach Protection Authority (BPA) recorded wave data at their Townsville site between May, 1975 and December, 1987. This wave data was collected in a depth of approximately 20m at 19°10'S, 147° 04'E. The highest wave height (H_s) recorded at this installation was 2.76m during cyclone Kerry in March, 1979. In general, two hourly (1975 - 1981) or four hourly (1981 - 1987) recordings of 20 minutes duration, were made in analogue form. Hence it is possible that peak storm wave conditions were not recorded.

5.4 Cyclone Data

Cyclone track data was required to describe the characteristics of historical cyclones that have affected the Hinchinbrook Shire coastal region. Although some data was held in L&T archives, the Bureau of Meteorology advised during the course of this study that all data is now available from their web site in digital form. This data was downloaded for this study. Additionally, cyclone impacts have been discussed generally with Mr Jim Davidson and Mr Jeff Callaghan of the Bureau of Meteorology and data presented in the Bureau's Impacts Report was provided to assist this study.

Generally, cyclone track data has improved in quality since about 1959 when satellite imagery and over-the-horizon radar sampling provided better records of important parameters. Events occurring since 1959, and which have had a significant effect in the study area, have been included.

Appendix A lists all cyclones included in this investigation that have occurred since 1959, and which have passed close by the study area. Those cyclone tracks are plotted in Figure 3.

5.5 Storm Surge Data

Calibration of the proposed storm surge model was required to provide confidence in simulated storm surge results. Two severe events, Cyclone Althea and Cyclone Winifred were identified in this area since 1959 where reliable water level data had been recorded. Prior to this time, data is less reliable. Reliable water level records for these events were only available for the port of Townsville. Therefore the surge model was developed to include Townsville, with sufficient model extent and resolution to ensure physically realistic surge development at Townsville.

Recorded water level and predicted tide data for Cyclone Althea and Cyclone Winifred is presented in Figure 4 and Figure 5. This data was provided by the Department of Transport, Queensland and was used for model calibration/verification.

5.6 On-Shore Survey Data

The following survey data was used during the study for on-shore areas:

- **Photogrammetric survey from Herbert Resource Information Centre**
  The Herbert Resource Information Centre (HRIC) provided photogrammetric survey information of the entire Hinchinbrook Shire. This information did not include bathymetric data for the rivers and creeks.

- **Detailed survey from Herbert Resource Information Centre**
  The HRIC also provided detailed survey information of the populated areas of the Hinchinbrook Shire coastline including Lucinda, Dungeness, Taylors Beach, Allingham and Cassady Beach. This detailed survey included topographic and infrastructure details for these populated areas.

- **Bathymetric data from the TUFLOW model developed by WBM**
  Topographic data from the Herbert River TUFLOW model was provided by WBM. The bathymetric data from this model was used where the photogrammetric data did not contain such information.

- **Roughness data from the TUFLOW model and aerial photos**
  Roughness data was provided with the TUFLOW model. This data was extended to cover the entire study area using aerial photo analysis.

5.7 Other Data

Information was also obtained from previous scientific studies of the area. A number of these previous reports are discussed in Section 4. Anecdotal observations made by local community members also assisted in gaining an important historical perspective on past cyclone events.

Study team members conducted a number of site inspections where they visited Council, investigated the study area and spoke to local residents. Some of the key activities that were undertaken during this site visit included:

- SAG Meeting with Council officers and representatives to ascertain Council's expectations of the study and to determine what data Council had available for use in the study;
- Thorough inspection of all of the areas at risk; and
- Meetings with emergency response officers who provided information on experiences with cyclone events in the area.

In addition, local residents provided anecdotal information on previous cyclonic events.

Historical flows in the Herbert River were obtained from the Department of Natural Resources and Mines (NRM). This data was used for the joint probability analysis of storm surge and Herbert River flooding.

6. Study Approach

The purpose of this study was to develop detailed storm tide statistics at selected sites within the Council's areas of responsibility and then to assess and map their associated hazard to the local communities. The following sites were selected:

- Cassady Beach;
- Allingham;
- Taylors Beach South;
- Taylors Beach North;
- Lucinda;
- Dungeness;
- Farm (Great Palm Island);
- Mission (Great Palm Island); and
- Butler Bay (Great Palm Island).

There are two basic approaches that can be adopted to determine storm tide statistics. They are to:

- Hindcast historical cyclone events using actual cyclone tracks and tides; or
- Analyse the historical cyclone track data to develop a parametric description of variables such as central pressure, track direction and distance from the study area in terms of probabilities of occurrence. This task is followed by a series of cyclone simulations that provide basic time series of surge, wave and wind data, for example. These time series are then used in a Monte Carlo analysis in which cyclones are generated according to the parameterised cyclone wave climate. Estimated parameters for each simulated cyclone event are determined by interpolation/extrapolation from the base simulation results.

Both approaches produce time series of parameters that are subjected to extremal and correlation analyses. However, the Monte Carlo approach lends itself to the preparation of data covering much longer periods of time, and because ARI up to about 10,000 years were required for this study, the Monte Carlo procedure was adopted. All water level recurrence statistics were based on a 10,000 years period of simulations.

Once storm tide level and wave setup predictions have been made it was necessary to determine the extent of inland inundation. In general, this is not simply a matter of adopting the storm tide level at the coast and projecting it inland at a constant level. There are numerous physical features that can influence the inland propagation of an elevated storm tide, and it is important to take these into consideration.

The coastal areas of Hinchinbrook Shire contain an extensive, low-lying, interconnected network of streams and rivers that can influence the inland propagation of a storm tide. It was determined that the most appropriate way to assess the inland inundation was to develop a single two-dimensional MIKE 21 hydrodynamic model of these coastal areas.

The topography of Great Palm Island however, rises directly from the coastline to inland mountains such as Mount Lindsay and Mount Bentley. It was therefore considered that storm tide level prediction at the coastline of the selected areas was sufficient to assess the extent of inundation.

From these predictions of storm tide inundation, an assessment of the hazard to the local community and its infrastructure was assessed.

7. Cyclone Data Analysis

7.1 General

The Monte Carlo procedure requires that a range of historical cyclone simulations be undertaken to provide basic time series input data for interpolation/extrapolation. As part of this process it was necessary to describe each simulated cyclone in a general way, choosing the following principal parameters:

- Track direction;
- Distance from the study area centroid (taken to be Lucinda);
- Central pressure and forward speed; and
- Based on historical cyclone data.

In this analysis, only those historical cyclones affecting the area over the 42 years period from 1959 to 2000 were considered. Based on previous experience, only those cyclones passing within a defined zone of influence were included in parameter analyses. This zone of influence was chosen on the basis of those cyclone tracks that might produce the highest storm surges along the study area coastline. For this study, the zone of influence was defined as being the area between latitudes 15°30'S and 21°30'S and longitudes 143°30'E and 149°30'E. Lucinda is located at 18°33'S, 146°20'E, see Figure 3.

7.2 Track Direction

An inspection of the available cyclone track data led to the decision to adopt two direction categories:

- From north-east to south-west (south-westward); and
- From north-west to south-east (south-eastward).

These are generally equivalent to coast crossing and coast parallel tracks, respectively. The selection basis is related to cyclone track direction within Hinchinbrook Shire region. Although many cyclones do not wholly fit these descriptions, each of the 50 identified cyclones could be placed satisfactorily in one of these two direction categories.

On this basis, (23/50 =) 46% of cyclones were classified as south-westward and (27/50 =) 54% south-eastward.

7.3 Track Distance

Due to the clockwise rotating wind field structure of a cyclone, track location relative to the coastline is an important characteristic when determining the impact a cyclone will have on a coastal location. For example, a south-westward moving cyclone that crosses the coast to the north of this region will cause a strong storm surge to occur due to onshore winds as the cyclone crosses the coast, whereas a south-westward tracking cyclone crossing to the south of the site will cause offshore winds that push water away from the coastline. For cyclones of similar central pressure and forward speed, the inverse barometer effect and the strength of the cyclonic winds in the study region are proportional to the distance the cyclone is from the site. Wind direction is also dependent on track location and for cyclones that pass within approximately 30km of the site, full reversal of wind direction will occur as the cyclone passes the site.

This parameter is more complex than one might expect, because it is interlinked with central pressure and location north or south, east or west of the Hinchinbrook Shire. This issue is also related to the clockwise rotating wind field structure of cyclones. For example, within the adopted 6° of latitude/longitude extent from the Hinchinbrook Shire region, the lowest central pressure may not occur when the cyclone is closest to the Hinchinbrook Shire. Second, a cyclone passing 100km north of the Hinchinbrook Shire region may cause greater surge in the study region than a similar cyclone passing 100km to the south. The differences in outcome also depend on seabed topography.

For this study, track distance was defined by simplifying each cyclone track into a linear track within the study area and determining the radial distance from this track to Lucinda. If a cyclone significantly changed intensity as it passed through the selected zone of influence, the distance to the most intense section of the cyclone track was chosen.

For shore crossing (south-westward) cyclones, tracks that passed north of the site were defined as positive distances; those that crossed to the south were defined with negative track distances. Similarly, for shore parallel (south-eastward) cyclone tracks, those that passed offshore of the site were defined as being positive, while those that travelled over land (west of the coastline) were defined as being negative. These parameters are described in Appendix A.

Figure 6 shows the tracks used in the 54 basic cyclone simulations used to prepare time series data for the Monte Carlo analyses and indicates the above definition of positive and negative track distances.

7.4 Forward Speed

Forward speed may influence cyclone surge in two ways. First, the cyclonic winds may be increased by this speed on the south-eastern side of the cyclone and decreased on the north-western side. Second, when forward speed is close to the celerity of long waves $(\sqrt{gd})$, a resonance state can develop which causes an increased surge. Wind field changes would also affect waves near the Hinchinbrook Shire Coast.

Average forward speeds were estimated in the region near the Hinchinbrook Shire. The results are presented in Appendix A, separately for south-westward and south-eastward tracking cyclones.

7.5 Cyclone Central Pressure

Central pressure is the cyclone parameter that has the dominant impact on wind speed. Representative cyclone central pressures for those cyclones that were assessed to have had the most significant impact on the site were determined in the Hinchinbrook Shire region and analysed separately for all cyclones and also for the south-westward and south-eastward tracking cyclones. Results are presented in Table 2.

Table 2 Cyclone Central Pressures for Selected Populations of Historical Cyclones

ARI (years)	SW Population		SE Population		All Cyclones Since 1959	
	CP (hPa)	95% CL (hPa)	CP (hPa)	95% CL (hPa)	CP (hPa)	95% CL (hPa)
5	984	9	990	4	978	7
10	973	13	985	6	971	9
20	964	17	981	8	964	11
50	952	23	975	11	955	14
100	943	28	971	12	948	16
500	922	38	961	17	932	22
Data Points	23		27		50	
Years of Analysis	42		42		42	

The results show that south-westward tracking cyclones (coast crossing) are generally more severe than coast parallel cyclones.

Note that Walsh and Ryan (2000) advise that current climate change investigations show that there is unlikely to be an increase in coast crossing cyclone severity.

7.6 Greenhouse Related Climate Change Issues

At the commencement of the study, two documents provided the information most relevant to this matter. They were:

- Climate Change in Queensland under Enhanced Greenhouse Conditions, Second Annual Report (1998-1999) prepared by CSIRO Atmospheric Research; and
- Walsh, K.J.E. and Ryan, B.F (2000): Tropical cyclone intensity increase near Australia as a result of climate change. Journal of Climate, Vol.13.

The (1998-1999) CSIRO report is now superseded by the (1999-2000) report.

The main issues are:

- What is the likely magnitude of change, if any, in MSL over a 50 years planning period (say)?;
- What is the likely change in cyclone occurrence frequency, if any?; and
- What is the likely change in cyclone intensity, if any?

7.6.1 MSL Rise

The issue of Mean Sea Level (MSL) rise is addressed by Walsh and Ryan (2000). Walsh is also an author of the CSIRO reports and this matter does not appear to be addressed in the CSIRO report. The increase in MSL advised is 0.2m over 50 years. A range of 0.1m to 0.4m was advised in the CSIRO (1998-1999) report as part of discussions on storm surge analyses for Cairns.

From discussions with Council it is understood that, a MSL rise of 0.3m over the next 50 years has been adopted. This parameter should be re-assessed on a decadal basis, or as substantial new information became available. This is consistent with basic hydrodynamics and with recommendations in CSIRO (1999-2000).

7.6.2 Cyclogenesis Changes

Section 3 of CSIRO (1999-2000) addresses the likely (though not definite) changes in cyclone activity in the Queensland region. This CSIRO report also discusses the Monte Carlo procedure, which is consistent with the overall study approach adopted for this study.

Parameters proposed by CSIRO for inclusion in the Monte Carlo analysis are central pressure, radius to maximum winds, forward speed and coast parallel and coast crossing cyclones. CSIRO have analysed data for the Hervey Bay region using a regional extent similar, but not identical, to that adopted for this study. There is no definitive basis for this choice, both are realistic; the basic assumption being that cyclone parameters within the adopted region are similar throughout the adopted region. Note that CSIRO's main purpose in their Section 3 was to examine design wind speeds over land and cyclone filling was also considered by them.

7.6.3 Extreme Event Analysis

Although CSIRO discusses frequency of cyclone occurrence on the basis of coast parallel and coast crossing cyclones, they do not appear to describe cyclone intensity recurrence in terms of these separate populations, see Figure 3.5 of CSIRO (1999-2000).

CSIRO (1999-2000) adopt the Generalised Pareto Distribution (GPD) to describe the frequency of recurrence of cyclones with specific central pressures. CSIRO choose the GPD rather than the more common Extreme Value Type 1 (EXV1) distribution (termed Gumbel by CSIRO) because the GPD has an upper limit extreme value. They also state ' . . . that the GPD also has an advantage over the Gumbel distribution in that all available data are used to fit the distribution rather than just the extreme value within a specified time interval'. That statement is correct only when annual minimum central pressures are used, for example, in a Gumbel analysis procedure. Using the EXV1 (same theoretical formulation as Gumbel) though, all data is used in either the Method of Moments, Least Squares or Maximum Likelihood Method. Moreover, there is the issue of adopting a physically realistic minimum central pressure. CSIRO do not specify how this should be done, or whether they did for the CSIRO (1999-2000) report. However, their Figure 3.5 implies a minimum central pressure of about 940hPa for the Hervey Bay region.

For this study a minimum central pressure of 920hPa was adopted for present day cyclone simulations.

7.6.4 Forward Speed

This study has analysed cyclone forward speed in a manner very similar to that applied by CSIRO. However, the parameters have been developed separately for coast parallel and coast crossing cyclones for this study. The probability density function was described as a cumulative frequency distribution, see Appendix A, and sampled using random numbers within the Monte Carlo analysis.

7.6.5 Radius to Maximum Winds (Rm)

CSIRO (1999-2000) propose 30km. Note that the BoM report that cyclone Kerry had a very large Rm, closer to 90km at it's maximum size. A radius of 40km was used for this study. Additionally, the sensitivity of the results to choice of Rm was examined.

7.6.6 Direction of Approach

A similar procedure to that of CSIRO (1999-2000) has been followed in this study. However, a theoretical distribution was not fitted to the data, as was adopted by CSIRO, but rather a cumulative probability distribution based on the actual occurrences was used, see Appendix A.

Distance from the study area was also included in this study using a statistical description.

7.6.7 Changes in Cyclogenesis

Estimated changes in cyclone intensity are summarised in Table 3.4 of CSIRO (1999-2000). For the Queensland region cyclone central pressures are likely to reduce by 5hPa, on average, over the next 50 years.

The CSIRO (1999-2000) report discusses the point that cyclones with central pressures greater than 985hPa may not change, whereas cyclones more intense than 985hPa may increase in intensity by more than the average 5hPa.

For this study, two Monte Carlo analyses were undertaken. They were:

- An analysis based on existing cyclone data, to which could be added 0.3m (to be confirmed) for projected MSL rise over the next 50 years. Results prepared from this investigation exclude any Greenhouse related rise in MSL, but include a note that a MSL rise (0.3m) is to be adopted.

- An analysis based on an average increase in cyclone intensity of 5hPa, to which could be added 0.3m for projected MSL rise over the next 50 years.

7.7 Basic Cyclone Simulations

The parameters discussed in Sections 7.1 to 7.4 were used to describe 54 basic cyclone simulations. The results of these simulations were used to prepare time series surge data for the subsequent Monte Carlo analyses. A summary of the adopted parameters is provided in Table 3. Six additional cyclones were included to investigate the effect of the radius to maximum winds and astronomical tide level or surge height.

Table 3 Parameters Adopted for Basic Cyclone Runs

Run No.	Track Direction	Track Distance (km)	Vf (m/s)	CP (hPa)	Comments
1	SW	100	4	950	-
2	SW	100	4	970	-
3	SW	100	4	990	-
4	SW	100	8	950	-
5	SW	100	8	970	-

Run No.	Track Direction	Track Distance (km)	Vf (m/s)	CP (hPa)	Comments
6	SW	100	8	990	-
7	SW	40	4	950	-
8	SW	40	4	970	-
9	SW	40	4	990	-
10	SW	40	8	950	-
11	SW	40	8	970	-
12	SW	40	8	990	-
13	SW	-40	4	950	-
14	SW	-40	4	970	-
15	SW	-40	4	990	-
16	SW	-40	8	950	-
17	SW	-40	8	970	-
18	SW	-40	8	990	-
19	SW	-100	4	950	-
20	SW	-100	4	970	-
21	SW	-100	4	990	-
22	SW	-100	8	950	-
23	SW	-100	8	970	-
24	SW	-100	8	990	-
25	SE	150	6	950	-
26	SE	150	6	970	-
27	SE	150	6	990	-
28	SE	150	12	950	-
29	SE	150	12	970	-
30	SE	150	12	990	-
31	SE	75	6	950	-
32	SE	75	6	970	-
33	SE	75	6	990	-
34	SE	75	12	950	-
35	SE	75	12	970	-
36	SE	75	12	990	-
37	SE	-75	6	950	-

Run No.	Track Direction	Track Distance (km)	Vf (m/s)	CP (hPa)	Comments
38	SE	-75	6	970	-
39	SE	-75	6	990	-
40	SE	-75	12	950	-
41	SE	-75	12	970	-
42	SE	-75	12	990	-
43	SE	-150	6	950	-
44	SE	-150	6	970	-
45	SE	-150	6	990	-
46	SE	-150	12	950	-
47	SE	-150	12	970	-
48	SE	-150	12	990	-
49	SE	0	6	950	-
50	SE	0	6	970	-
51	SE	0	6	990	-
52	SE	0	12	950	-
53	SE	0	12	970	-
54	SE	0	12	990	-
55	SE	0	6	950	Radius to Max. Winds = 30km
56	SE	0	6	950	Radius to Max. Winds = 20km
57	SE	0	6	950	Radius to Max. Winds = 10km
58	SE	0	6	970	Water Level = MSL
59	SE	0	6	970	Water Level = MSL+1.0m
60	SE	0	6	970	Water Level = MSL-1.0m

The basis for selection of these basic simulation cyclones was their overall representation of severe, but not extremely rare cyclones. Because of the limited number of these basic runs, it was important that a reliable basis for describing the parameters of the greatest number of simulated cyclones was developed.

8. Numerical Storm Surge Modelling

8.1 Model Setup

The numerical current and storm surge modelling package applied to this investigation was the Delft3d hydrodynamic modelling system. This system provides a third order finite difference solution to the equations of mass and momentum conservation. It uses an alternating direction, implicit solution scheme.

The model has been applied to many investigations throughout Australia by L&T. It includes tidal and wind forcing, wetting and drying and turbulence model eddy viscosity terms. The model also includes spatially variable bed friction. Roughness height was set to be 0.03m in the sea and 0.1m on land. The system also has a range of other modules, such as an advection-dispersion module, which can be operated in parallel with the hydrodynamic module.

Wind setup develops across the nearshore area as the result of interfacial shear between the wind and sea surface and the consequent onshore currents. The Coriolis acceleration acting on northward flowing currents may also cause a storm surge component. Setup is inversely proportional to water depth, directly proportional to fetch and proportional to the square of wind speed. A large area model was established to ensure physically realistic development of these currents and surge. This model area extended north to Innisfail, south to Bowen and seaward beyond the 200m depth. Grid sizes vary from about 100m near the coastline to 500m offshore and at the northern and southern ends of the model.

Two other features of the Delft3d model were important to this study. Firstly, the model has an advanced curvilinear grid system. This grid system enables preparation of a grid which better follows the natural curvature of waterways such as Hinchinbrook Channel, between Hinchinbrook Island and the mainland, to the north of the immediate study area. Preparation of the grid in curvilinear form reduces the so-called stair-case problem of fixed grid size rectangular grids, which tend to falsify bed friction along narrow waterways that are not closely aligned with the grid. It also allows fine grid resolution in these narrow waterways and near the coastline, whilst allowing a coarser grid further away where high resolution is not required.

Secondly, a cyclone crossing the coast and passing to the south of the study area will drive water northward into Halifax Bay where some flow constriction between the coast and the Palm Islands may occur. When this happens, there will be a natural tendency for some back-flow near the seabed to occur, as well as horizontally at different locations. This flow structure can be described better by three dimensional modelling, which also allows better application of wind stress to the water column.

Horizontal grid sizes down to about 100m were used near the shoreline. Three vertical layers were used.

Wind fields were computed from the available historical cyclone track data for model calibration/verification and from idealised cyclone track data for the basic Monte Carlo simulations. The wind and pressure fields were prepared using the Holland wind model developed for the Bureau of Meteorology. This model is considered to provide the most realistic description of cyclonic wind fields for the Australian region.

8.2 Model Verification

Two historical cyclones were selected for model verification. They were Cyclone Althea in 1971 and Cyclone Winifred in 1986 (refer to Figure 4). Peak surge for Cyclone Althea was recorded as approximately 2.9m at Townsville Harbour and occurred close to low tide. For Cyclone Winifred, peak surge was about 0.49m at Townsville Harbour and occurred close to high tide (refer to Figure 5). Water levels returned to normal levels quickly during both those events.

Figure 7 and Figure 8 show that the modelled surge peak was reproduced well for both cyclones.

Figure 9 shows the extent of storm surge in the Hinchinbrook region, near the time of peak surge, produced by Cyclone Althea. A surge in the order of 1m occurred near Taylors Beach, but, being near low tide, water levels did not exceed those of a high astronomical tide.

This outcome shows that the model system can be used confidently to predict cyclone surge in the Hinchinbrook region.

9. Monte Carlo Analysis Procedure

9.1 Analysis Process

Previous sections have discussed the fifty-four basic model cyclone surge simulations that were undertaken to produce basic time series for the Mote Carlo analysis. These results provided time series of cyclone surge over 72 hour periods at intervals of 0.5 hours.

In reality, cyclone central pressures will vary considerably and coastline crossing may be any distance from the Hinchinbrook Shire, or cyclones may run parallel to the coast, offshore or inland of the Hinchinbrook Shire region. Furthermore, the phases of peak surge relative to high and low water of the astronomical tides will be random. Extreme water levels could be determined from very long term tidal records, which would include cyclone occurrences, but these are not available for the Hinchinbrook Shire coast. A practical alternative is to perform a Monte Carlo modelling exercise.

Monte Carlo modelling requires the generation of a large number of simulated cyclone events. These simulated cyclones are generated by randomly selecting parameters from distributions created from the analysis of historical cyclones, which have affected the area (as described in Section 6).

Because the historical data showed that south-westward tracking cyclones exhibited different characteristics to the south-eastward moving cyclones, they were considered separately. Distributions of historical cyclone parameters for characteristics including distance to landfall, forward speed and central pressure were developed for both cyclone data populations, see Appendix A and Table 2.

Fifty cyclones were defined as being significant cyclonic events occurring in the region since 1959. This suggests that the average inter-arrival time of cyclones that affect the Hinchinbrook Shire coast is 0.84 year. Of these, 46% are expected to be coast-crossing (south-westward) while 54% are expected to be coast-parallel (south-eastward) tracking cyclones. Coast crossing cyclones are normally more severe than coast parallel cyclones in this region.

Once track direction had been selected, the simulated cyclone was given other cyclone parameters. Track distance from the Hinchinbrook Shire coast and forward speed were selected using random numbers to select values from the distributions of cyclone parameters, see Appendix A.

Central pressures were determined independently by sampling randomly and fulfilling the central pressure versus probability of non-exceedance distributions determined from the cyclone data. This was done using the analytical expressions (Extreme Value Type 1) representing the best fit to the data. They were:-

South-westward Cyclones

$0.0797 \times (p - 933.1) = \ln(-\ln P)$

$P = 1-1/\lambda R$

$\lambda = 0.548$

South-eastward Cyclones

$0.1696 \times (p - 995.5) = \ln(-\ln P)$

$P = 1-1/\lambda R$

$\lambda = 0.643$

where

R is average recurrence interval (years)
λ is average number of cyclones per year
p is cyclone central pressure
P is probability of non-exceedance

Central pressures higher than 1001hPa were discarded as these events were not considered to be cyclones. Similarly, central pressures lower than 920hPa were discarded. This was a slightly arbitrary choice, but is lower than any cyclone that has affected the Queensland area and recognises that sea temperatures limit the possible minimum central pressure. These modelling processes were checked by calculating the average recurrence interval relationship(s) of simulated central pressures against that of the historical data (see Table 3). The agreement was good. Long term mean atmospheric pressure was adopted to be 1010hPa. This parameter affects the computation of the wind field.

Because the hydrodynamic model was set up to calculate storm surge only, suitable time-series of tide elevations were required to allow the calculation of total water elevations (storm tide) during the simulated cyclones. Nineteen years of astronomical tides at half-hourly intervals were predicted using the so-called Canadian tidal package (Foremann, 1977) and tidal constants for Lucinda and Albino Rock provided in Australian National Tide Tables, 2002. This period of time allowed for recession of the lunar nodes along the plane of the ecliptic and the associated changes in tidal range.

Random numbers were used to select a time series of tidal levels from any one of the nineteen years and any of the months between December and May (the typical cyclone season for this area). In this manner the correct arrival time structure was formed and cyclone arrival times and tides varied randomly.

Random numbers were used to select other cyclone parameters based on distributions of historical cyclones in the region of influence, see Appendix A. Once all the parameters (track direction, minimum track distance to site, forward speed and central pressure) of each simulated cyclone were determined, time series of storm surge were interpolated from the fifty-four base simulation runs. Total water level (storm tide) for the event was then calculated by adding the storm surge time-series to the time series of randomly selected tidal levels.

In addition to the basic fifty-four simulations undertaken to provide input to the Monte Carlo analyses, it was important to test the sensitivity of the analyses to tide levels and radius to maximum wind speed. Wind set-up is inversely proportional to water depth. Figure 10 compares the results for four values of Rm - 10km, 20km, 30km and 40km. The track selected for this comparison was SE with 0km track distance from the coastline. This track would typically cause the greatest storm surge in the Hinchinbrook Coast region. The simulation was undertaken with a central pressure of 950hPa. The results show that for this site Rm can be important, with peak surge varying over a range of about 0.5m. The greatest surge was caused by a Rm of 40km, which was adopted for this study.

Storm surge (wind setup component) is dependent on water depth. Therefore in the very nearshore region, where tide range has a significant influence on water depth, it is an important issue. However, in the Monte Carlo based analysis, the basic simulations were undertaken at MSL (the most common tide level). This means, in a simple topographical region, that surges occurring at high tide would be over-estimated, whereas those occurring near low tide would be underestimated to some extent. Figure 11 compares surge time series at Cassady Beach for three tide levels adopted for describing the effect of tide level on storm surge - MSL-1.0m, MSL and MSL+1.0m. The result is consistent with the concepts discussed above and shows that the effect is in the order of 10%. Those results were applied on a site specific basis to the Monte Carlo procedure for the tidal levels of each simulated cyclone event.

9.2 Results

Simulations of 10,000 years were undertaken and the simulated time series of results stored. Monte Carlo simulation results were analysed by ranking them in terms of peak event storm tide and then undertaking an Extreme Value Type 1 Analysis using the method of moments.

The outcome of the analyses is presented in Table 4 in terms of datum AHD. Water levels are presented for the 50 years average recurrence interval (ARI) and longer. Time series plots of combined astronomical tide and storm surge water level for the selected ARI events are presented in Figure 12. These time series provide a basis for assessment of inundation times and for assessment of inland flows. Storm tide is dominated by the astronomical tide and peak water levels caused by cyclone surge will persist only for durations up to six hours

Table 4 Peak Storm Tide at Selected Locations

Location	**Peak Storm Tide (m AHD)***				
	50yr ARI	**100yr ARI**	**500yr ARI**	**1000yr ARI**	**10000yr ARI**
Allingham	2.32	2.63	3.25	3.51	4.37
Cassady Beach	2.31	2.62	3.25	3.51	4.36
Dungeness	2.13	2.37	2.87	3.07	3.75
Lucinda	2.09	2.32	2.78	2.97	3.60
Taylors Beach North	2.19	2.49	3.11	3.37	4.22
Taylors Beach South	2.21	2.52	3.16	3.43	4.30
Farm	2.03	2.25	2.70	2.88	3.49
Mission	2.07	2.30	2.75	2.94	3.56
Butler Bay	2.15	2.39	2.86	3.06	3.71

** Excl. Wave Setup and Greenhouse Related Climate Change*

Previous studies have shown that at more frequent ARI (eg. 20 years), high water levels are more likely to be produced by high astronomical tides together with other meteorological events such as east coast lows, rather than cyclone storm surge.

The highest storm tides generally occur at Allingham, with significant variations in water levels amongst the other sites within the study area.

Table 5 presents equivalent results for the case where central pressures have been reduced by 5hPA in order to represent possible Greenhouse related climate change cyclonic response. No MSL rise has been included.

Table 5 Peak Storm Tide at Selected Locations (incl. Greenhouse Related Central Pressure Change)

Location	Peak Storm Tide (m AHD)**				
	50yr ARI	100yr ARI	500yr ARI	1000yr ARI	10000yr ARI
Allingham	2.48	2.80	3.45	3.72	4.62
Cassady Beach	2.47	2.79	3.45	3.72	4.61
Dungeness	2.25	2.51	3.04	3.26	3.98
Lucinda	2.20	2.44	2.94	3.15	3.83
Taylors Beach North	2.33	2.66	3.32	3.60	4.50
Taylors Beach South	2.36	2.69	3.38	3.66	4.59
Farm	2.13	2.37	2.85	3.06	3.72
Mission	2.18	2.42	2.91	3.12	3.79
Butler Bay	2.27	2.51	3.03	3.24	3.94

*** Excl. Wave Setup and Greenhouse Related MSL Change*

10. Wave Setup

10.1 General

Wave setup is caused by the conservation of wave momentum flux in the surf zone, Goda (2000). The shoreward decrease in wave height in the breaker zone leads to a gradient in wave radiation stresses and a consequent increase in the 'still water level' in the shoreward direction. Wave grouping causes some fluctuations in this still water level. At the breaker line there is a setdown.

This shoreward increase in water level is called wave setup and it increases non-linearly in the shoreward direction. It is greatest at the shoreline and is additional to storm tide. Illustration 1 demonstrates this phenomenon.

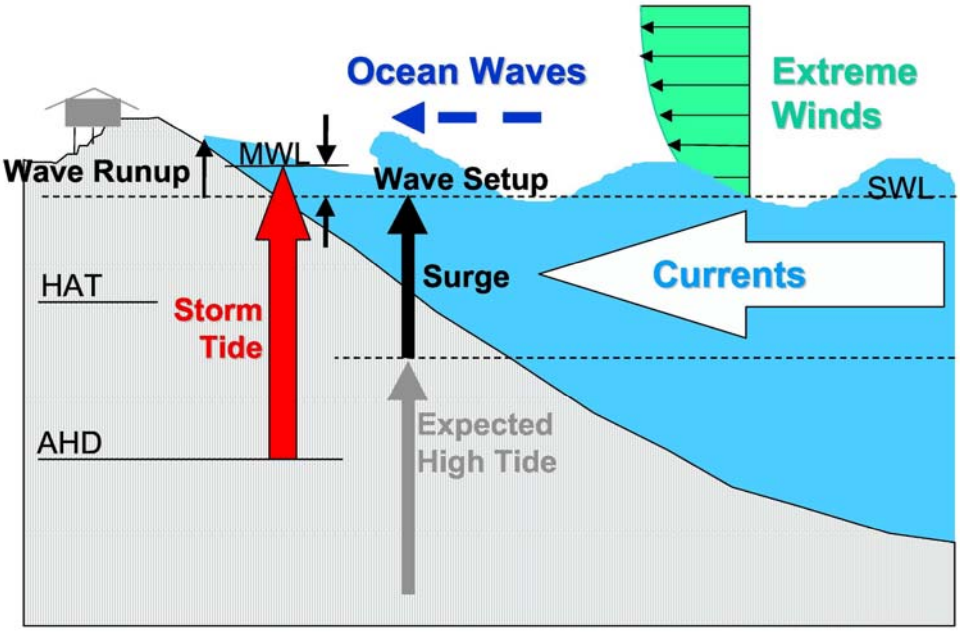

Illustration 1 – Water Level Components of a Storm Tide

Wave setup depends upon 'nearshore' wave height. Ten historical cyclone events were selected for this investigation. They were:

- Althea (1971);
- Keith (1977);
- Winifred (1986);
- Una (1973);
- Ivor (1990);
- Otto (1977);
- Joy (1990);
- Flora (1964);
- Gertie (1971); and
- Tessi (2000).

These cases were selected because offshore wave heights were expected to be high and they represented different offshore wave directions.

10.2 Wave Modelling

The first step in this investigation was to set up an offshore wind/wave model based on the second generation wind/wave modelling system, ADFA1, developed by Dr Ian Young of the Australian Defence Force Academy. The model is based on a numerical solution of the Radiative Transfer Equation and is applicable in water of any depth. It predicts the evolution of the directional wave energy spectrum as a result of the processes of wind energy transfer, propagation, refraction, shoaling, bed friction, white capping, nonlinear wave-wave inter-action and depth limited wave breaking. Output from the model includes significant wave height, dominant wave direction and spectral peak wave period at selected grid points.

The wind/wave model was established on a 5km computational grid with an origin at 20^{0}04’S:146^{0}02’E.

The model extended northward to approximately 17^0 15’S and eastward to approximately 149^0 E. A time step of 7.5 minutes was adopted to ensure physically realistic wave propagation and growth. The frequencies selected for spectral description ranged from 0.03Hz to 0.423Hz - a total of fifteen frequencies being used. Directional resolution was based on sixteen divisions of the compass. The Holland wind model, developed by the Australian Bureau of Meteorology for tropical regions of Australia, was used to calculate cyclone wind fields from the cyclone track parameters. The model extent and spatial resolution are considered more than adequate for the description of peak storm wave conditions arising from tropical cyclones. Generally, little wave energy propagates through the Great Barrier Reef and wave generation occurs within the reef lagoon for waves affecting this shoreline.

For wave modelling of cyclones a Rm of 40km was adopted. This is representative of the dimension of the larger cyclones in this area.

In addition to this regional model, a finer grid (150m) SWAN wave model was established for the inshore area. The SWAN model is part of the Delft3d system and was developed at the Delft Technical University. It includes natural bathymetry, offshore wave input (parametric or spectral), wind input, refraction, shoaling, bed friction, full frequency-direction wave propagation, white-capping, wave/current interaction and solutions to 3rd order. Fine grids can be nested within coarser outer grids. The model system is considered to be one of the most reliable. Output from the ADFA model was used as boundary input data for the SWAN wave propagation model, which transferred offshore waves to the nearshore region extending along the coastline from Hinchinbrook Island to Cassady Beach.

Output locations from the SWAN model were located in approximately 6m of water depth (MSL) at each of the study sites. Figure 13 shows the SWAN model for the study area, together with example output for cyclone Althea.

Using a surf zone model with beach profiles provided by the EPA, wave setup was calculated using SWAN model output as input in water depths of about 6m, where wave setup is negligible. Wave setup was calculated following the procedures developed by Goda (2000). In general, wave setup is 10% of the nearshore significant wave height at this site. The relationship depends on seabed slopes and wave period.

10.3 Model Verification

In order to verify the ADFA and SWAN wave models, output for Cyclone Winifred was compared with peak wave conditions recorded at the Townsville Waverider Buoy and provided by the BPA. Additionally, the daily synoptic charts prepared by the Bureau of Meteorology were inspected to estimate offshore wave direction. The peak recorded conditions during Cyclone Winifred in February 1986 were:

H_s = 2.5 m
T_p = 7.4 seconds
Direction E (estimated from synoptic charts)

Modelled results for Cyclone Winifred, at the closest model output location to the location of the Townsville waverider buoy, were as follows

H_s = 3.3 m
T_p = 7.2 s
Direction = 86^0

Modelled wave heights at this location were marginally higher than were recorded during Cyclone Winifred. This can be accounted for as the model output location was further to the north, closer to Cyclone Winifred, than the waverider buoy location. This is confirmed by the increase in wave height at model output points north (closer to the eye of the cyclone) of the chosen verification model output buoy site. Additionally, it is unlikely that the Waverider buoy recorded peak wave conditions.

This verification indicates good agreement between modelled results and recorded data for this event and gives confidence in the modelling system to predict wave parameters reliably for this location.

10.4 Results

Results from the wave modelling, as described above, are presented in Table 6. As can be seen from these results, the nearshore wave height (6m depth) is typically only 2.5m with wave direction between 75^0 and 160^0 at peak wave conditions at onshore locations in this study area. This seems low, but is the outcome of wave propagation across extensive, shallow seabed areas inside the offshore islands and the Great Barrier Reef. Butler Bay, which is located on the more exposed southern coast of Great Palm Island, typically has a higher wave climate than onshore locations and locations on the western coast of Great Palm Island. At this particular location the model produced waves up to 3.5m from the south-east, for Cyclone Althea. Cyclone Althea produced the maximum offshore wave height scenario that was modelled.

Wave setup analysis for these peak inshore wave conditions and subsequent extremal analysis leads to the following wave setup heights:

- 50 year ARI = 0.2m;
- 100 year ARI = 0.35m; and
- >100 year ARI = 0.45m.

These are the wave setup heights occurring jointly with storm tide at the specified ARI.

Table 6 Wave Modelling Results at Surf Zone

Cyclone Event	Wave Model (offshore) Output			Inshore Location	SWAN (inshore) Output		
	H_s (m)	T_p (s)	Direction (°)		H_s (m)	T_p (s)	Direction (°)
Althea	8.4	13.3	121.3	Butler Bay	3.45	13.75	146.76
				Farm/Mission	1.97	8.35	205.06
				Lucinda	2.45	6.50	130.74
				Taylors Beach	2.76	10.71	128.52
				Cassady Beach	2.59	7.37	136.29
Winifred	5.8	11.3	101	Butler Bay	3.16	10.71	138.84
				Farm/Mission	1.21	10.71	187.25
				Lucinda	2.30	5.74	102.10
				Taylor's Beach	2.62	6.50	100.23
				Cassady Beach	2.53	7.37	108.73
Keith	3.2	7.3	70	Butler Bay	1.61	7.37	130.12
				Farm/Mission	0.77	2.71	22.61
				Lucinda	2.16	7.37	78.54
				Taylor's Beach	2.29	7.37	72.35
				Cassady Beach	2.15	5.74	86.22
Una	3.9	9.1	168	Butler Bay	2.67	7.37	164.94
				Farm/Mission	1.77	5.74	230.55
				Lucinda	2.19	7.37	146.28
				Taylor's Beach	2.36	7.37	144.48
				Cassady Beach	2.09	6.50	147.90
Ivor	2.6	6.8	126	Butler Bay	2.37	7.37	145.63
				Farm/Mission	0.83	7.37	200.08
				Lucinda	1.84	6.50	129.03
				Taylor's Beach	2.00	6.50	129.30
				Cassady Beach	2.06	7.37	125.59
Joy	4.4	9.8	110	Butler Bay	2.92	9.46	140.90
				Farm/Mission	1.00	9.46	196.87
				Lucinda	2.23	6.50	111.75
				Taylor's Beach	2.43	6.50	112.18
				Cassady Beach	2.49	9.46	115.60

Cyclone Event	Wave Model (offshore) Output			Inshore Location	SWAN (inshore) Output		
	H_s (m)	T_p (s)	Direction (°)		H_s (m)	T_p (s)	Direction (°)
Gertie	3.7	8.5	105	Butler Bay	2.70	8.35	139.46
				Farm/Mission	0.90	8.35	185.10
				Lucinda	2.20	6.50	104.58
				Taylor's Beach	2.35	5.74	106.12
				Cassady Beach	2.44	8.35	111.57
Tessi	4.8	8.9	133	Butler Bay	3.11	9.46	149.83
				Farm/Mission	1.54	8.35	207.26
				Lucinda	2.34	8.35	134.19
				Taylor's Beach	2.63	8.35	130.71
				Cassady Beach	2.51	7.37	131.32
Flora	2.6	7.5	122	Butler Bay	2.40	7.37	144.53
				Farm/Mission	0.81	7.37	199.79
				Lucinda	1.83	6.50	127.83
				Taylor's Beach	1.99	6.50	128.43
				Cassady Beach	2.06	7.37	124.79
Otto	Local Sea Only			Butler Bay	2.34	6.50	178.06
				Farm/Mission	2.00	5.74	247.40
				Lucinda	1.99	6.50	152.69
				Taylor's Beach	2.07	6.50	155.92
				Cassady Beach	1.70	5.74	163.45

10.5 Inclusion of Wave Setup in Water Level Statistics

In order to include wave setup in total water level in a manner that is physically realistic, it would be necessary to undertake a detailed joint occurrence study of cyclone surge and wave setup so that relative phasing and duration were explicitly included. Such a study would also be best undertaken together with rainfall/runoff modelling so that fresh water flows in the principal estuaries were included also.

This study has shown that offshore wave direction does not have a major influence on nearshore wave heights and setup, at least over the commonly occurring wave directions. Wave setup heights are to be added to the storm tide statistics of Table 4 and Table 5 at the specified ARI.

Note that peak storm wave heights for storms of 20 or more years ARI will not vary greatly and that peak wave setup for those events will be similar. It is also unlikely that peak wave setup will occur at the same time as peak storm tide. The wave setup heights presented in Section 10.4 reflect that characteristic.

11. Property Design Water Levels

To determine appropriate design levels for new properties in the coastal townships of Dungeness, Lucinda, Taylors Beach and Allingham, it is recommended that the following be considered:

- 100 year ARI storm tide (as shown in Illustration 1, this figure includes astronomical tide, storm surge and wave setup);
- A 0.3m allowance for long term sea level rise (Greenhouse); and
- Where appropriate, wave runup. (Note that where applied, wave runup includes the effects of wave setup). See description below.

It is recommended that the 100 year ARI storm tide be adopted as the appropriate design level for coastal property in the study area. This level should include the possible Greenhouse related MSL rise of 0.3m, that is, 0.3m should be added to the 100 year ARI water levels given in Table 4. Wave setup for 100 years ARI is 0.35m (see Section 10.4) but is included in wave runup, see below.

For coastal sites it is necessary to include wave runup for habitable and commercial floor levels of buildings in order to prevent ocean inundation. Wave runup is difficult to assess because it depends on seabed slope near the breaker line. Based on survey profiles provided by the EPA, the foreshore slope at this elevation, is approximately 1:9. However, steeper slopes could be achieved if wave action were to cause rapid sand transport offshore and an erosion escarpment forms in the back-beach area.

Previous studies have found the relationship of Holman (1986) to be realistic for runup calculation on natural and near natural shoreline areas. It is:

$R_2 = (5.2\beta+0.2)H_s$

Where R_2 is the wave runup height exceeded by only 2% of waves
β is bottom slope near the Still Water Line.
H_s is significant wave height in 6m depth

From Section 10.4 the nearshore wave height (H_s) (6m depth) is typically about 2.5m at peak wave conditions in this study area. The outcome then is that a realistic 'estimate' of R_2 is 2m. This includes wave setup implicitly. Note that:

- Wave setup is manifested as a relatively steady increase in water level that occurs at the coastline and can propagate into bays and creeks. It varies over times in the order of hours. Hence it was included in the boundary time series input to inland inundation modelling; and

- Wave runup is what one sees at the beach when a wave breaks and rushes up the beach face. It varies metres over times of a few seconds. It is only important at the coastline, unless it causes significant overtopping and filling of the area behind the beach. This can lead to a drainage problem if the water can not escape back to the beach.

On this basis, floor levels for coastline sites should be set at 2m above the 100 year ARI water level for the specific locations indicated in Table 4, plus a 0.3m MSL rise. The definition of a coastline site is a little unclear, but applies to all properties on the frontal dune itself and extends back 100m or to the first row of buildings. For example at Allingham, design level would be 4.93m AHD (2.63m + 0.3m + 2.0m).

Where properties are 'back' from the shoreline, waves may overtop the back beach area and propagate in-land, for an unspecified distance, but in the order of 100m. Calculation of floor levels in this area is complicated by the presence of a high frontal dune crest level and lower "swale" area behind. This topography inhibits the description of wave propagation inland and corresponding design floor levels. As discussed in Section 13, the volume of water expected to overtop the frontal dunes has been calculated and taken into account by the onshore modelling.

Further inland at this example Allingham site, and where hinterland modelling shows that the coastline storm tide of 2.63m AHD remains applicable, the floor levels should be set at 2.63m + 0.35m + 0.3m + 0.3m freeboard = 3.58m AHD. The freeboard allowance includes uncertainties and potential MSL rise caused by possible Greenhouse related climate change (Note that the 0.3m freeboard allowance used was specified by Council officers).

Table 7 presents the results of similar design water level calculations for the remaining sites included in the scope of this study.

Table 7 Design Levels for Properties Affected by Storm Surge

Location	**Levels (m AHD)**	
	Coastal Site	**Wave Overtopping not Affecting Inland Site (Including Freeboard)**
Allingham	4.93	3.58
Cassady Beach	4.92	3. 57
Dungeness	4.67	3.32
Lucinda	4.62	3.27
Taylors Beach North	4.79	3.44
Taylors Beach South	4.82	3.47

Areas in which these floor levels should be applied are displayed in Figure 46 to Figure 48.

12. Historical Cyclone Storm Tide Hindcasting

To provide further description of the occurrence of cyclone events and the associated elevated water levels in the study area, modelling of historical cyclones was undertaken. Ten historical cyclone events were selected for this investigation. They were:

- Althea (1971);
- Keith (1977);
- Winifred (1986);
- Una (1973);
- Ivor (1990);
- Otto (1977);
- Joy (1990);
- Flora (1964);
- Gertie (1971);
- Tessi (2000);

These cases were selected from the fifty cyclones since 1959 that were used in cyclone parameterisation for the Monte Carlo simulations, because of their proximity to the study area and the expected occurrence of onshore winds to produced elevated water levels and wave heights. The numerical storm surge model was used to simulate each historical event and produce a time series of storm surge induced water level changes.

Because the hydrodynamic model was setup to calculate storm surge only, suitable time-series of tide elevations for each event were required to allow the calculation of total water elevations (storm tide) during the simulated cyclones. Historical astronomical tides at half-hourly intervals were predicted using the so-called Canadian tidal package (Foremann, 1977) and tidal constants for Lucinda provided in Australian National Tide Tables, 2002.

Wave modelling of the chosen historical cyclones was undertaken to describe nearshore wave conditions for each event as described in Section 10. Investigation of wave setup within the surfzone was then undertaken using a nearshore surfzone model utilising Goda's breaking wave algorithm and the nearshore results produced. Wave conditions, and therefore wave setup, were assumed to be a maximum at the time of the peak storm surge. These conditions are a realistic representation of the maximum intensity of the cyclone effect at the site.

Wave setup heights were added to the combined storm tide (astronomical tide + storm surge) to produce time series of total water level for each historical cyclone. These time series are presented in Figure 14. These figures indicate that during the past 42 years since 1959, combined storm water levels have not reached higher than a "normal expected" astronomical high tide at Lucinda. This can be attributed to significant historical storm surges coinciding with the low tide phase of the astronomical tide and the lack of a closer proximity severe cyclone in this time period.

13. Foredune Wave Overtopping Discharges

The local topography along some sections of the coastline within the study area is characterised by the presence of a frontal dune crest level and lower "swale" area behind. For this lower swale area behind the frontal dune, waves may overtop the back beach area and propagate in-land and produce a significant contribution to flood flows arising from inland surge penetration. These flows are in addition to the influence of rainfall/runoff and storm ocean levels.

To assist with the flood modelling of coastal areas, as described in Section 14, an estimation of these foredune wave overtopping discharges has been undertaken. The method used to calculate the discharges was based on wave overtopping for the design of flexible revetments (PIANC, 1992). The application of these formulae required the extrapolation of results for the flatter slopes of the natural foredune in comparison to constructed revetments. Wave parameters were chosen from wave modelling of historical events (Section 10) to represent realistic wave conditions incident on the foredune at the time of peak storm conditions. A range of different water levels were investigated to produce overtopping discharges that could be applied to a time series of storm water levels for modelling purposes.

Results from the calculations are presented in Table 8 for the inhabited areas of Lucinda and Allingham. These locations exhibit the characteristic high foredune and lower "swale" area topography that is conducive to this type of foredune overtopping and consequent flooding, with slow drainage when the cyclone abates.

Table 8 Foredune Wave Overtopping Discharges

Location	Storm Water Level (m AHD)	Wave Overtopping Discharge ($m^3/s/m$)
Allingham	3.70	1.90
	3.50	1.30
	3.25	0.60
	3.00	0.20
	2.75	0.10
	2.67	0.08
	2.50	0.05
	2.25	0.00
Lucinda	2.67	1.90
	2.50	1.45
	2.30	0.80
	2.00	0.30
	1.75	0.20
	1.50	0.10
	1.25	0.05
	1.00	0.00

14. Storm Surge Inundation

Having predicted storm tide levels and wave setup along the coastline, it was then appropriate to determine resulting inundation of inland areas. In general, this is not simply a matter of adopting the storm tide level at the coast and projecting it inland at a constant level. There are numerous physical features that would influence the inland propagation of an elevated storm tide, and it was important to take these into consideration.

To assist in the prediction of inland inundation, a two-dimensional MIKE 21 hydrodynamic model of the coastline from south of Cassady Beach to north of Dungeness was developed. The topographic data adopted for this model was a combination of photogrammetric survey data, detailed ground survey and bathymetric data from WBM's TUFLOW model developed for the current "Herbert River Flood Study". The roughness data used in the TUFLOW model was extended to cover the entire model area.

The model boundary conditions were created from the results of the offshore modelling and consisted of the storm tide levels plus the wave setup data. All tide surge levels were calculated as a time series for a period of 72 hours. Wave setup values were added to the storm surge levels in these time series. Wave setup was assumed to rise from 0m at 0 hours and peak at the time of the peak storm tide, then decay to 0m at 72 hours. The wave setup data detailed in Table 9 was applied to the downstream boundary of all models.

Table 9 Wave Setup Estimates

Average Recurrence Interval	Wave Setup (m)
50 year	0.20
100 year	0.35
Greater than 100 year	0.45

In addition to these boundary series (applied at the "ocean" boundaries of the onshore model), additional inflows were included behind the foredune as described in Section 13. In this way, the onshore model incorporated (volumetrically) the additional discharge that could reasonably be expected to overtop the foredunes due to wave runup.

It should be noted that over the "coastal zone" (extending from the foredune to the first row of houses or 100m where no buildings exist), the additional allowances described in Section 11 should be applied to the predictions of the onshore modelling.

Inundation and hazard plans have been prepared for the following ARI:

- 50 years;
- 100 years;
- 500 years;
- 1,000 years; and
- 10,000 years.

Hazard was assessed using the criteria displayed in Illustration 2. This criteria is outlined in the SCARM report "*Floodplain Management in Australia, Best Practice Principles and Guidelines*" (2000). It should be noted that due to wave setup and runup effects at the coastline an extreme hazard category has automatically been applied extending back 100m from the coastline, or to the first row of buildings.

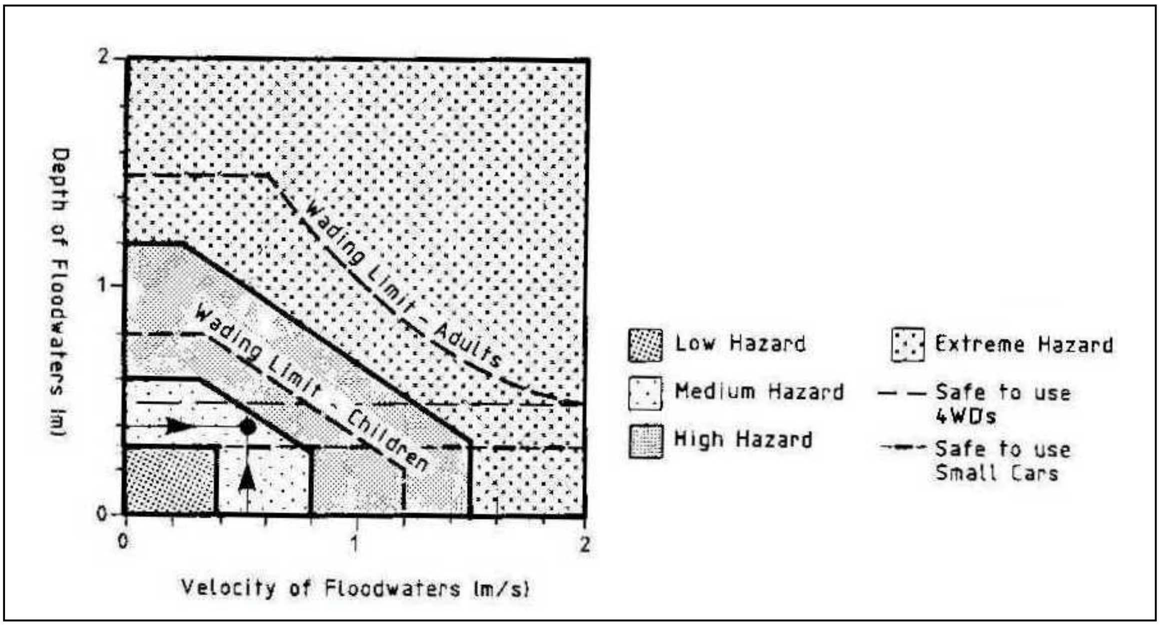

Illustration 2 – Estimation of Hazard

A series of plans have been prepared presenting the estimated inundation and hazard associated with each of these events. A series of three base plans have been produced covering:

- Lucinda and Dungeness;
- Taylors Beach; and
- Allingham and Cassady Beach.

The potential impact on population and infrastructure is discussed in detail in the following sections.

15. Factors Affecting Flood Hazard

As discussed in SCARM (2000), factors affecting flood hazard can be grouped into four broad categories:

- Flood behaviour (ie severity, depth, velocity, rate of rise, duration);
- Topography (ie evacuation routes, islands);
- Population at risk (ie number of people and developments, type of land use, public awareness); and
- Emergency management (ie forecasting/warning, response plans, evacuation plans, recovery plans).

It is appropriate to make comment upon all of these issues in general terms, before proceeding on to the specific hazard assessment.

15.1 Flood Behaviour

In accordance with risk management guidelines, this study has assessed and quantified the full range of potential storm tides, from the 50 year ARI event to the extreme 10,000 year ARI event (the "Probable Maximum Surge" event).

The rate of rise of ocean level associated with a storm surge is dependent on many factors, as discussed in Sections 7 to 10. Irish (1977) reported an indicative storm surge hydrograph based on Queensland data. Close to landfall, storm surge typically increases gradually, followed by a rapid rise to peak water level and an equally rapid fall, and then gradual decay. This general description is verified by Figure 7 and Figure 8. However, storm tide is dominated by the astronomical tide and peak water levels will persist only for durations up to six hours and rely on the coincident occurrence of high tide and storm surge.

However, the total time of inundation of any particular area is not only dependent on the time the peak surge is sustained, but also on the local drainage features once the ocean level has dropped. For example, in many of the residential areas, the inundation time will be dictated by the time the local drainage network takes to drain the excess water.

The hazard posed by the flood waters themselves is directly related to the depth of storm tide flooding and the velocity of the flow. SCARM (2000) presents a series of hazard categories related to the depth and velocity of flow, and the relative evacuation time. Whilst the latter factor will only be known by the counter-disaster managers, this report assigns an initial hazard assessment based on the first two factors, with the understanding that this will be adjusted upon consideration of evacuation measures. Depth of inundation maps have been prepared in critical areas, based on the following depth ranges (note that these figures do not include the wave runup applicable to the coastal zone):

- 0 to 0.25m;
- 0.25 to 0.5m;
- 0.5 to 0.75m;
- 0.75 to 1m;
- 1 to 1.25m;
- 1.25 to 1.5m; and
- >1.5m.

Inundation maps have been prepared for Hinchinbrook Shire under the following storm surge events:

- 50 year ARI (Figure 16, Figure 17 and Figure 18);
- 100 year ARI (Figure 19, Figure 20 and Figure 21);
- 500 year ARI (Figure 22, Figure 23 and Figure 24);
- 1,000 year ARI (Figure 25, Figure 26 and Figure 27);
- 10,000 year ARI (Probable Maximum Surge) (Figure 28,Figure 29 and Figure 30).

These figures are presented for the areas of Lucinda and Dungeness, Taylors Beach and Allingham and Cassady Beach respectively.

Hazard maps of the three areas have also been prepared. These maps are based on hazard categories of:

- Low hazard;
- Medium hazard;
- High hazard; and
- Extreme hazard.

The hazard maps have been prepared for the following surge events:

- 50 year ARI (Figure 31, Figure 32 and Figure 33);
- 100 year ARI (Figure 34, Figure 35 and Figure 36);
- 500 year ARI (Figure 37, Figure 38 and Figure 39);
- 1,000 year ARI (Figure 40, Figure 41 and Figure 42);
- 10,000 year ARI (Probable Maximum Surge) (Figure 43, Figure 44 and Figure 45).

15.2 Topography

The availability of effective access routes from flood-prone areas and developments can directly influence the resulting hazard when a flood occurs. Specific comment on access and evacuation are provided in Section 16.

15.3 Population at Risk

The degree of hazard and social disruption varies with the size of the population at risk. An estimate of the total population at risk (PAR) at critical locations has been made (refer Section 16).

The awareness of the population is typically related to past experiences with flooding, and regular public awareness campaigns.

15.4 Emergency Management

The emergency management procedures adopted will greatly affect the potential hazard to a community. Development of these procedures is discussed in Section 17.

16. Risk to Population and Infrastructure

16.1 Population at Risk

For the purposes of this risk/hazard assessment, all areas inundated under the 100 year ARI event and the Probable Maximum Surge event (10,000 year ARI) have been examined. The areas that have been examined are those on which buildings currently exist.

An attempt has been made to quantify the potential population at risk (PAR) under each of these events, based on the detailed survey information supplied by HRIC (exact survey date unknown), DCDB and Australian Bureau of Statistics (ABS) 1996 Census data. The methodology involved a count of all residential properties from the survey data and DCDB to provide the basis number of properties affected. Only properties within the medium, high and extreme hazard zones were included in this count. The ABS data was used to provide average numbers of persons per household. The combination of these two figures provides an indicative population figure for each zone. Table 10 and Table 11 below provide a summary of this data and the PAR for the 100 year ARI and 10,000 year ARI events respectively.

Table 10 PAR Per Zone – 100 year ARI

Area	Approximate No of Properties[1]	Ave Household Size[2]	Estimated PAR[3]
Dungeness	0	2.6	0
Lucinda	137	2.6	357
Taylors Beach	0	2.6	0
Allingham	0	2.6	0
Cassady Beach	0	2.6	0
		Total Population At Risk	**357**

Table 11 PAR Per Zone – 10,000 year ARI

Area	Approximate No of Properties[1]	Ave Household Size[2]	Estimated PAR[3]
Dungeness	6	2.6	16
Lucinda	329	2.6	856
Taylors Beach	93	2.6	242
Allingham	208	2.6	541
Cassady Beach	21	2.6	55
		Total Population At Risk	**1710**

Notes 1. Count based on detailed survey and DCDB and (not verified on site).
2. Source – Australian Bureau of Statistics – 2001 Census data.
3. Estimated PAR based on residential dwellings only and does not include persons residing in units or shops.

16.2 Risk to Infrastructure

16.2.1 General

The onshore modelling results (Figure 16 to Figure 30) show that Dungeness and Lucinda will become completely isolated during any storm tide event with an ARI equal to or greater than 50 years, due to inundation of the Lucinda Road. These communities could not expect external assistance during an event and would need to be evacuated before significant sea level rise occurred as all of these communities may become completely inundated under an ARI event of 10,000 years.

Taylors Beach may become isolated during a storm tide event with an ARI equal to or greater than 100 years. Significant inundation of the township itself is not predicted for events less than the 10,000 year ARI event. Evacuation would need to occur prior to an event of this magnitude.

Allingham would remain accessible during all storm tide events smaller than the 10,000 year ARI event when Forrest Beach Road is expected to be untrafficable. Shallow inundation of the road is predicted during the 50 year ARI and the 100 year ARI storm tide events. Cassady Beach and the south-eastern portion of Allingham are predicted to be inundated during the extreme events of 1,000 years and 10,000 years ARI. During these extreme events properties close to the beachfront may need to be evacuated.

16.2.2 Roads

Inundation of significant roads is expected to occur under all significant storm tide events. Flow velocities in most areas are expected to be minimal, in the order of 0 to 0.5m/s, and scour of road pavements or footpaths is unlikely to be a source of major road infrastructure damage in these cases. However roads which exist in medium, high or extreme hazard areas could experience flow velocities greater than 0.5m/s and these roads may be subject to damage.

Long term inundation of roads and footpaths (say 12 hours) may allow the underlying road pavements and subgrade to become waterlogged, resulting in a softening of the pavement structure. The pavement should return to pre-inundation strengths when the underlying pavement and subgrade has sufficiently dried. This may take some time after the surface water has receded.

When the inundation has receded it is recommended that vehicular use on recently inundated roads be restricted to single axle vehicles and emergency vehicles. Heavy or commercial vehicles, not required for emergency access, should be prevented from travelling on the affected roads until the pavements have regained sufficient strength. This time can vary considerably and is particularly dependent on the type of subgrade material. It is therefore recommended that Council engineers be consulted before unrestricted access is permitted.

16.2.3 Sewer

During wet weather, sewerage inflows tend to increase dramatically due to illegal stormwater connections and groundwater ingress. It could be expected that this would also be the case during periods of storm tide inundation.

Many areas at risk of inundation are not sewered and some damage to individual septic systems due to seawater ingress or flotation of septic tanks could be expected. Seawater ingress would stop or hinder treatment of sewage in the tank, and result in release of essentially untreated waste into adsorption trenches in saturated ground. Septic systems are typically private infrastructure (except for public toilets) but release of sewage has significant public health implications.

The sewage treatment plant at Lucinda is at risk from inundation under 500 year ARI or greater events. Inundation of this plant may cause release of essentially untreated waste into the storm tide receiving waters. It should be noted that GHD are currently undertaking a sewerage planning study of the area and it is possible that the Lucinda treatment plant may be removed as a result of this study.

Also of concern would be the sewerage pump/lift stations throughout the lower lying areas subject to inundation. Lift pumps which are at risk of inundation are located at the end of Bruce Parade in Lucinda and on Dungeness Rd in Dungeness. The pump station overflow systems would also be inundated, allowing salt water to enter the pump well. If the pumps remain in service, the shock loading from salt water influx could result in a complete loss of biological treatment performance at the sewage treatment plant. Following this shock loading, the quality of treatment plant outfall would initially have pathogen and pollutant levels similar to that of raw sewage. This would gradually return to normal levels after treatment bacteria have fully re-established, ie perhaps after six to twelve weeks of operation.

The pumps rely on electrical power, which may be interrupted during the inundation and could result in the overflow of raw sewage into waterways. Power supply to the pump stations is sourced via overhead transformers and is at no greater risk from inundation than general power supply failure (see Section 16.2.5). Switch boards and motor control cabinets are generally located at or near ground level and may require maintenance or replacement following seawater inundation.

16.2.4 Water Supply

The water supply system is typically located on elevated ground and or sealed underground infrastructure. All taps in areas of inundation, particularly those in public areas of the beachside parks, should be fitted with backflow prevention devices to prevent surge water inflows and contamination of the water supply.

16.2.5 Electrical and Communications

General

The electrical infrastructure in the areas includes overhead and underground reticulation owned by Ergon to provide electricity supply and underground reticulation owned by Telstra to provide communications and phone services.

Communications

The Telstra infrastructure is understood to consist generally of underground reticulation. The system is therefore designed and installed to be robust against the ingress of water. The pit and conduit system is regularly inundated with water as part of the natural storm water dissipation. The cables and cable joints used are grease filled which can be submerged in low level water with no adverse affects. The cable connection pillars, which are located above ground, are also sealed and positively pressurised to prevent the ingress of water, however are not submersible.

It is recommended that copies of the study drawings be provided to Telstra for review and assessment of any other infrastructure that may be at risk.

Electrical Supply

It is understood that Ergon infrastructure in the area of interest consists primarily of above ground reticulation, with several below ground cables. Below ground infrastructure is known to be installed near the Lucinda caravan park and at the Dungeness boat ramp particularly.

Overhead reticulation is suitably segregated from the rising water. The poles supporting the cables are able to withstand minor water flow around the base of the pole. Underground reticulation is robust against water due to the inherent resistance required for underground installations. However, susceptible points in the system include:

- Locations where overhead and underground reticulation is joined at connection boxes; and
- Ground mounted and low level equipment, which may not be waterproof including house meter panels, distribution pillars, padmount transformers and 11kV Ring Main Units (RMU).

Ergon has a series of cascading protection, which includes meter panels protected at distribution pillars protected at the transformer protected at the RMU protected at the zone substation. As the protection trip proceeds from meter panel to substation a greater area is affected by a loss of power. Power will not be able to be returned until the water level lowers and new equipment is installed. In the case of transformers and RMU's this may take 2 to 3 weeks for supply and installation. Ergon will determine the locations of the potential problem areas when final inundation drawings are available.

There is potential loss of life situations if the electricity is not shut off prior to water levels rising. The speed at which water rises and warnings, which may be available, will play an important role in assisting Ergon in the maintenance and safety of the network. However, the protection mechanisms are generally set to provide power shutoff in less than 1 second.

It is recommended that a full copy of the report be provided to Ergon for a comprehensive review of their infrastructure at risk of inundation.

16.2.6 Other Infrastructure

Public buildings at risk from inundation include the school in Lucinda, as well as the school and community centre in Allingham.

16.3 Risk Assessment

As outlined in Section 15, depth of inundation maps have been prepared for critical areas, based on increments outlined in SCARM (2000). Figure 16 to Figure 30 present the inundation mapping for the full range of storm tide events considered. Figure 32 to Figure 45 present the hazard mapping for the storm surge events under consideration. A review of hazard at each of the communities is provided below:

Dungeness and Lucinda

Under all storm tide events analysed, the (Herbert River delta) Enterprise Channel, Dungeness Creek and the lower reaches of Gentle Annie Creek are predicted to be substantially inundated. The storm tide would fill each of these systems, and the network of small creeks in the low-lying area behind Dungeness and Lucinda. This would cause inundation of Lucinda Road and isolate the two townships.

As a result of this isolation, the only access to the towns would be by boat (as helicopter access is unlikely during a cyclone event). During the smaller events (50 year and 100 year ARI), boat access may be possible along the inundated road corridor. If this were the case, high level flood markers should be provided along the road so boats could have a safe route to follow and could determine the depth of water. Some higher sections of the road may remain free from flooding and a side track should be cleared around these sections so boat access would be possible.

Dungeness
Under a 50 year ARI event, Dungeness would be located in a low hazard area. Dungeness Road would remain trafficable and buildings located in Dungeness may be inundated by up to 0.25m.

During a 100yr ARI event, Dungeness would still remain in the low hazard category. Dungeness Road may become inundated by up to 0.75m, isolating the township from Lucinda. Buildings in Dungeness may be surrounded by more water than in the 50 year ARI event, however the water depth would still be less than 0.25m in most cases.

Evacuation of Dungeness would need to occur in an event greater than the 100 year ARI storm tide event.

Under the Probable Maximum surge event, the entire town may become inundated (depth up to 1.5m) and Dungeness Road is likely to become covered by more than 1.5m of water.

Lucinda
In some areas, Lucinda Road is predicted to be inundated under all storm surge events greater than the 50 year ARI event. During the 50 year ARI event, most areas of Lucinda are predicted to be inundated with shallow water (up to 0.25m). Most roads would also be covered with up to 0.25m of water and trafficability would be reduced.

Due to their proximity to the beachfront, the properties located at the northern end of Dennis Parade, Rigby Street, Bruce Parade and Vass Street are susceptible to wave runup effects and have therefore been designated as extreme hazard. Most properties on Patterson Parade are also located within this extreme hazard zone.

Under the 100 year ARI event, Rigby Street, Bruce Street, Vass Street, Dennis Parade, and the northern and southern ends of Patterson Parade are likely to experience inundation, greater than 1.5m in some cases. Buildings on properties adjacent to these streets are also likely to become inundated. These properties would need to be evacuated before an event of this magnitude. Buildings and streets in the remainder of Lucinda may experience inundation of up to 0.5m.

Lucinda would need to be evacuated during any event greater than the 100 year ARI storm surge event.

Under the Probable Maximum Surge event, inundation would extend into all areas of Lucinda, with most streets and properties being inundated by more than 1.5m.

Taylors Beach
Storm surge events are able to enter Victoria Creek and cause inundation of the low-lying areas behind this township. Under events greater than the 50 year ARI event, this inundation would join with inundation from Gentle Annie Creek to the north of the township and may cause some inundation of Taylors Beach Road and isolation from Halifax and Ingham.

During the 50 year ARI event, only minor inundation some areas of Taylors Beach would occur, this inundation would remain in the low hazard category.

Under the 100 year ARI storm surge event, Taylors Beach remains in the low hazard category. The extent of inundation would be greater than for the 50 year ARI event, however inundation would remain below 0.25m depth. Access to the township may be cut at the low-lying section of Taylors Beach Road, near the intersection with Secret Harbour Road. Council may wish to consider raising this section of the road to provide trafficability under the 100 year ARI event. This would not need to be a specific exercise but may be incorporated into future roadworks along this road.

Under the Probable Maximum Surge event, the situation is predicted to worsen substantially. Properties on the western side of John Dory Street may be inundated by up to 1.25m of water. The remainder of Taylors Beach may be inundated by up to 0.5m of water, however most of the township would remain in the low hazard category.

During this extreme event, sections of Taylors Beach would remain in the low hazard category, it may be possible for residents to stay in these areas, or alternatively to be evacuated into Halifax and Ingham. This evacuation would have to be performed by boat due to the depth of inundation of Taylors Beach Road. Comments regarding boat access along Lucinda Road are also applicable in this case.

Allingham (Forrest Beach)
Properties along Fern Street are located within the extreme hazard zone caused by wave runup effects along the coastline.

During the 50 year ARI event, Allingham would be in the low hazard category and may be inundated by up to 0.25m of water in some areas close to the coastline. Access to the townships would remain open, but streets may be inundated by up to 0.25m of water.

Under the 100 year ARI storm surge event, the properties in Allingham would remain in the low hazard category.

Under the Probable Maximum Surge event, the community of Allingham would be completely isolated and surrounded by deep water (> 1.5m in most places). Inundation of properties along the coastal strip, as far back as the intersection of Beatts Road and Cedar Street, is likely to occur, with properties in Fern Street, Allamanda Avenue, Pine Street, Ash Street and Acacia Street being the worst affected. The remainder of Allingham would be in the low hazard category with inundation of up to 0.25m occurring in most areas. It may be possible to erect an emergency shelter in these areas, however wind effects from a high magnitude cyclone may cause problems.

Cassady Beach
Under the 50 year ARI storm surge event, access to Cassady Beach would be maintained, however the community has been assigned a hazard category of “extreme” due to the wave runup effects along the coastline. During the 100 year ARI event Cassady Beach Road may become untrafficable and evacuation before an event of this magnitude may be the best option.

Under the Probable Maximum Surge event access into and out of the area would be blocked. Evacuation should occur before an event of this magnitude.

Halifax

Halifax would be unaffected by all storm tide events less than the 10,000 year ARI event. Under this Probable Maximum surge event, properties on Mona Road between Shaws Avenue and Hoffenstetz Street may become inundated by up to1m of water. Access between Halifax and Ingham would remain open during all storm surge events.

Crystal Creek and Mutanee

Inundation modelling was not extended as far south as these communities, therefore the water levels predicted at Cassady Beach are the most appropriate levels to adopt for these areas.

Orpheus and Pelorus Islands

Water levels were not predicted at these sites specifically. For the western sides of these islands it would be appropriate to adopt the levels predicted at Taylors Beach (north). This is a more conservative approach than adopting the levels at Lucinda.

General Notes

It should be noted that the hazard estimates presented in this section are initial estimates based only on the mapping produced. The hazard classifications should be confirmed by the counter-disaster managers, taking into account local knowledge and emergency procedures.

17. Emergency Management

17.1 Emergency Response Mapping

Emergency Response Maps were prepared in accordance with the recently developed DES guidelines (National Storm Tide Mapping Model for Emergency Response). These maps categorised properties into evacuation zones, defined by colours, to be used in the event of an expected storm tide.

The maps were prepared using aerial photogrammetry survey. This data was contoured into 0.5m zones, above the HAT level, as defined by the DES guidelines. Once the contours were generated affected properties were grouped into blocks. This was done, as it will be easier to evacuate an entire block during an emergency situation than to identify specific properties to evacuate. Each block was rationalised to a single evacuation zone based upon:

- Ground levels of the individual properties;
- Access to and from the block; and
- The possibility of isolation of properties on the block.

These maps are presented in Appendix C.

It should be noted that the emergency response zones do not represent a real assessment of the storm tide risk, as they are based upon land level only. The real assessment of storm tide risk is presented in Section 16 and displayed in the inundation and hazard maps (Figure 15 to Figure 45)

The following discussion provides a summary of the zones applied to each town.

17.1.1 Dungeness

Dungeness Road is extremely low-lying, causing the township to become isolated during a Zone 1 storm tide event, therefore the entire township has been categorised as Zone 1.

17.1.2 Lucinda

The beachfront properties in Lucinda have been assigned a Zone 1 emergency response category as they are subject to potential wave impacts. A number of other low-lying blocks have been assigned a Zone 1 category, including:

- Houses located on Ferrero Street and Carr Crescent near Lucinda Road; and
- The two blocks on Keast Street that are bounded by Dungeness Road/Bruce Parade to the north and Hobbs Street to the south.

The remainder of Lucinda is categorised as Zone 2 as the land is all low-lying.

17.1.3 Taylors Beach

The properties along both the south eastern and north western ends of John Dory Street have been assigned a Zone 1 category as they are low-lying. Properties along Trevally and Barramundi Streets are categorised as Zones 4 and 5, and the remainder of the township is classed into Zones 2 and 3.

It should be noted that the Taylors Beach Road is low-lying and causes isolation of the township during any Zone 1 or greater storm tide event.

17.1.4 Allingham

The majority of Allingham is categorised as Zones 5 or 6. The beachfront properties are categorised as Zone 1 as they are subject to potential wave impacts. The remainder of properties are categorised as follows:

- Properties on Wattle Street, south of Pine Street, and those on the western side of Palm Street are categorised as Zone 4;
- A range of zones (from Zone 1 to 4) were used to classify properties on Forrest Drive and the northern end of Beatts Road;
- Zone 3 category covers properties on Willow Street and those on the western side of Allamanda Avenue; and
- The caravan park and properties on Acacia Avenue are categorised as Zone 2.

17.1.5 Cassady Beach

Cassady Beach has been categorised as Zone 1 due to a combination of the extremely low-lying Cassady Beach Road and the proximity of residences to the beachfront.

17.2 Tropical Cyclone Storm Tide Warning-Response System

The Bureau of Meteorology and the State Counter Disaster Organisation have recently released the fifth edition of the Tropical Cyclone Storm Tide Warning-Response System. The procedures outlined in this document are vital to any emergency response that Council and the State Emergency Services can have to a storm tide threat.

A summary of Connell Wagner's understanding of this procedure is as follows:

- Preliminary Storm Tide Warnings – given 24 and 18 hours prior to 100km/h wind gusts reaching the coast. There are no specific water level predictions available with these preliminary warnings, but advice is provided on what time these warnings can be expected;
- Storm Tide Warning – given 12 hours prior to cyclone landfall. At this stage, a predicted peak storm tide level is provided; and
- Follow-up Warnings – issued every 3 hours thereafter. Each of these warnings will update the predicted peak storm tide level based upon the latest cyclone information available.
- Final Storm Tide Warning – given approximately 6 hours after 100km/h winds have reached the coast. The timing of this warning will vary depending upon the cyclone characteristics. Emergency response personnel must not begin the 'return' stage of evacuation until this warning has been issued.

The Bureau of Meteorology will only issue quantitative storm tide warnings if the storm tide is expected to reach levels above HAT. All quantitative storm tide warnings will be issued as a level relative to Australian Height Datum (AHD).

17.3 Emergency Response Procedures

A workshop was held to determine the issues associated with storm tide emergency response in Hinchinbrook Shire. A summary of the workshop outcomes can be found in Appendix C.

A flow chart of the required storm tide emergency response procedures was developed to aid the Counter Disaster Committee during a storm tide event. A copy of this flow chart can be found in Appendix C.

17.4 Public Awareness Strategy

A brochure describing storm tides, their risk in Hinchinbrook Shire and the preparation residents can make for a storm tide event was developed. This brochure also described the emergency mapping process described in Section 17.1 and the importance of the mapping zones. This brochure was distributed to all costal town residents, with large scale copies of the emergency response maps displayed in the:

- Forrest Beach News;
- Taylors Beach Caravan Park & Store; and
- Lucinda Store.

The public awareness strategy also consisted of two public presentations of the study findings and the emergency response mapping and procedures.

17.5 Beach Town Access Road Inundation

To assist the Counter Disaster Committee in their decision to evacuate an investigation of the possible inundation times of the beach town access roads was undertaken. This involved the extraction of water level data adjacent to each of the access roads (Lucinda Road, Taylors Beach Road and Ingham Forrest Beach Road) and comparison to the road levels.

Table 12 Road Inundation Timing and Duration

	ARI (years)	Difference Between Peak Storm Tide and Inundation Time (hrs)	Approx Inundation Time (hrs)
Lucinda Road Road Level = 1.5m AHD	50	+1	4
	100	+0.25	7
	500	-0.5	10.5
	1000	-1.25	12.75
	10000	-1	9.5
Taylors Beach Road Road Level = 2.0m AHD	50	-	-
	100	-	-
	500	+3.25	2.25
	1000	+2	6
	10000	+0.25	>12
Ingham Forrest Beach Road Road Level = 3.0m AHD	50	-	-
	100	-	-
	500	-	-
	1000	-	-
	10000	+1.25	2.75

+ve values indicate inundation occurs after peak storm tide
-ve values indicate inundation occurs prior to peak storm tide

The road inundation timing shown in Table 12 indicates that, in most cases, inundation is expected to occur within approximately one hour of the peak storm tide level. During a cyclone it is expected that 100km/hr winds will reach the coast a number of hours before the eye of the cyclone, and the peak storm tide, cross the coast. As it is deemed unsafe for people to be outside during the occurrence of 100km/hr winds it is believed that inundation of the roads is not likely to occur during the available evacuation time.

It should be noted at this point that in some locations, comprehensive survey of pavement levels is not available. The adopted digital terrain model (upon which the information in Table 12 is based) draws upon the information available at the time. The figures in Table 12 could be refined in the future as more survey data becomes available.

18. Joint Probability Analysis (Storm Tide and Freshwater Flooding)

An important issue for emergency planning is the joint occurrence of river flooding and storm tide induced flooding. In the event that both of these flooding mechanisms occur concurrently, or within the influence of each other, the potential loss of life and damage to infrastructure may increase above that which would be expected from each mechanism in isolation.

For this study, a preliminary analysis of the joint occurrence of Herbert River flooding and storm surge was undertaken. The Department of Natural Resources provided historical flows in the Herbert River at Ingham. The Bureau of Meteorology provided the predicted flows from its URBS model for cyclones Winifred, Otto and Ivor. Where available the predicted flows were used, as these flows are not subject to interference from downstream water levels, which may be elevated due to storm tide occurrence.

The Herbert River flows were compared to the storm surge and wave setup for each of the historical cyclones simulated during the wave setup investigation (these cyclones are defined in Section 10). The results of these comparisons are presented in Appendix B.

As presented in the figures of Appendix B, no direct relationship between Herbert River flow and storm surge can be inferred from this preliminary study. It can be seen that in most cases the storm surge occurs prior to increase in Herbert River flow. However, the timing of this relationship does not appear to be constant and ranges from 0.5 days (Ivor) to 3.5 days (Tessi). This relationship does not occur for all cyclones assessed. Cyclone Una shows that no significant increase in Herbert River flows occurred following the storm surge occurrence, and during cyclone Otto, Herbert River flow increase and storm surge occurred simultaneously.

The relationship between Herbert River flooding and storm surge would need to be investigated by a more detailed study before any conclusions could be made.

19. Conclusions

This report describes the data, methods and results of an investigation of storm tide levels in the nominated Hinchinbrook Shire Storm Surge Study area situated on the North Coast of Queensland.

An analysis of historical cyclones that have affected the area was undertaken. Only cyclones recorded since 1959 were included. That data was analysed to provide statistical descriptions of the principal cyclone parameters.

Numerical storm surge and wave models were set up using bathymetric data prepared from available charts. The surge model was validated using historical storm surge data provided by the Department of Transport.

Numerical simulations of fifty-four selected base cyclones were undertaken. A further six sensitivity simulations were undertaken to describe the effects of tide level and radius to maximum winds. They provided time series of storm surge over periods of seventy-two hours at half hourly intervals. Those simulations provided basic data for cyclone surge descriptions used in a Monte Carlo analysis, together with the analysed historical data.

Cyclones and astronomical tides over a period of ten thousand years were simulated using this method. The resulting storm tide time series were analysed using the Extreme Value Type 1 distribution to provide peak storm tide levels corresponding to selected average recurrence intervals (ARI's).

It was noted that other meteorological events such as East Coast lows may also cause elevated ocean levels.

Wave setup was also investigated and likely water level increments determined. In addition, wave runup heights, which implicitly include wave setup, were determined specifically for locations along the beach frontal dune. Design levels for different locations in the coastal region that might be affected by storm tide have been provided for an example site. A procedure for undertaking similar calculations for other sites has been prepared.

The peak storm tide levels presented in Table 4 do not include an increment for possible MSL rise caused by Climate Change. This may be 0.2m by 2048 (CSIRO, 2000). Wave setup increments were added to peak storm design surge levels, other than for frontal dune sites where wave runup was included.

The modelling of historical cyclone events to provide coincident occurrence of storm surge, astronomical tide and wave setup heights was undertaken. These water levels were incorporated into the detailed onshore numerical model to quantify the onshore penetration of storm surge.

Wave overtopping discharges for the foredune area were provided to assist in the onshore inundation modelling of the study area.

Onshore inundation modelling was carried out using a 2-dimensional model of the coastal areas of the Shire. Results from the model were used to develop inundation and hazard maps for the coastal townships under the range of storm tide events selected.

The inundation and hazard maps were used to undertake a risk assessment for each of the townships. This assessed the potential risk to infrastructure including roads, sewer, water supply, electrical and communications systems.

To assist with emergency response in a storm tide event, a flow chart was developed outlining the emergency response procedures to be used. To further assist emergency response mapping of the coastal townships was carried in order to classify properties into zones which should be evacuated in the event of a storm tide.

A public awareness strategy was undertaken to inform the coastal town residents of the risk of storm tide impacts within their communities. This strategy involved the distribution of storm tide risk brochure to all coastal community residents as well as a number of public presentations of the study findings.

20. Recommended Future Work

The investigations undertaken to date have provided a storm tide risk assessment for the townships of Dungeness, Lucinda, Taylors Beach and Allingham and have identified appropriate fill levels to be adopted for future developments.

To further improve the understanding of storm tide hazard in the area and to develop strategies to reduce this hazard the following works are proposed:

- The Hinchinbrook Shire development guidelines should be revised to incorporate the recommendations of this report with respect to property development levels in coastal townships (as per Section 11).

- Upgrades to access roads and provision of emergency infrastructure be undertaken (refer Section 16).

- A continuing public awareness strategy should be undertaken. It is widely recognised that public awareness of risk and the applicable emergency procedures significantly affects the hazard associated with an event such as a storm tide.

- A more detailed investigation into the likelihood of coincident storm tide land river flooding should be undertaken. However, it should be noted that the preliminary investigations undertaken as part of this study were not able to identify a correlation at this stage.

21. References

Blain, Breman and Williams Pty Ltd (1977): Capricorn Coast Strategy Investigation, Livingstone Shire Council.

Bureau of Meteorology (2003): Tropical Storm Tide Warning Response System. Third Edition.

Capricorn Coast Beaches (1979): Beach Protection Authority, Queensland.

CSIRO (2001): Climate Change in Queensland under Enhanced Greenhouse Conditions. Third Annual Report 1999-2000.

Department of Transport, Queensland (2001): The Official Tide Tables & Boating Safety Guide.

Foreman, M.G.G. (1977): Manual of Tidal Heights Analysis and Prediction, Pacific Marine Science Report 77-10. Prepared for the Institution of Ocean Science, Patricia Bay, Victoria, B.C., Canada.

Goda, Y. (2000): Random Seas and Design of Maritime Structures. World Scientific. Advanced Series on Ocean Engineering, Volume 15.

Holman, R.A. (1986): Extreme Value Statistics for Wave Runup on a Natural Beach. Coastal Engineering, Vol. 9 No. 6, Elsevier Scientific Publishing.

Nelson, R.C. (1983): Wave Heights in Depth Limited Conditions. Sixth Australian Conference on Coastal and Ocean Engineering.

PIANC (1992): Guidelines for the Design and Construction of Flexible Revetments Incorporating Geotextiles in Marine Environment. Report of Working Group No. 21 of Permanent Technical Committee II.

Tropical Cyclone Impacts along the Australian East Coast from November to April - 1858 to 2000. Report Prepared by the Brisbane Office, Bureau of Meteorology.

Walsh, K.J.E. and Ryan, B. F. (2000): Tropical Cyclone Intensity Increase near Australia as a Result of Climate Change. American Meteorological Society.

Appendix A

Statistical Analysis of Historical Cyclone Data

Appendix B

Joint Probability of River and Storm Tide Flooding

Appendix C

Emergency Response Documentation

HINCHINBROOK SHIRE COUNCIL

Connell Wagner