SOPAC

Coastal Protection Feasibility Study

Final Report

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Executive Summary

This study is focused on coastal protection options for the Avarua – Avatiu area on the northern coastline of Rarotonga.

Several previous studies at this location have been carried out but their recommendations have not been fully implemented, probably due to cost considerations. This may be an advantage as the understanding of the critical differences between coastal processes on reef fringed coastlines and more conventional beach profiles, that are common in highly developed areas, was not well advanced at the time these studies were carried out and some recommendations were inappropriate.

The state of understanding of critical coastal processes in coral reef areas, in particular the understanding of wave setup and wave induced flows, is still at a basic level. As a result of this basic level of understanding, the only definitive design tool available is properly conducted 3-D physical modelling. Any coastal protection scheme should be modelled to verify its effectiveness prior to implementation.

A key outcome of this study is the provision of defendable design criteria in terms of reef flat water levels and wave heights. These are derived from a Monte Carlo modelling process for cyclones in the area and translation of the effects of these cyclones on the Avarua/Avatiu coastline. This process combines the variabilities inherent in cyclone effects including:

» Intensity of the cyclone

» Direction and travel speed of the cyclone

» Spatial extent of the extreme winds and waves

» Closest approach distance and relative location of cyclone path

Thus design conditions can be assigned a statistical return period or Average Recurrence Interval (ARI).

Wave setup, the process where large breaking waves pile water onto the reef flat resulting in reef flat water levels several meters above the ocean water level is considered the primary cause of coastal flooding and damage.

In fringing reef areas, resonance driven by wave groups, or combination of wave surges, on top of the reef flat can lead to short term surges or bores which overtop the shoreline for a minute or two. These bores are difficult to deal with theoretically and basic assumptions on theoretical limits have been used and compared to historical records from Cyclone Sally and other storms to establish coastal protection criteria.

Reef top breakwaters have been considered in addition to conventional shore revetments. These offer advantages in disrupting bore formation processes as well as reducing the design requirements for shore revetments.

The findings of the study with regard to the coastal protection requirements include:

» The area between Avarua and Avatiu is already protected with a seawall that will meet these design requirements with the addition of reef top breakwaters at RL +2.5, thus provision of reef top breakwaters is the preferred option for this area rather than reconstructing the seawall to a higher standard.

- The area west of Avatiu is not currently protected by a seawall and the cheapest solution may be to construct a new seawall without reef top breakwaters. The seawall crown level would have to be higher than existing walls (blocking a view of the ocean from ground level) and this would have to be taken into consideration from a social amenity perspective.
- The area east of Avarua has an existing seawall, however it is doubtful that it is of sufficient standard even with reef top breakwaters at RL + 2.5. This area is also under consideration for a road realignment project that may involve some reclamation. Reef top breakwaters at RL + 2.5 offer the most cost effective solution for new construction. Floodgates would also be recommended for the streams entering these harbours to reduce upstream flooding by seawater. These floodgates would need to be designed to provide outgoing flow capacity equivalent to that of the existing culvert structures to avoid detrimental effects or flooding due to rainfall.
- At the actual harbour areas in both Avatiu and Avarua, logistical constraints preclude construction of rock revetments and concrete barrier walls should be constructed behind the port / harbour area. These would be designed to block the majority of the flow from overtopping waves and bores to limit overtopping discharge to a level that can be accommodated by the drainage systems landward of the walls. L-section reinforced concrete retaining walls with toes facing seawards are probably the most economical configuration for these barrier walls and may be incorporated into traffic and port security barriers.

1. Introduction

This study is focused on coastal protection options for the Avarua – Avatiu area on the northern coastline of Rarotonga.

Several previous studies of this location have been carried out but their recommendations have not been fully implemented, probably due to cost considerations. This may be an advantage as the understanding of the critical differences between coastal processes on reef fringed coastlines and more conventional beach profiles, that are common in highly developed areas, was not well advanced at the time these studies were carried out and some recommendations were inappropriate.

The state of understanding of critical coastal processes in coral reef areas, in particular the understanding of wave setup and wave induced flows, is still at a basic level. Gourlay (1997) has derived basic equations for reef setup and Nakaza et al (1990) has highlighted the susceptibility of the system to resonance effects due to shorter period infragravity waves caused by wave groups (surf beat). Jones (2001, 2003) has highlighted observations of solitary wave interactions on reef flats and general issues.

At Rarotonga, the reef flat occurs between low and high water and the relatively small tidal range results in ocean levels near the critical reef flat level several times a day. Consequently, wave setup is primarily dependant on wave climate during a cyclonic event and because the peak cyclone waves are spread over several hours, it is highly probable that close to the maximum possible reef setup levels and associated wave effects will occur with relatively low sensitivity to ocean tidal levels.

As the magnitude of wave setup effects are closely linked to the wave heights, effects due to swell penetration from distant areas north of Rarotonga that have been noted in calmer periods are ignored as these will be significantly lower in magnitude than the cyclone wave effects. This is not necessarily the case for the other portions of the island as there is a reasonably high probability of cyclone wave conditions combining with large waves generated from the southern ocean.

Another implication of the basic state of theoretical understanding of reef flat setup and hydraulics is that any proposed coastal protection scheme should be verified by 3-D physical model testing.

A key outcome of this study is the provision of defendable design criteria in terms of reef flat water levels and wave heights. These are derived from a Monte Carlo modelling process for cyclones in the area and translation of the effects of these cyclones on the Avarua/Avatiu coastline. This process combines the variabilities inherent in cyclone effects including:

- Intensity of the cyclone
- Direction and travel speed of the cyclone
- Spatial extent of the extreme winds and waves
- Closest approach distance and relative location of cyclone path

Thus design conditions can be assigned a statistical return period or Average Recurrence Interval (ARI).

Application of this methodology allows identification of the relative effects of cyclones having the same intensity but following different paths relative to the island. In this way, a cyclone such as *Sally* delivering a peak wind speed equivalent to only a 60 year ARI condition, can cause wave and storm tide conditions equivalent to a much more extreme event (ARI of several hundreds of years) as a result of a direct hit on the island and peak waves coinciding with water levels that produce maximum setup (near reef flat level). Alternatively, similar effects could be experienced if a much more intense cyclone passed slightly offshore from the island.

Basic concepts and issues of coastal protection in coral reef environments will also be outlined and examined.

There is a high probability that implementation of any comprehensive coastal protection strategy for the area will be beyond the means of the Cook Islands Government and it will have to compete with Tsunami restoration programs for alternate funding from other sources. It is therefore proposed to give a basic understanding of the key coastal protection issues so that as general works (e.g., road upgrades) are considered in the area they will be directed towards reducing and containing impact from cyclone induced flooding and wave attack, rather than exacerbating it in different areas.

Thus as funding permits, parts of the scheme can be implemented for a gradual reduction in damage risk.

Social issues have been considered when assessing protection works. An example of this is the favouring of solutions that do not totally impede a view of the horizon or have significant adverse effects on traditional fishing and collecting practices on the reef. Concerns relating to potential exacerbation of flooding due to rainfall will have to be addressed by appropriate design of floodgates, or similar.

2. Cyclone Design Condition Methodology Overview

2.1 Overall Philosophy

The Rarotonga site is an open ocean location in deep water within a tropical cyclone environment. The meteorological and oceanographic features can be summarised as:

- Low tidal environment
- Exposed wind environment
- Exposed wave environment

It can therefore be expected that the principal coastal protection threat will be due to the combined effects of (i) the pressure deficit component of a storm surge acting coincidently with (ii) a high tidal level and (iii) high wave setup, caused by wave breaking on the fringing reef and (iv) localised wave runup on beaches (refer Figure 2.1).

Wave setup is likely to be the dominant water level controller, with pressure deficit (or inverted barometer effect - IBE) being a secondary component. The maximum potential inverted barometer effect would be of the order of 1 m and would only be realised during a very close approach of a very intense tropical cyclone (e.g. 900 hPa). It is expected that wind-stress induced storm surge setup will be generally negligible because of the surrounding deep ocean environment and that wave setup plus tide is therefore expected to largely control the statistics of the nearshore ocean water levels and the exposure of the foreshore to damaging wave events.

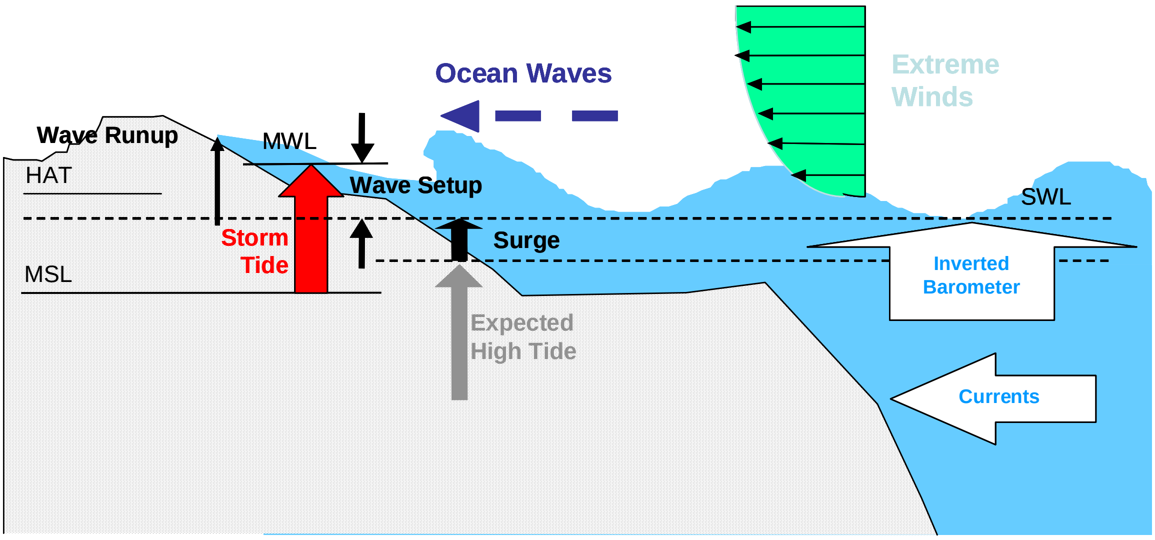


Figure 2.1. Factors influencing extreme water levels at Rarotonga.

Because of the potentially complex interactions of storm surge, tide and breaking wave setup, a statistical simulation methodology has been adopted. This firstly comprises a statistical analysis of the storm climatology to provide a complete range of potential storm parameters. These have then been used to control a Monte Carlo simulation which invokes deterministic (parametric) models of each phenomena (surge, waves, setup) and assembles a synthetic time history of extreme water levels from which return periods of water levels may then be derived. Other factors such as localised wave runup have been considered in conjunction with the elevated ocean levels when assessing the foreshore protection options.

This technique allows a fully objective assessment which accounts for the joint probability of all the necessary parameters. A complete return period estimation of water levels is obtained to any desired average recurrence interval (ARI) which does not directly rely on data fitting assumptions. The simulation can be easily altered to test the sensitivity to the controlling assumptions taken from the storm climatology and to any other assumptions such as reef wave setup response. This technique however, relies on establishing parametric models of the various factors.

2.2 Detailed Methodology

Details of the various numerical modelling systems used are provided in Appendices, namely:

1. Tropical cyclone wind and pressure model (Appendix B);
2. 2D spectral wave model ADFA1 (Appendix C);
3. SATSIM parametric storm tide model (Appendix D);
4. Analytical breaking wave reef setup (Appendix E).

2.3 Assessment of Recorded Data

Principal recorded data sources for the study have included Australian Bureau of Meteorology National Tidal Centre (NTC) SEAFRAME hourly water level, wind and atmospheric pressure data from 1993 to 2003 and long-term Cook Islands Meteorological Service (CIMS) wind and pressure records for the airport site on Rarotonga, provided via NIWA in New Zealand. Tidal data has also been sourced from the University of Hawaii Sea Level Center.

2.3.1 Deterministic Model Checks

A series of deterministic model checks were then undertaken to:

a) examine the characteristics of water level response from selected tropical cyclones of record, and

b) prove the accuracy of parametric models of surge, wave height and setup to be applied during the simulation phase

This involved:

» Establishing nested numerical model domains for the 2D ADFA1 spectral wave model

» Assembling historical track details for selected storms (e.g. *Sally* Jan 1987, *Gene* Mar 1992, *Pam* Dec 1997)

» Running the models with applicable tidal levels to estimate the various water level components during those events

» Comparing the model results with any available measured data (wind, pressure, wave height and period, water level)

» Comparing the numerical model performance with the embedded parametric models to be used in the SATSIM simulations

2.3.2 Simulation Production Modelling

The SATSIM simulation model (BPA 1985, Harper *et al.* 1989, Harper 1999, Harper *et al.* 2001) was initialised with the statistical tropical cyclone parameters for the region and the local tidal constituents provided by the NTC. To this was added the fringing reef parameterisation required for the Gourlay (1997) methodology for reef wave setup at the selected study locations.

SATSIM embodies a full analytical model of tropical cyclone wind and pressure fields (Harper and Holland 1999) which permits direct calculation of the local hydrostatic pressure deficit (the nominal total storm surge in a deepwater open ocean environment). The model has been successfully validated against long-term wind records around Australia and at Cocos Keeling Island in the Indian Ocean. The parametric wave model is derived from a multi-dimensional interpolation of many hundreds of detailed ADFA1 model simulations for an open ocean site. The wave setup method of Gourlay (1997) is added to the parametric wave model allowing wave setup estimates to be calculated at any wave-exposed site.

The embedded parametric wind, surge, wave and setup models in SATSIM then generate synthetic time-aligned histories of the following parameters:

- Mean and gust wind speed
- MSL atmospheric pressure
- Storm surge pressure deficit
- Open coast wave height and period and wave setup

for many thousands of regional storm scenarios. The statistics of exceedance of the water level components are then accumulated for direct analysis of the return periods of interest. The model also automatically generates joint probability statistics of storm parameters such as intensity, proximity, speed etc. The period of model simulation can be varied and is typically 10,000 years, thus providing 10 estimates of the 1,000 year return period event. The highest simulated level also provides an estimate of the probable maximum water level under present climate.

3. Regional Tropical Cyclone Climatology

The Fiji Meteorological Service (Nadi) has WMO/RSMC operational jurisdiction for issuing tropical cyclone warnings in the Cook Island region. However, the New Zealand Meteorological Service (NZMS) has developed and maintained a “best track” database of tropical cyclones for the region on behalf of the many South Pacific nations. The latest dataset (current to 2003/2004 season) was kindly provided for the present analysis (Ready 2004). Long-term wind data at Rarotonga was provided through the New Zealand National Institute of Water and Atmospheric Research (NIWA) by arrangement with the Cook Islands Meteorological Service (CIMS).

3.1 Statistical Assessment of the Tropical Cyclone Hazard

The supplied regional tropical cyclone track data is limited to the period 1969/70 through to 2003/2004. The 1969 start marks the routine availability of satellite imagery, which largely ensured that all potential cyclones were at least detected and could be tracked. This period coincides also with the use of objective intensity estimation methods based on satellite imagery (e.g. Dvorak 1975). A total of 33 seasons of track data was therefore available for analysis and a number of decadal climatology summary reports were also provided by NZMS (Kerr 1976, Revell 1981, Thompson *et al.* 1992).

For the purpose of the analysis, a statistical "control volume" is selected, taken as a nominal 500 km radius of Rarotonga. This radius ensures that all cyclones which could have influenced the island within a travel period of approximately 24 h are considered in the analysis. Clearly, if too small a radius is selected, the resulting storm sample size is also very small and the estimated point statistical properties are much less certain. If too large a radius is taken then the climatology may not be spatially stationary. The 500 km radius has been found to be adequate for these purposes in a number of previous analyses.

On the basis of considering data only since 1969 and within 500 km of Rarotonga, Figure 3.1 presents a summary of the annual frequency of occurrence in histogram format. The total number of storms during this period was 43, with the greatest number in any season being five each in 1981/82 and 1986/87. The 5 year average frequency of occurrence is also shown in order to smooth the annual variability and shows an annual average of about 1.3 storms. Also shown on Figure 3.1 is the annual and five year averaged Southern Oscillation Index (SOI), which is known to be an indicator of monsoonal intensity in Northern Australia and India. Whilst not overly compelling in regard to the occurrence of tropical cyclones within 500 km of Rarotonga, there is a hint of increased occurrence during El Niño periods (-ve SOI).

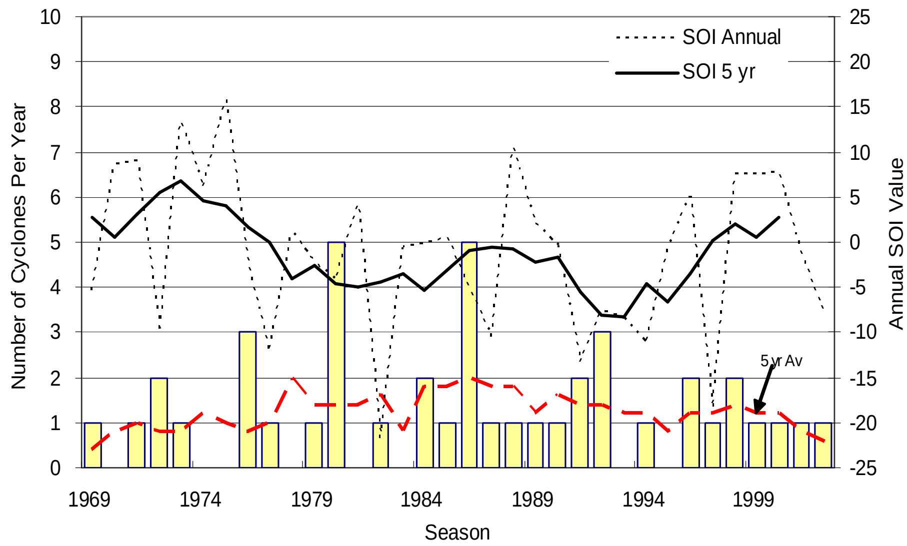


Figure 3.1 Frequency of occurrence of tropical cyclones within 500 km of Rarotonga.

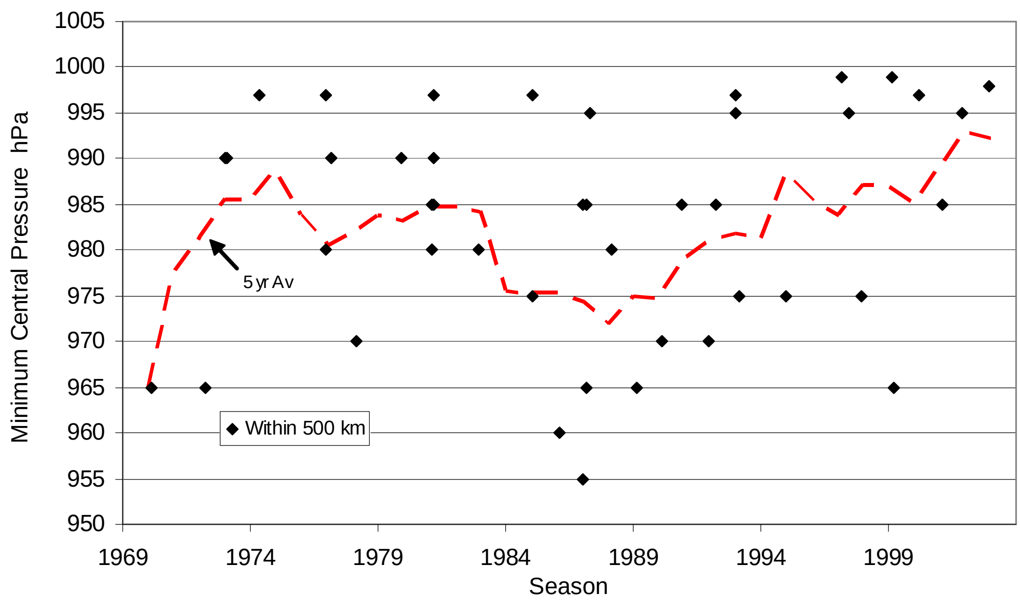


Figure 3.2 Intensity of tropical cyclones within 500 km of Rarotonga.

Figure 3.2 presents the corresponding time history of maximum intensity estimates (central pressures) of tropical cyclones within the adopted 500 km radius. This shows a high inter-annual variability but with a possible trend towards decreasing estimated intensities in recent years. The most intense period was in the late 1980s, at a time of near-neutral ENSO.

Figure 3.3 presents the seasonal distribution by month, showing February has the highest incidence at 30% of the total, followed by January and March at 20% and a further smaller peak of 16% in December. A single storm in June is also indicated, which appears potentially anomalous but has been retained. The duration within radius distribution is given in Figure 3.4, showing a mode of 24 h but a mean of only 18 h. This suggests relatively fast forward speeds or that the location is just outside of the principal region of activity. Almost 90% of all storms are resident within 500 km for less than 2 days. Figure 3.5 indicates how close these storms have come to Rarotonga in the past 33 seasons as a histogram of the closest approach distances. In this case the straightness of the cumulative distribution line for the distance of closest approach shows this to be a reasonably even distribution of tracks, at least within about 300 km, which is what could be expected in the open ocean environment free from the influences of continental landmasses. Also shown is the distance distribution when the storms were at their maximum intensity within the radius. This shows a bias towards larger distances; again suggesting the region is just away from a more intense area.

Figure 3.6 presents the histogram of forward speeds of the storms at their point of closest approach. This shows an average speed of about 5.5 ms^{-1} and a modal value of 6 ms^{-1}. This is reasonably high and there is a further long tail out to 18 ms^{-1}, supporting the rather short durations seen earlier. Some of these very fast storms would be expected to be south of the site, tending extra-tropical and weakening. The next figure (Figure 3.7) shows the corresponding distribution of track bearings at time of closest approach, which highlights the strong preference for storms to originate in the NW and track SE (135˚). Interestingly, the modal track direction of 150˚ is almost N-S. Besides these dominant features, there is also a small group of storms that track from NE/ENE towards the west.

Figure 3.8 presents the actual tracks of all 43 storms during the 33 year period of data. Although reasonably chaotic, the preference for a SE movement is clear. Rarotonga is located by the circle near the centre of this figure, which covers the 500 km radius statistical control volume.

Appendix A provides a summary of all tropical cyclones within the 500 km radius based on the NZMS track dataset. This base information needs to be further enhanced for modelling purposes, as discussed in the next section.

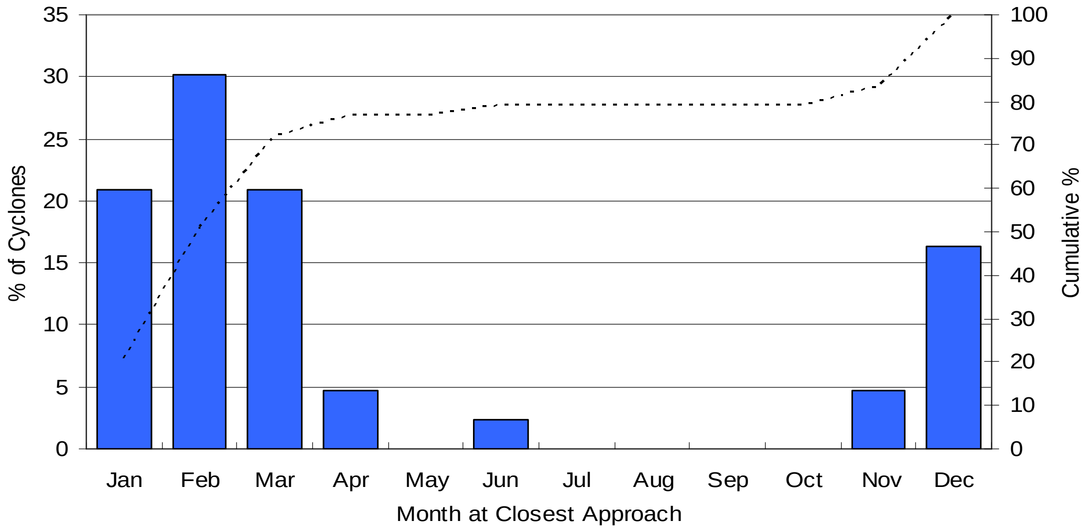


Figure 3.3 Seasonal distribution of cyclone occurrence

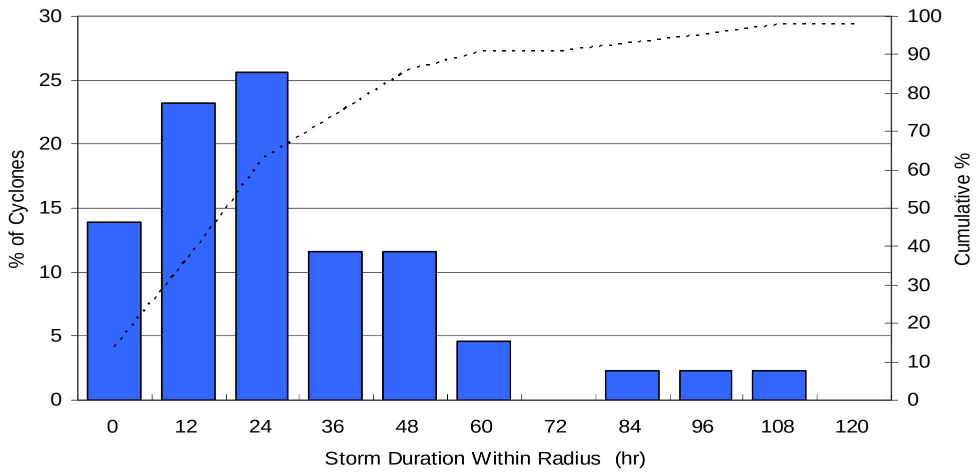


Figure 3.4 Duration distribution within radius.

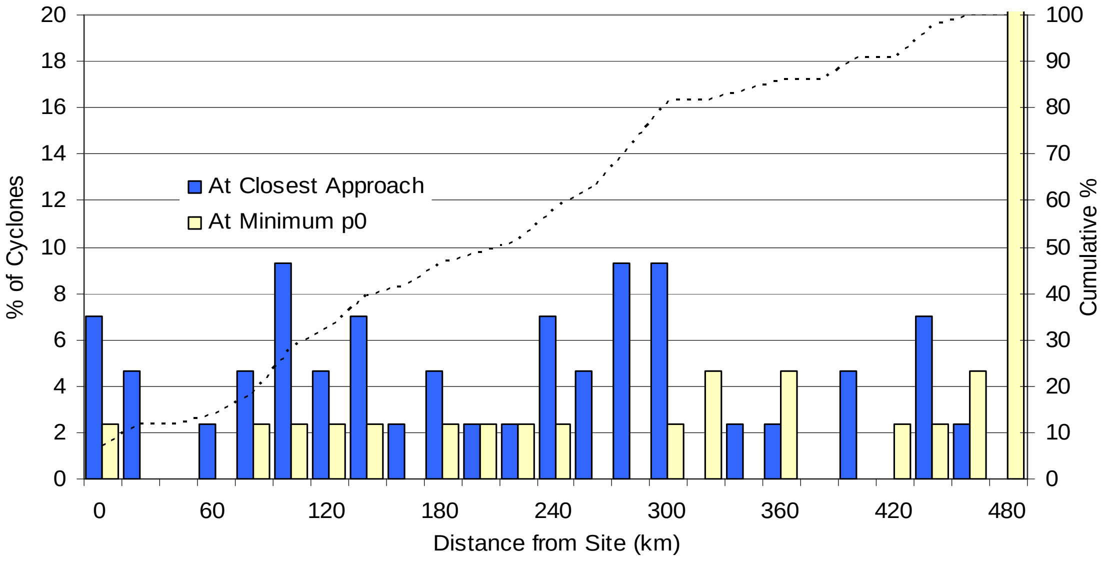


Figure 3.5 Distance of approach distribution.

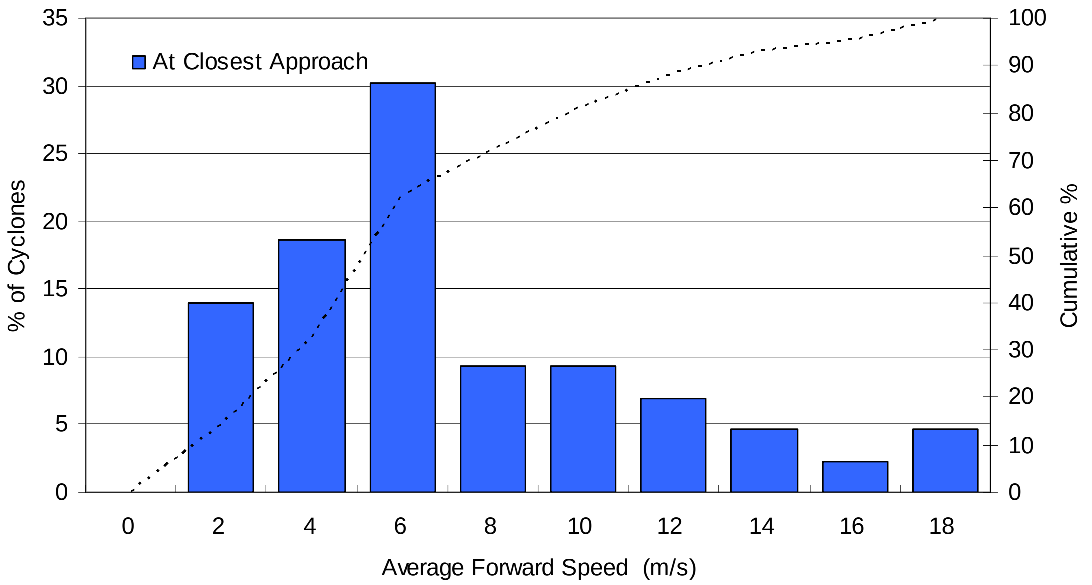


Figure 3.6 Forward speed distribution.

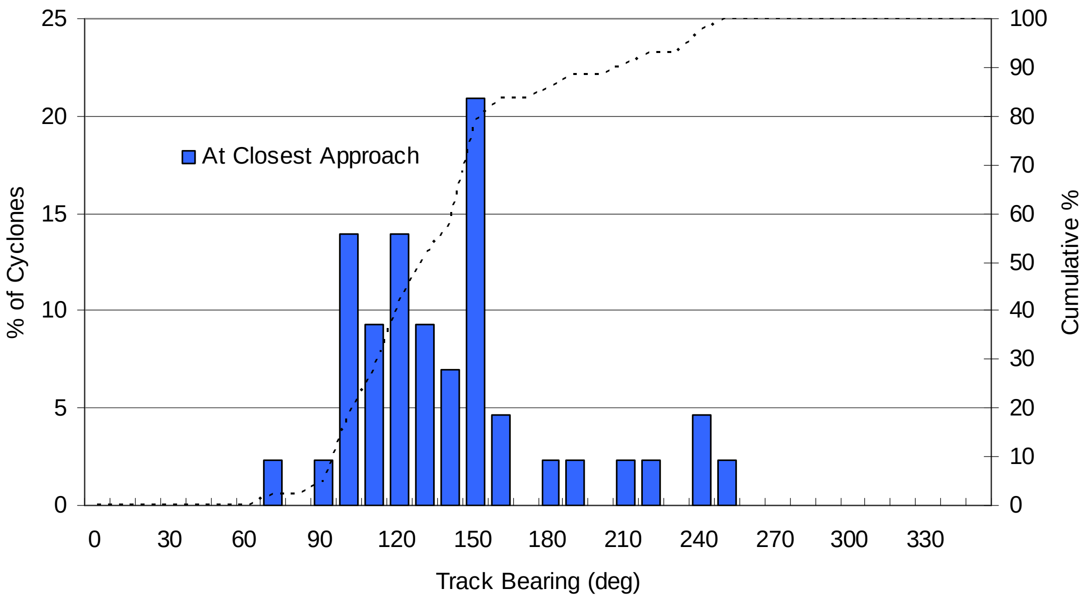


Figure 3.7 Track bearing distribution

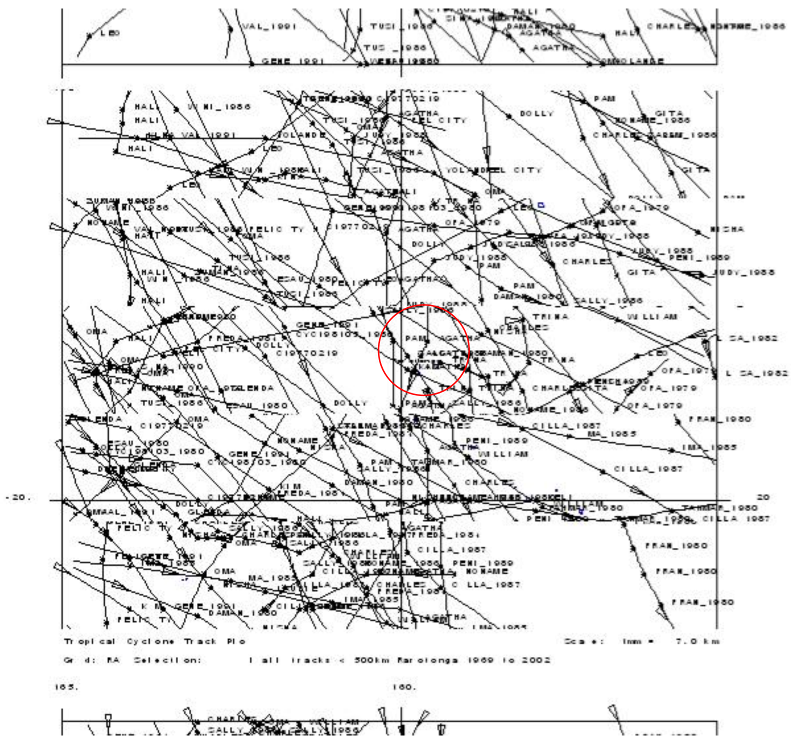
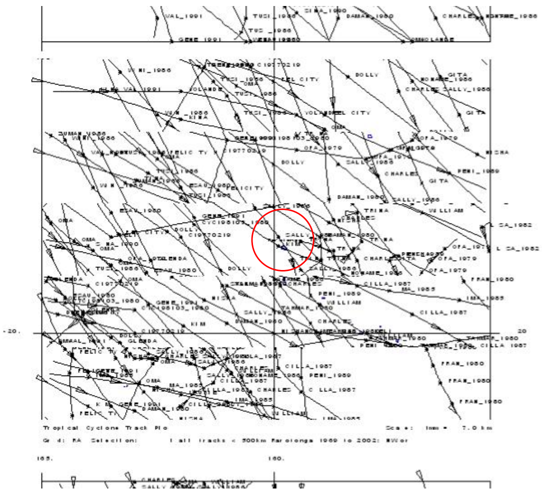


Figure 3.8 Combined tropical cyclone tracks from 1969/70 to 2003/2004.

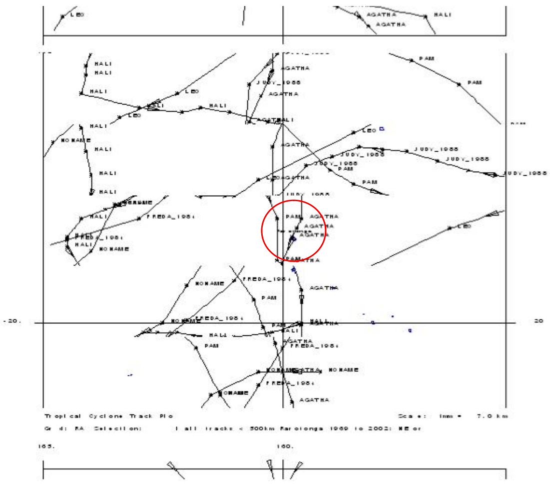
3.2 Parameterisation for Modelling Purposes

The adopted climatology model has the ability to describe the regional behaviour of tropical cyclones in terms of a mixture of different storm "populations". This allows for potential differences in the intensity, track and speed of tropical cyclones from different sources or under different broadscale climate influences. The initial climatology analysis points to the fact that there are essentially two storm population sources in the region - the North-West source with storms moving typically SE and the Easterly source typically moving W or WSW. The data set was then stratified so as to allocate each storm in the dataset to one of these two broad populations and the statistics were re-worked to determine if any significant differences could be found.

Firstly, Figure 3.9 presents the separated track plots for each case. Many individual storms have erratic paths and may form loops with extensive north-south or even east-west excursions but generally each of the storms can reasonably be allocated to one of these track origins. What appears immediately obvious from this separation is that the density of tracks is greatest for the north-east origin class.



North-West track origin class.



North-East track origin class.

Figure 3.9 Tropical cyclone tracks separated by origin class.

Following separation of the tracks, extreme value statistical analysis was carried out on the estimated central pressure data. Figure 3.10 presents these results in terms of the best fit line for each origin class (NW and NE), the combined class line and the combined dataset. These curves were obtained following the method of Petrauskas and Aagaard (1971) and are specified as Extreme Value Type I *Gumbel* curves (Benjamin and Cornell 1970) for input into the SATSIM model, the parameters for which are summarised in Table 3.1. Overall, the analysis shows that the tropical cyclone intensity within 500 km is relatively low when compared with other more westerly regions of the South West Pacific. The analysis suggests that the NW origin class is the more intense storm population on a return period basis. However, the estimated intensities assigned to the very small number of storms from the NE origin suggest that these may be more severe on a storm-by-storm basis. The net result is that the two classes have a near-parallel statistical behaviour. In each case the continuous distributions are truncated in the modelling process at a nominal *Maximum Potential Intensity* (MPI) following the work of Holland (1997), which considers the theoretical thermodynamic limits as a function of regional climate and ocean indices. No specific assessment of the MPI for this region is available but a value consistent with the North West Shelf region of Western Australia has been adopted and is considered reasonable. This limits the maximum possible intensity in the region to 880 hPa which, based on Figure 3.10, has an estimated return period of well beyond 1000 y in this region.[1]

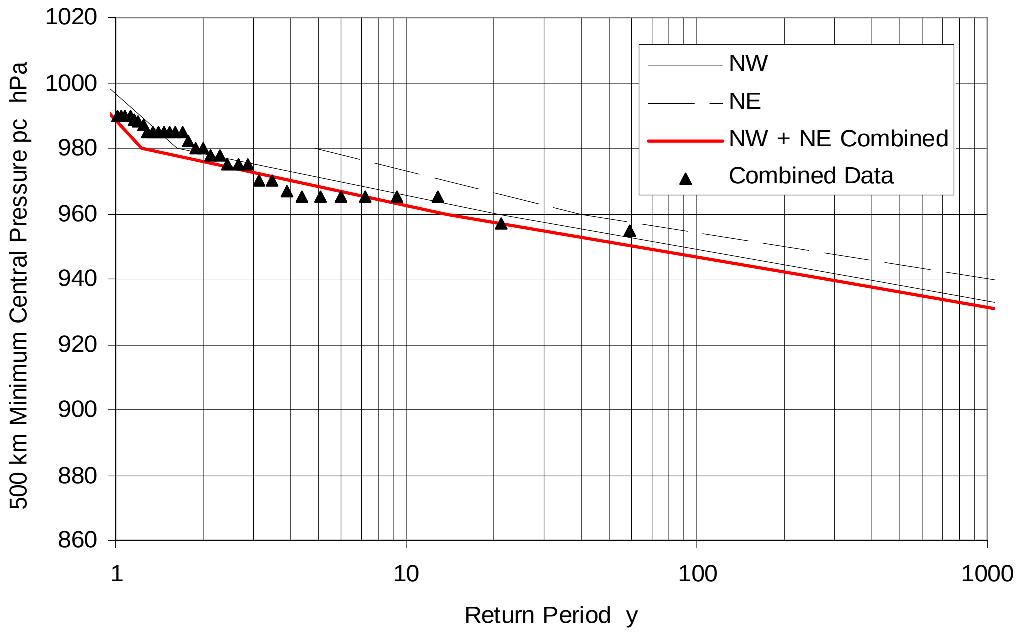


Figure 3.10 Extreme value analysis of central pressure estimates.

It remains to specify a number of other parameters to describe the regional tropical cyclone climate for each of the separated origin classes, i.e.

- distance of closest approach to target X_{prox}
- track bearing variability θ_{fm}
- forward speed variability V_{fm}
- relative filling of storm pressure

These are presented as histograms similar to the combined dataset shown earlier and are provided to the model as a series of smoothed data cumulative distribution functions (CDFs) as shown in Figure 3.11. Also, the best track dataset does not provide information on the variability of the important storm spatial parameters:

- radius to maximum winds parameter R_c
- Holland wind field peakedness B_0

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[1] Sensitivity analyses have shown a high spatial variability in storm intensity in this region and the model parameters selected here based on the 500 km radius may be adjusted after initial simulations if the long term regional wind record at Rarotonga is not well represented.

Distributions for the above parameters have therefore been estimated based on experience in fitting the wind field model to various Australian storms.

Table 3.1 Principal objective statistical parameters determined for modelling.

Track Population	Model Parameters < 500 km			Rarotonga Region
	Name	**Variable**	**Units**	
	Ambient Pressure	p_n	hPa	1011
NW Origin	% This Track			83.7
	Av. No. Per Year			1.09
	Gumbel Intensity	U	hPa	981.292
	Parameters	α		0.1458
	Max Potential Intensity	MPI	hPa	880
NE Origin	% This Track			16.3
	Av. No. Per Year			0.21
	Gumbel Intensity	U	hPa	972.608
	Parameters	α		0.1648
	Max Potential Intensity	MPI	hPa	880

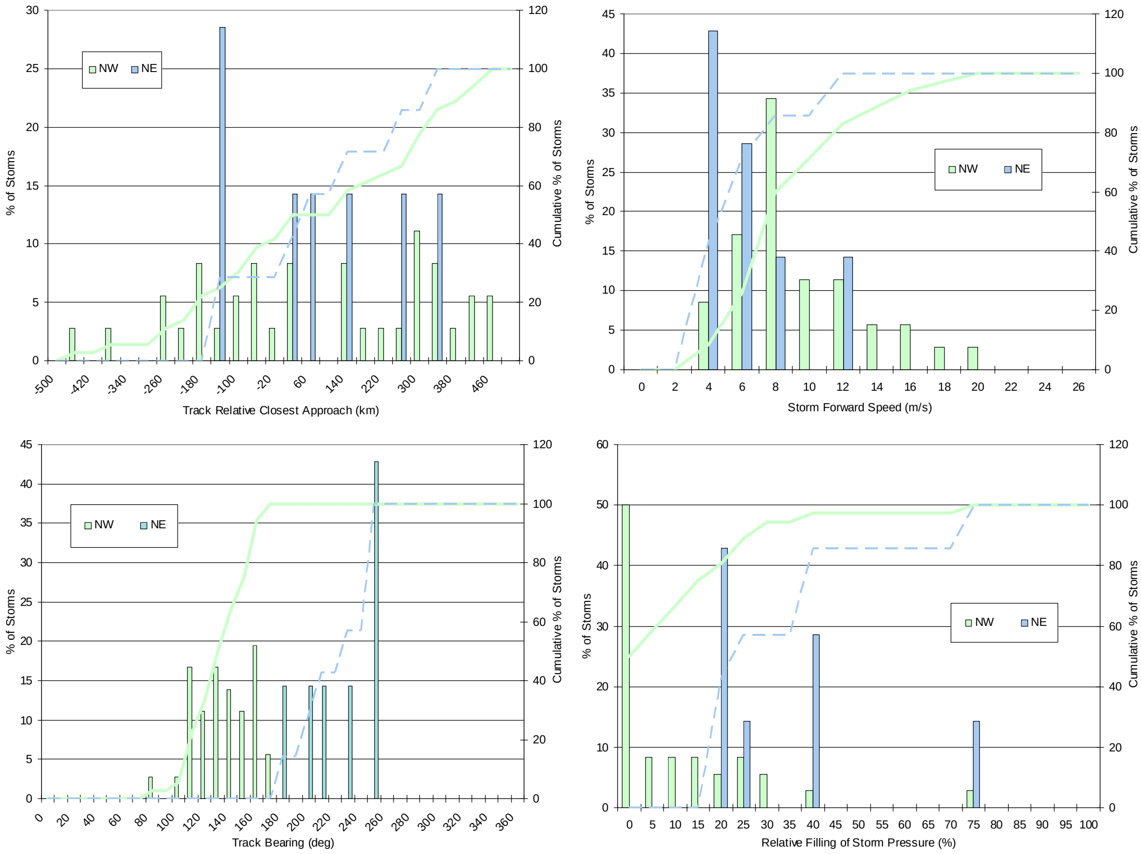


Figure 3.11 Separated storm parameter distributions provided to the model.

3.3 Regional Wind Speed and Pressure Data

All available wind data was obtained to estimate the long term wind speed return period relationship for tropical cyclones and to provide calibration data for a selection of historical cyclones. The Cook Islands Meteorological Service (CIMS) operates a WMO standard weather station at the island airport, which is situated near the NW tip of Rarotonga on flat ground 7 m above MSL. The station commenced operations in 1957, but data in electronic format is only available from late 1970 onwards. An additional AWS station was established nearby in 1999 by the NZ National Radiation Laboratory.

An additional 10 years of wind and pressure data was made available from the Australian National Tidal Centre (NTC), which operates a *Seaframe* precision tide recording station in Avatiu Harbour. The sensor is at standard +10m height and the tower base is at 2.5 m MSL but is backed by harbourside buildings and typically fronted by moored vessels.

Table 3.2 summarises the data sources and associated parameters of interest, where V_m is the 10 minute mean wind, θ_m the associated direction, V_3 is the 3-second peak gust speed and p is the MSL pressure. The data sets were provided through the New Zealand National Institute of Water and Atmospheric Research (NIWA) by arrangement with CIMS.

Table 3.2 Rarotonga long term wind records.

Station	Name	Type	Data	Start	End	Years
J84300	AERO	3 Hourly	V_m θ_m p	01-Dec-1970	12-Sep-2004	33.8
J84300	AERO	Daily	V_3	01-Dec-1970	12-Sep-2004	33.8
J84300	EWS	3 Hourly	V_m θ_m p	29-Sep-1999	12-Sep-2004	5.0
J84300	EWS	Daily	V_3	29-Sep-1999	12-Sep-2004	5.0
Seaframe	Avatiu	Hourly	V_m V_3 θ_m p	01-Feb-1993	31-Dec-2003	10.9

The 3 hourly and daily wind data sets from J84300 were analysed to check the consistency of the data and provide some indication of regional variability. During this process some inconsistency between the J84300 mean winds and the J84317 mean winds were discovered from 2001 onwards, whereby the J84300 reported winds are clearly anomalously high in an intermittent manner. This was also supported by comparison with the Seaframe dataset. Nevertheless, Figure 3.12 presents the resulting mean wind speed and direction distribution from the 33.8 years of J84300 data in histogram format, consisting of data for all months overlaid with the January data for comparison. The shape of the speed distribution suggests a bi-modal quality possibly related to diurnal effects at low wind speeds and shows the dominance of the Easterly trade winds throughout the year. Figure 3.14 presents the corresponding speed distribution for wind gusts, where the modal value is shifted and the distribution has an extended tail. There is also evidence of a "gales" artefact near 17 ms^{-1}. Atmospheric pressures were also examined and the mean summer MSL pressure of 1011 hPa was selected as the regional ambient pressure for tropical cyclone modelling purposes.

The 3 hourly mean and daily peak gust data sets for J84300 were then merged to form a composite set for extreme value analysis. In this case only the peak wind recorded during the period of activity of each recorded tropical cyclone was retained for analysis. This also prevented contamination from the previously mentioned anomalous data. For example, the peak winds of record in the electronic datasets provided are during cyclone *Sally* in January 1987; being V_m of 27.8 ms^{-1} and V_3 of 42.8 ms^{-1}. In this case the statistical analysis has been based on the method of maximum likelihood (Benjamin and Cornell 1970) again using Extreme Value Type I (Gumbel) guidance. The results are presented in Figure 3.14 for both the mean and gust winds, with predicted 100 year return period values (the solid lines) of approximately 31.2 ms^{-1} and 52.9 ms^{-1} respectively. The figure also indicates the names of the highest plotted events. These results are later compared with the SATSIM model predictions in Section 4.6.2. It should be noted that this analysis considers only winds generated by tropical cyclones and not winds from any other sources, such as squalls and the like.

One further consideration for analyses of this type is the representativeness of the available recorded winds relative to the nearby "open ocean" conditions that will be predicted by the numerical models. Due to the mountainous nature of the island it is likely that there is some shielding of the anemometer for winds in the ESE to SSW sector. Furthermore, the north facing coastal plain where the airport is situated may also experience some "funnelling" for winds close to E or W. However, without a site specific calibration it is not possible to propose an objective transformation and the data is therefore accepted as representative for the present purposes.

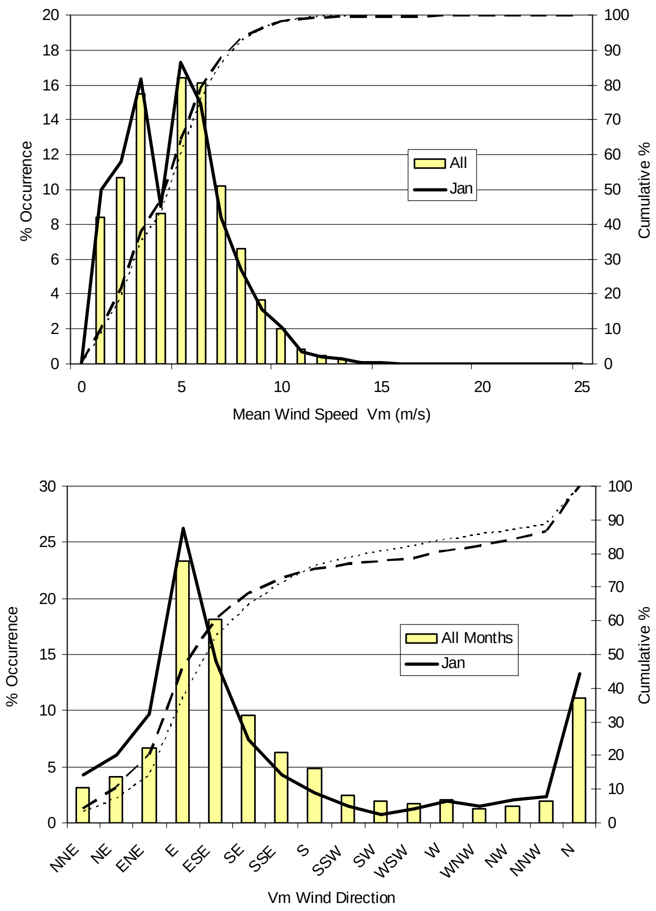


Figure 3.12 Mean wind speed and direction distributions.

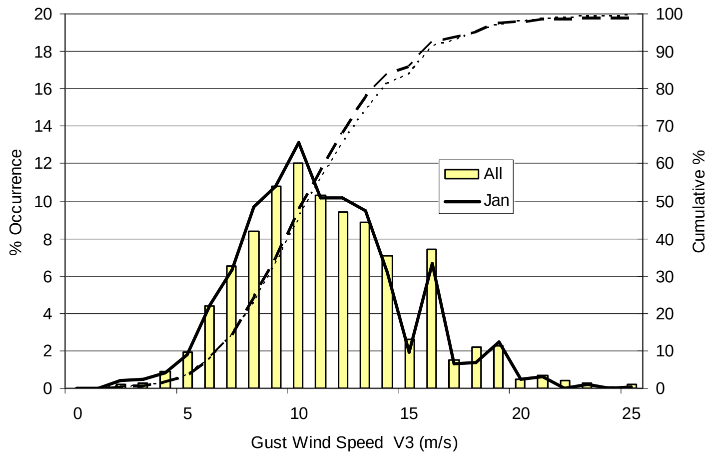
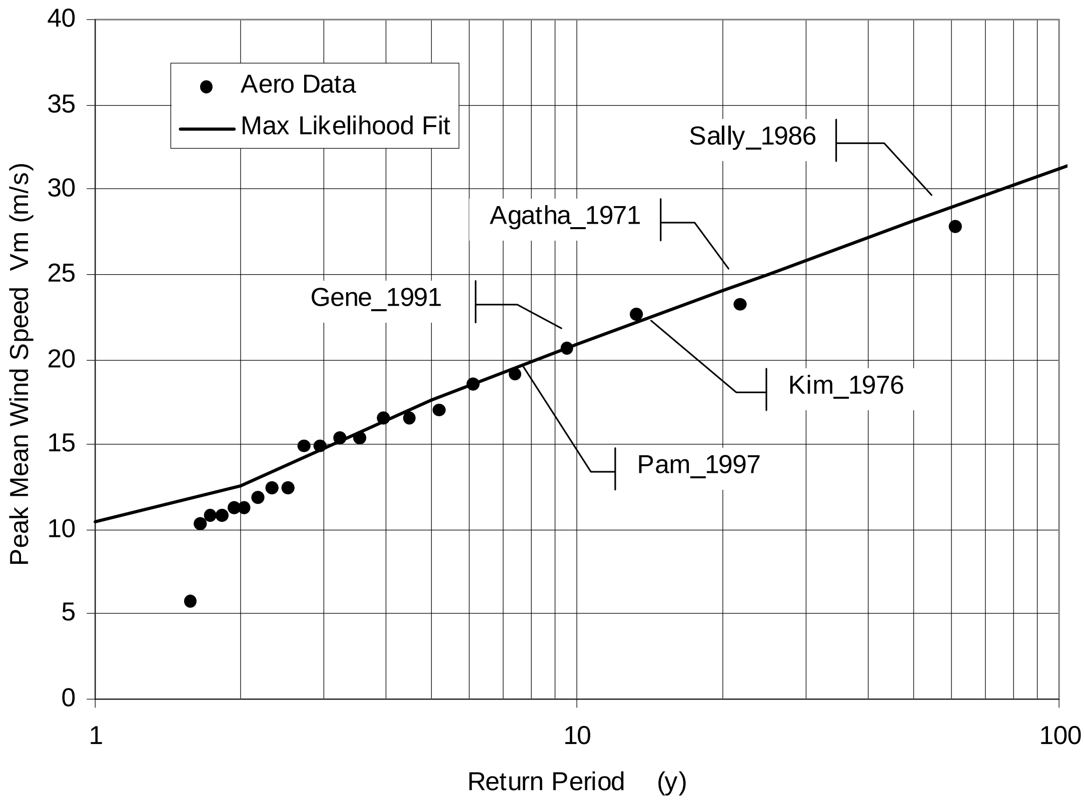


Figure 3.13 Peak daily wind gust speed distribution.



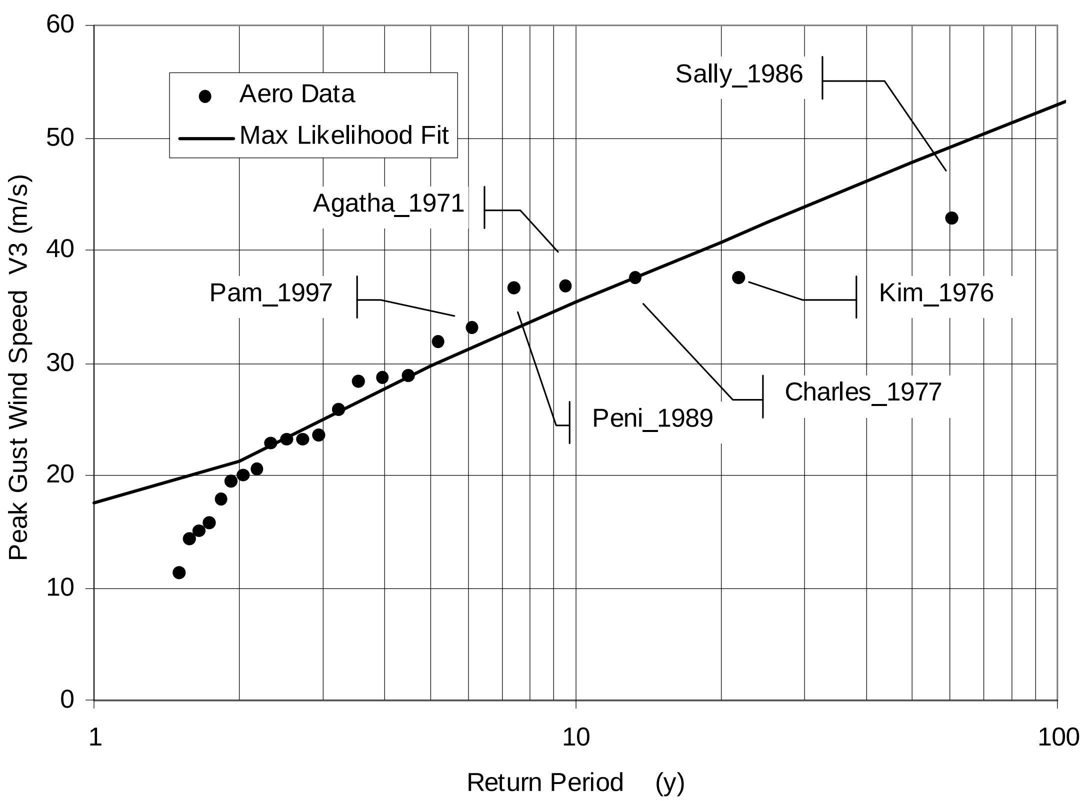


Figure 3.14 Extreme value analysis of wind speed during tropical cyclones.

3.4 Selection of Hindcast Storms

As part of the overall model validation process, a "top 5" storm set has been assembled for consideration. This is based on those storms which generated the five highest mean wind speeds at J84300. All of these storms passed within 150 km of Rarotonga with central pressure below 990 hPa. Table 3.3 summarises their parameters at closest approach while Figure 3.15 shows their combined tracks. Wind speed and pressure data for each event was extracted from the data record for use in Section 4.2.1.

It can be seen that for all of these storm tracks, it is likely that the airport anemometer experienced the peak wind conditions and was unaffected, at least directly, by terrain effects.

Table 3.3 "Top 5" hindcast storms.

		Aero Peak Data				At Closest Approach				
No.	**Name**	**V_m**	**V_3**	**p**	**Est p_c**	**Date**	**V_{fm}**	**θ_{fm}**	**Dist**	**Bear**
		ms^{-1}	ms^{-1}	hPa	hPa	UTC	ms^{-1}	°	km	°
198604	**SALLY_1986**	27.8	42.8	981.8	967	02-Jan-87	4.3	146	2	23
197110	**AGATHA_1971**	23.2	36.9	997.4	980	24-Mar-72	3.0	198	1	267
197601	**KIM_1976**	22.6	37.5	992.7	980	12-Dec-76	12.8	113	5	269
199110	**GENE_1991**	20.6	31.9	989.6	985	17-Mar-92	7.9	155	151	245
199705	**PAM_1997**	19.1	33.1	985.5	980	09-Dec-97	4.6	180	32	274

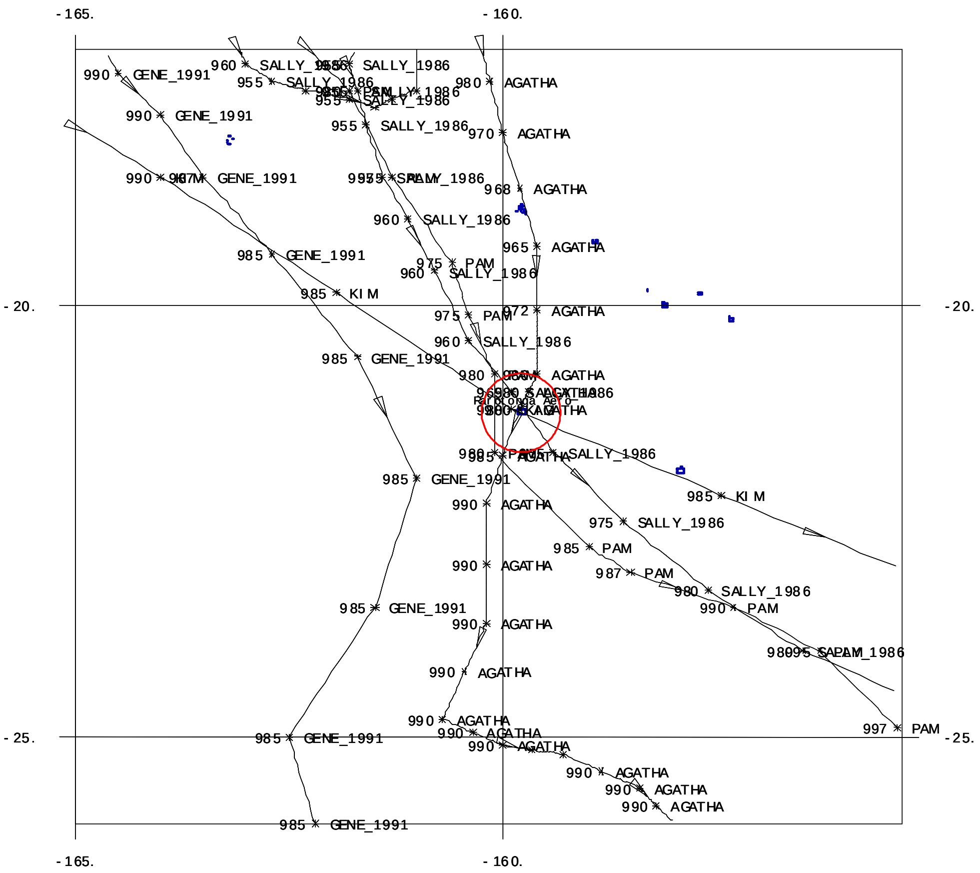


Figure 3.15 Tracks of the "top 5" storms.

4. Numerical Modelling

This section discusses all aspects of the numerical modelling of winds, waves and wave setup leading to the validation of the various models and finally to the prediction of design water level and wave conditions.

4.1 Representation of Localities

The focus of the investigation is the coastal section between Avarua harbour and Avatiu Harbour on the northern side of Rarotonga.

Avarua Harbour is a reef opening backed by low lying land and two streams. Local increases in water level due to reef setup on adjoining reefs or overtopping of the harbour wall by wave action will result in local damage from flow and flooding landward into low lying areas.

Avatiu Harbour is the main commercial port and also includes a marina for local and visiting yachts. The land elevation behind the harbour is generally higher than that at Avarua and the stream is more sheltered.

Both locations are located on the northern shore of the island and are sheltered from most large area ocean wave generation effects which typically originate to the south of the island.

Tropical cyclone effects are the major source of storm wave activity on this northern coast. Swell penetration from the northern hemisphere that has been noted in calmer conditions will be less severe than tropical cyclone effects and is not considered for extreme conditions.

Major storms in the southern ocean and or SE trade conditions will provide a low amplitude infragavity wave influence as waves wrap around the island. Generally this will be relatively insignificant but can possibly combine with wave group effects from an approaching cyclone.

At this level of investigation, the reef top wave processes are considered only in a one-dimensional sense. This approach, detailed in Appendix E, requires the specification of a number of reef profile parameters that have been derived from previous studies (JICA 1992 & 1994) and offshore survey data supplied by SOPAC. Specific information on reef top levels and land levels could not be supplied by the Cook Island Government in a suitable time frame for this study.

The adopted reef top parameters are summarised in Table 4.1. The principal difference between the various sites is that the reef flat east of Avarua is wide and has a steeper reef rim. This will be seen to produce a higher breaking wave setup condition for the same incident wave conditions than the other sites. Reef rim dimensions are set at 400m for calculation purposes. Because detailed survey data was not available and the reef widths varied, a series of sensitivity tests were carried out and the values producing the largest setup adopted for each region. The dimensions may differ from prototype dimensions at any particular point. Also, although the reef top is narrower west of Avatiu, the difference between sites 2 and 3 in terms of wave setup is generally negligible. Accordingly, the later discussion reduces to consideration of “East of Avarua” and “West of Avarua” localities.

Table 4.1 Site specific reef profile parameters.

Site	Name	Reef Crest	Reef Edge	Reef Flat	Reef Rim	Rim Slope	K_p	K_p'
		m MSL	m MSL	m	m	tan α		
1	**East of Avarua**	-0.40	-10.0	170	400	0.10	0.36	0.60
2	**Avarua - Avatiu**	-0.40	-10.0	150	400	0.04	0.25	0.49
3	**West of Avatiu**	-0.40	-10.0	75	400	0.04	0.25	0.49

4.2 Wind and Pressure Field Modelling

Each of the "top 5" historical tropical cyclones was modelled using the parametric wind and pressure model after Harper and Holland (1999). The model parameters were based on the NZMS "best track" estimates of central pressure and position over time and compared with the measured wind and pressure data at J84300 ("AERO"). While there are undoubtedly errors in the estimated central pressures, which are mostly derived using the Dvorak (1984) method, no subjective adjustment to these values has been made. The calibration process absorbs this error by estimating values for the radius to maximum winds (*R*) and the Holland peakedness factor (B) that lead to a minimisation of the error between modelled and measured values. The purpose of this exercise is to demonstrate that the wind and pressure model, which underpins the spectral and parametric models for waves, is capable of accurately representing conditions during a number of important historical tropical cyclones at Rarotonga.

4.2.1 Wind and Pressure Calibration

A brief summary of each tropical cyclone calibration follows, in the order given in Table 3.3, with reference to the comparison time history plots of winds and pressures in Figure 4.1 to Figure 4.6.

TC Sally (December 1986)

This storm is the most severe to impact Rarotonga in modern history, passing directly over the island on 2nd January, with a measured MSL pressure of 968.3 hPa and mean winds to 28 ms^{-1}, gusting to 43 ms^{-1} (Frost 1987, Kishore 1987). The wind and pressure record has been very well modelled here using an *R* of 84 km and a B_o of 7.2 (refer Figure 4.1), indicating a fairly large circulation and a relatively flat wind profile. Extensive damage was reported as a result of storm tide and wave action (refer Section 4.5), as well as wind damage to vegetation across the island but buildings faired reasonably well.

TC *Agatha* (March 1972)

This storm also passed directly over Rarotonga but was slightly weaker than *Sally*. No meteorological report was available for this storm, which might have been able to explain why the AERO barograph record showing only 997.4 hPa is inconsistent with the best track central pressure of 980 hPa at this time and also inconsistent with the measured mean winds of 23

ms^{-1}. Ignoring the barograph record, the leading edge of the wind profile was well modelled assuming an R of 32 km and a B_o of 7.2 (refer Figure 4.2). After the eye passage, the recorded winds dropped markedly, perhaps due to either instrument problems or shielding. Alternatively, the storm structure may have been affected by the island landmass.

TC *Kim* (December 1976)

This storm passed very close to Rarotonga as it moved SSE at quite a high speed (12 ms^{-1}). While the winds were able to be reasonably well modelled using an R of 26 km and a B_o of 6.6, the pressure differential is overestimated (refer Figure 4.3). This is likely due to error in the best track centre locations, which are sparse at this time. This would also account for an apparent phase error that is evident in the comparisons. No meteorological report was available.

TC *Gene* (March 1992)

This large but fairly weak storm passed 150 km west of Rarotonga on a southerly path. The winds were reasonably well modelled using an R of 150 km and a B_o of 6.8, indicating a very flat wind speed profile (refer Figure 4.4). The MSL pressure has not been particularly well matched and, with more time, a better calibration could probably be obtained. No meteorological report was available.

TC *Pam* (December 1997)

This storm passed about 30 km west of Rarotonga on a near-southerly track, causing a significant storm surge and widespread damage (Kumar 1998). It was estimated to have reached peak intensity when some 130 km NNW and was rapidly weakening as it passed the island. While a reasonable wind speed magnitude calibration was achieved with an R of 40 km and a B_o of 6.6, the pressure is not well represented and there is evidence of phase errors that are probably caused by errors in the centre position (refer Figure 4.5).

In addition to the "top 5" wind events, TC *Peni* was also selected for wind model calibration to enable hindcasting of the measured wave conditions off the south-east coast of Rarotonga (Barstow and Haug 1994).

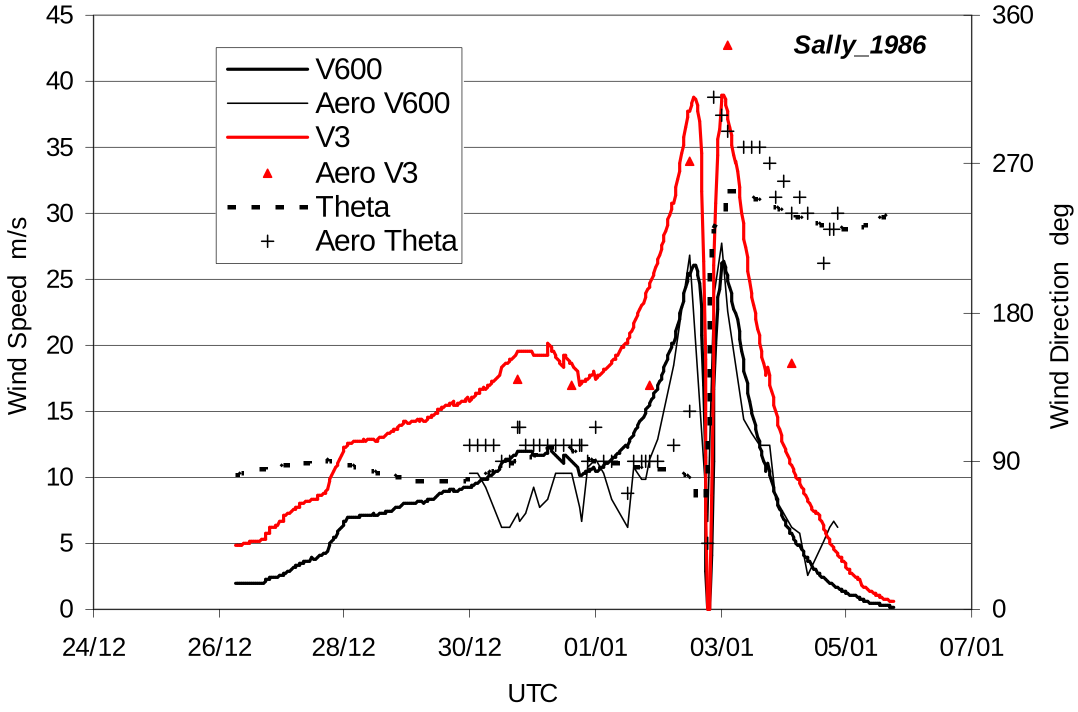
TC Peni (February 1990)

This storm passed about 200 km east of Rarotonga on 15th February, but within 50 km of Mauke, where a minimum MSL pressure of 972.2 hPa was recorded with mean winds to 33 ms^{-1} and an eye passage occurred over Atiu shortly afterwards, but with no pressure reading (Laird 1990). Based on the reported timing of the passage of maximum winds at Atiu (7 to 8 h) and the average speed of the centre at the time (7 ms^{-1}), an indicative R value is of the order of 115 km. Using this information, reasonably consistent re-creation of the winds and pressures at both Mauke and Rarotonga could be obtained, although there is some evidence of asymmetry. The resulting comparisons at Rarotonga using a B_o peakedness of 6.9 are given in Figure 4.6. TC *Peni* is the largest storm in this set and the calibration compares favourably with the broadscale wind pattern in Pandaram (1990).[2]

In summary, the wind and pressure record from each of the storms was able to be reasonably reproduced by the wind model within normally acceptable parameter ranges. Taken as a

[2] Section 4.3.2 further discusses the broader scale meteorology of this storm, which was imbedded within a very large monsoonal low.

whole, however, the data suggests a tendency for reasonably large scale systems affecting this region. This is consistent with the climatological aspects of it being generally outside of the main generation region, where storms are tending to be past full maturity and in decaying or transitioning phases. This likelihood has been accounted for in the statistical modelling of Section 4.7.



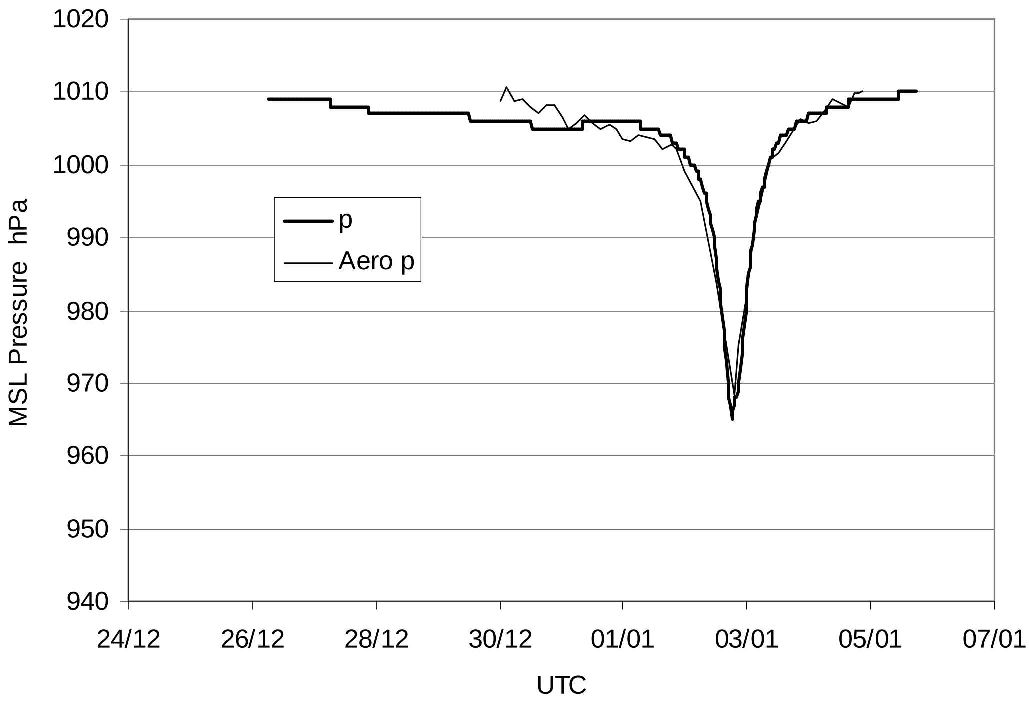
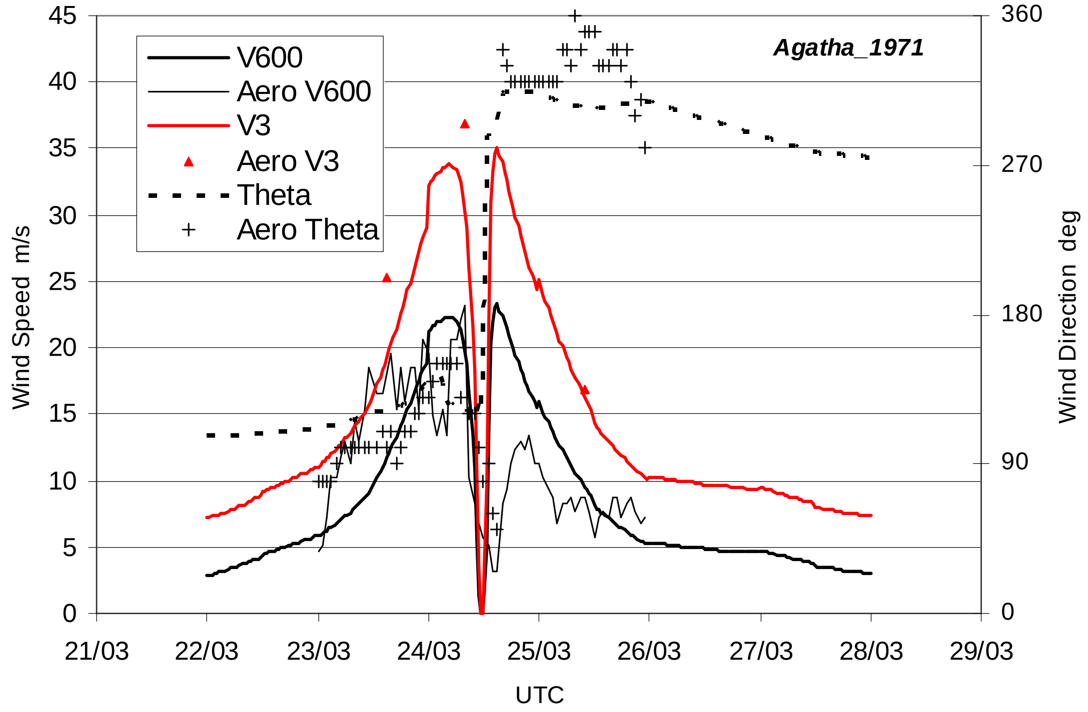


Figure 4.1 Wind and pressure calibration for TC Sally



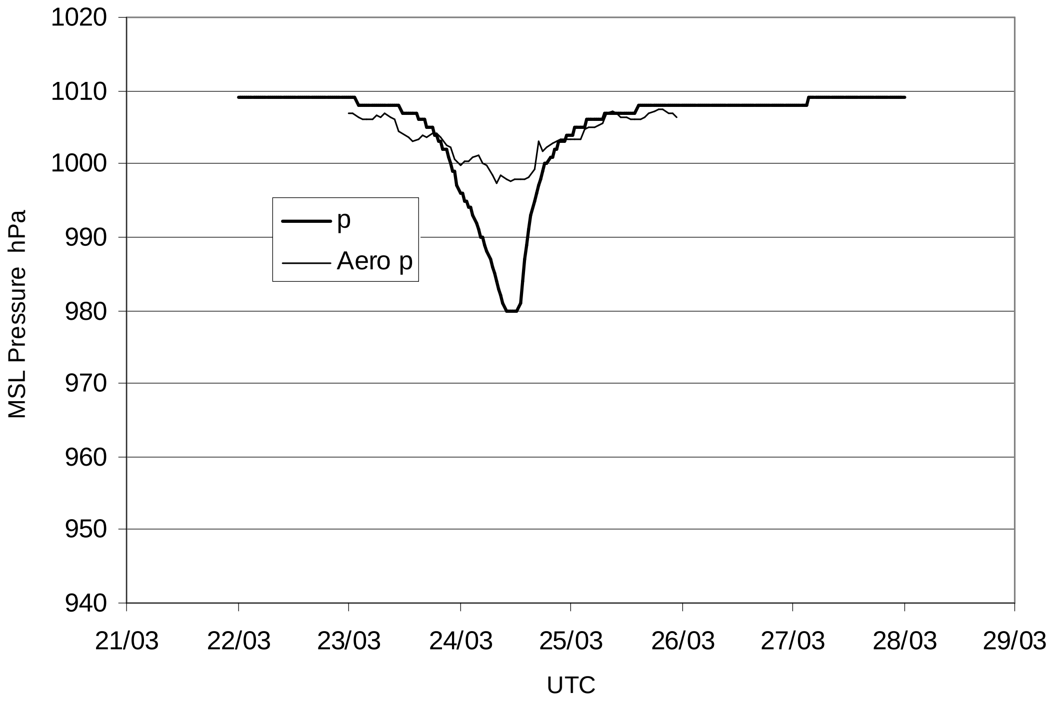


Figure 4.2 Wind and pressure calibration for TC Agatha

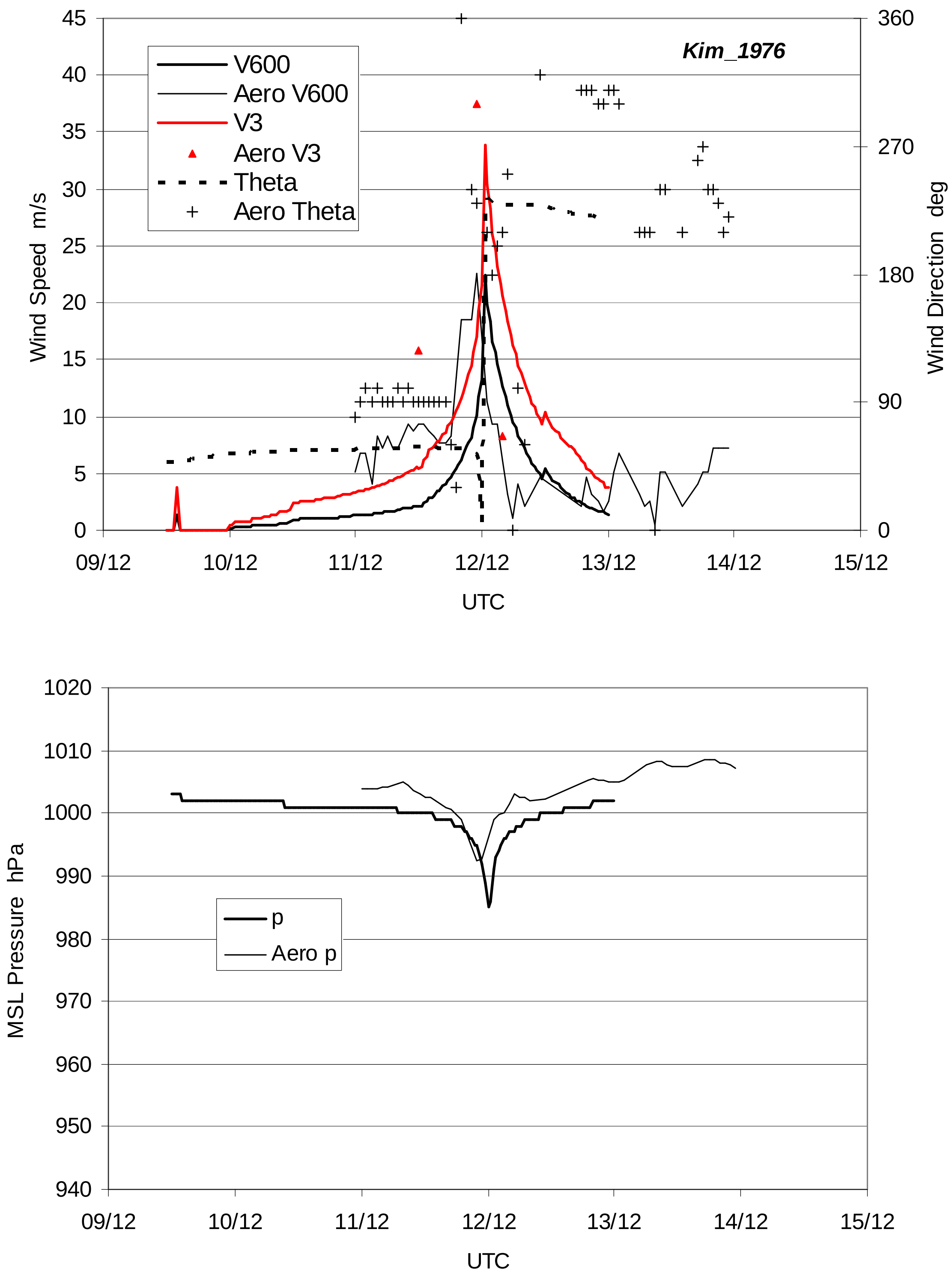


Figure 4.3 Wind and pressure calibration for TC Kim

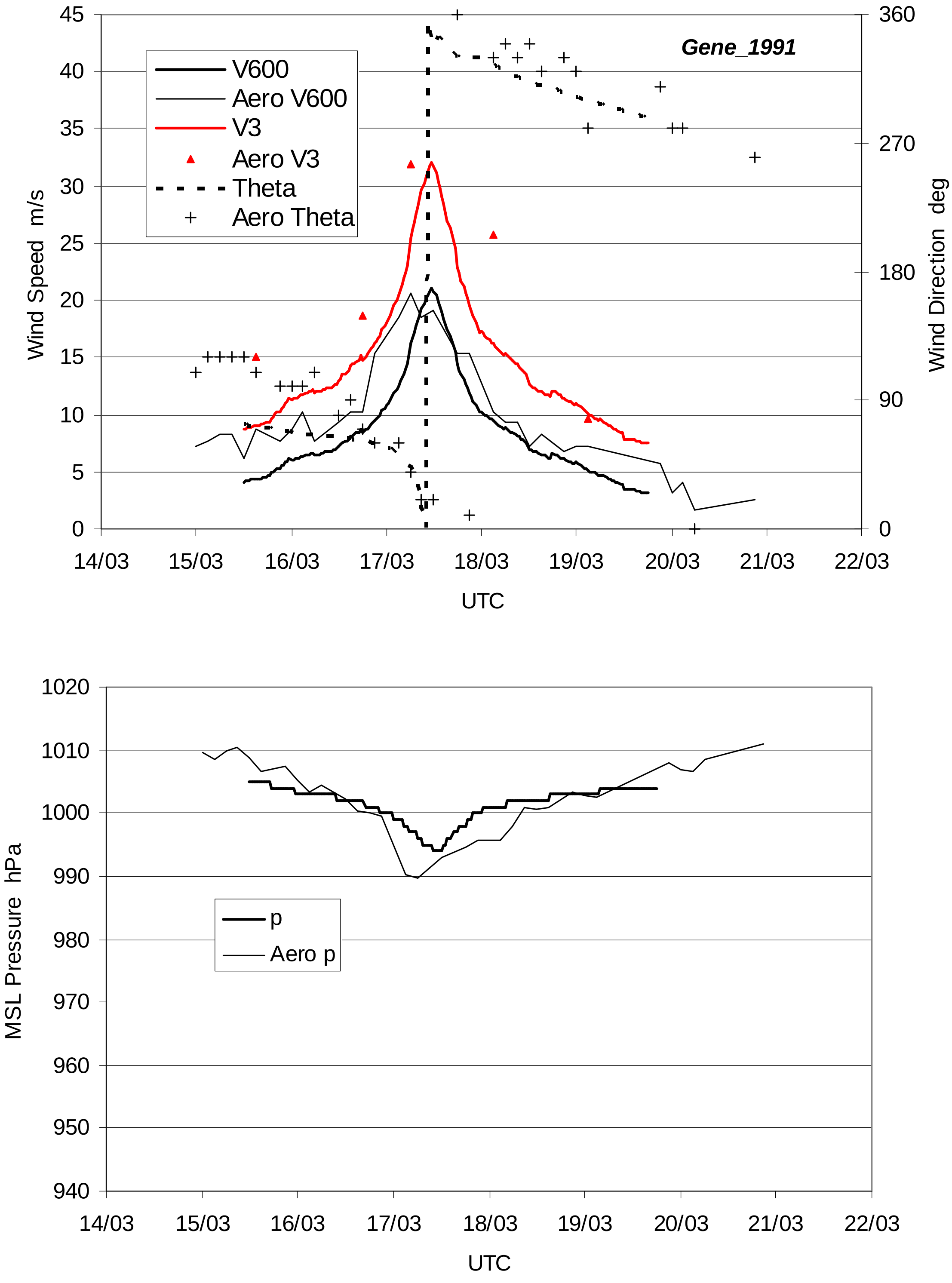
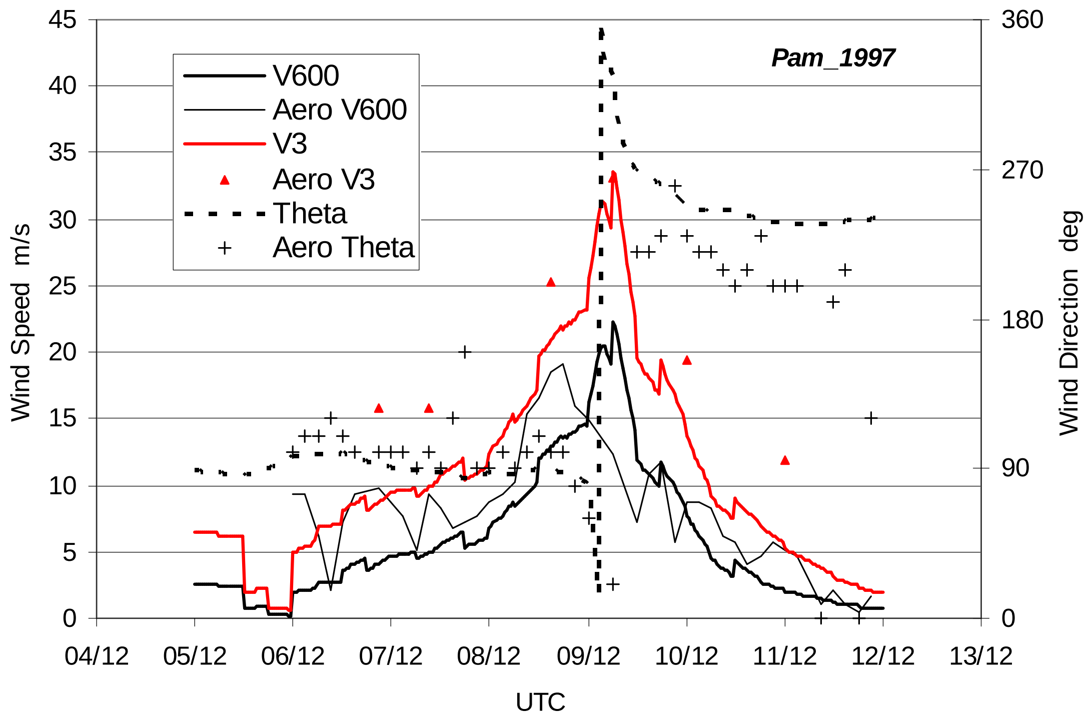


Figure 4.4 Wind and pressure calibration for TC Gene



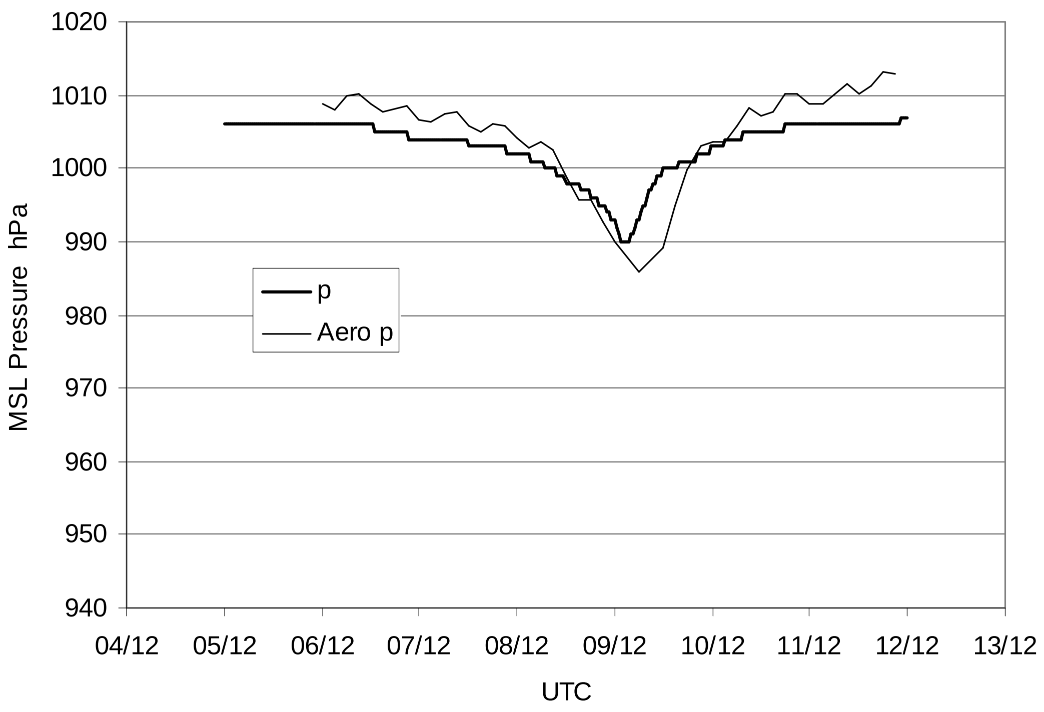
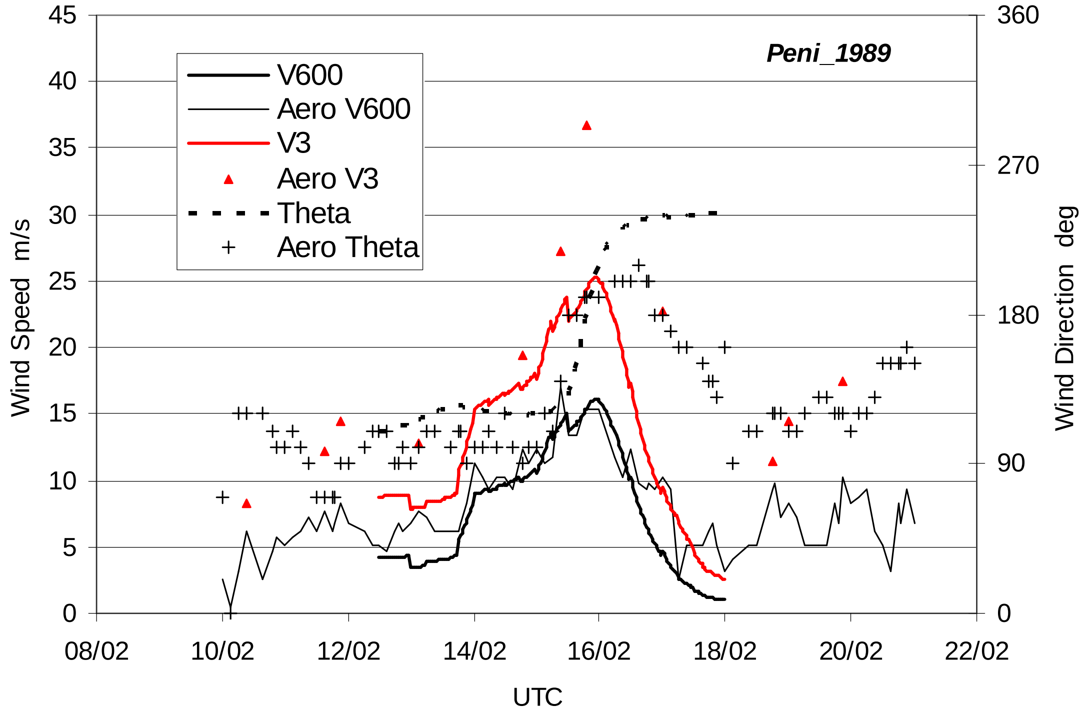


Figure 4.5 Wind and pressure calibration for TC Pam



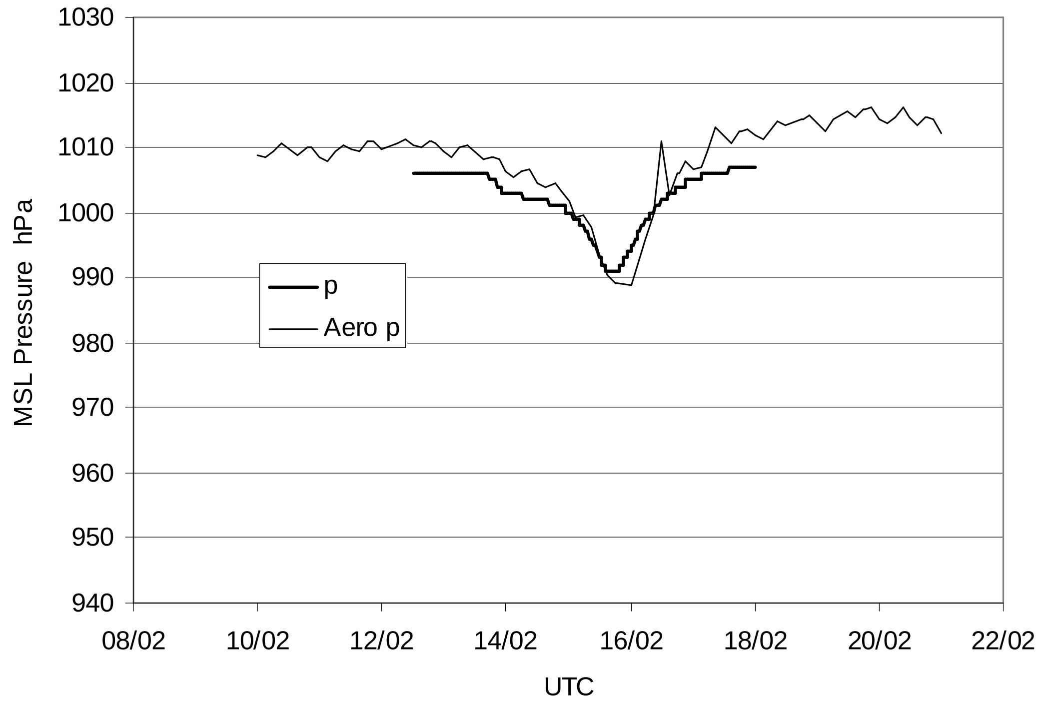


Figure 4.6 Wind and pressure calibration for TC Peni.

4.3 Spectral Wave Modelling

The 2nd generation ADFA1 spectral wave model (Young 1987) has been used to investigate the wave conditions likely to have been experienced in and near the island during each of the "top 5" cyclonic wind events and to form the basis of comparison with the simplified wave modelling system used in the SATSIM statistical storm tide model. TC *Peni* has also been modelled to allow comparison with the only other available detailed waverider record.

4.3.1 Wave Model Details

Three spectral wave model domains were established to undertake nested modelling of the historical tropical cyclones. The extent of these is shown in Figure 4.7 and summary parameters are listed below in Table 4.2.

Table 4.2 Spectral wave model parameters.

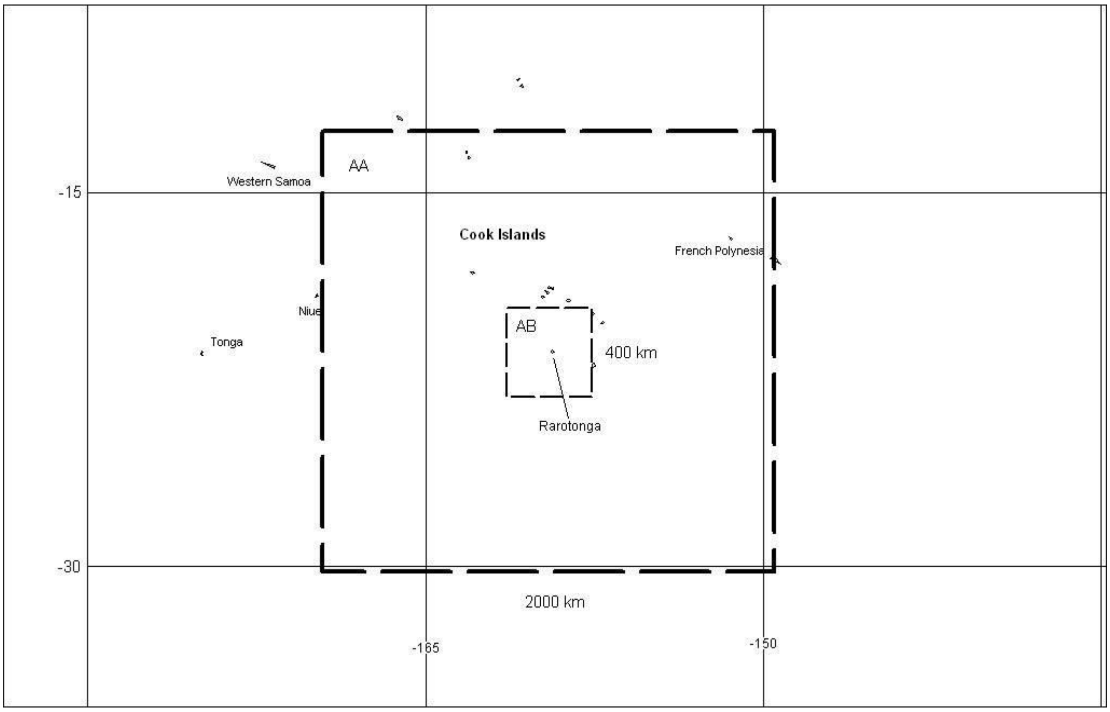
Domain	Extent	Resolution	Timestep
	km	km	s
AA	2000 x 2000	20	600
AB	400 x 400	4	600
AC	100 x 100	0.8	600

The water depths in the region are sufficiently deep (>1000m) such that no specific bathymetric details needed to be included, except in the AC grid, where representative depths were allocated to the nearest grid points to land, based on bathymetry from SOPAC Map 2A. This provides the model with some refractive capacity in close proximity to the island, which has been used to assess the likely extent of shielding from one side to the other during extreme wave events. However, the 100 m contour is typically within 500 m of the reef edge and the island presents much like a vertical cylinder in an infinite ocean. In the present study context the principal aim is to consider the likelihood of extreme deepwater waves impinging on the reef structures.

4.3.2 Wave Model Calibration

The only measured tropical cyclone wave data available to this study was obtained from reports on *Sally* (Croad 1989) at Avatiu and *Peni* (Barstow and Haug 1994), the latter recorded off the south-east coast of the island. Results of the wave modelling for these two storms are presented here to demonstrate the combined capability of the wind and wave models in reproducing the measured or reported conditions.

Figure 4.8 compares the modelled wave height and period for *Sally* with the plotted waverider data from Croad (1989), immediately offshore Avatiu – Avarua in a nominal depth of 100 m. The modelled H_s is slightly higher than reported at 8.8 m (versus approx 8 m) while the peak spectral period T_p of 12.5 s is well matched. The differences between modelled and measured T_p during the storm approach suggests some pre-existing sea condition that is not being represented in the cyclone vortex windfield model.



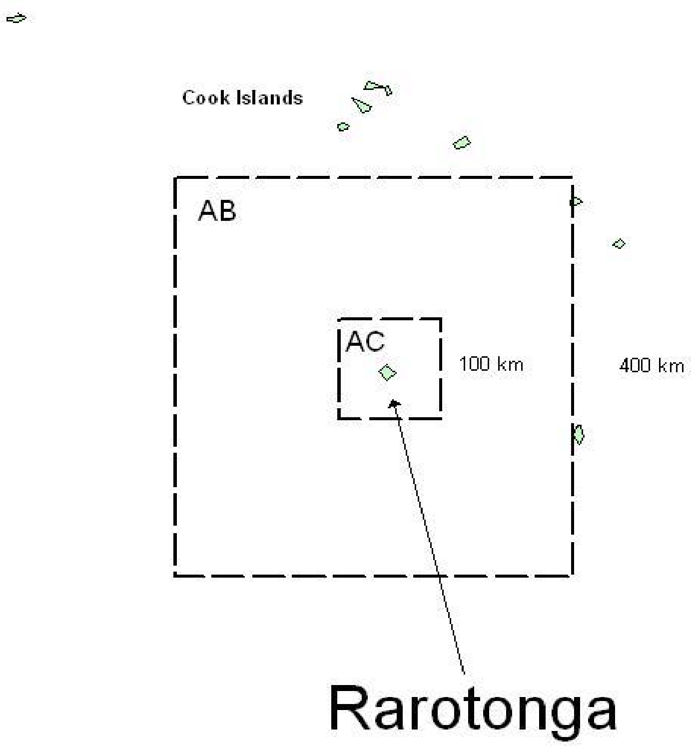


Figure 4.7 Spectral wave model domains.

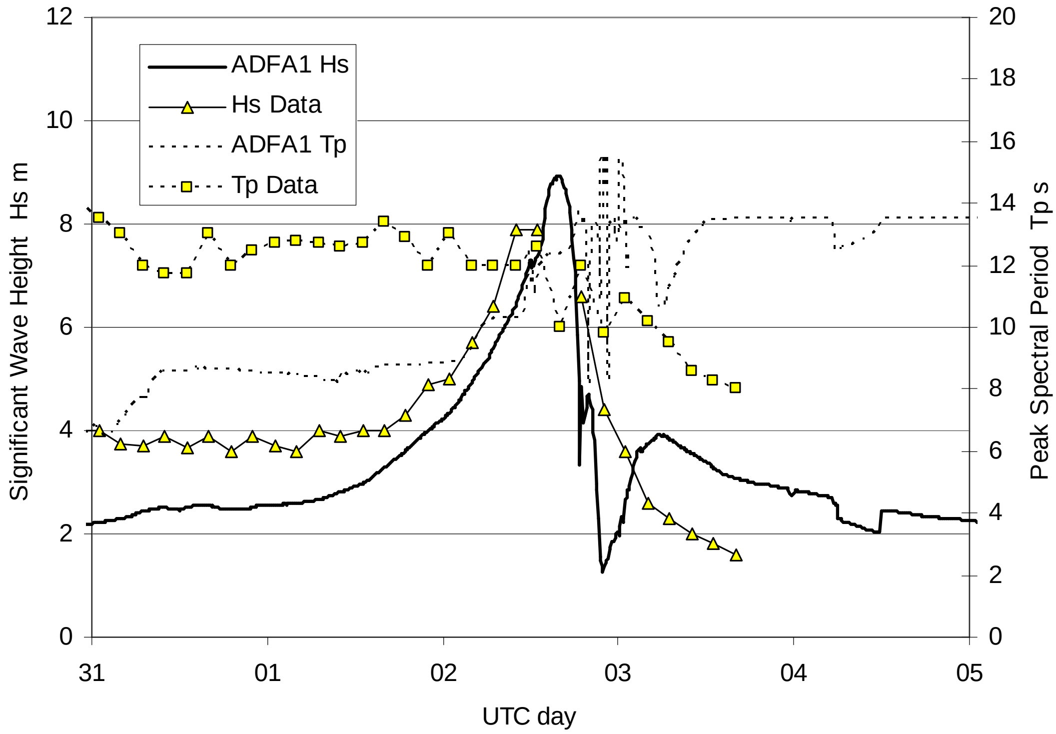


Figure 4.8 Wave model calibration for TC *Sally*.

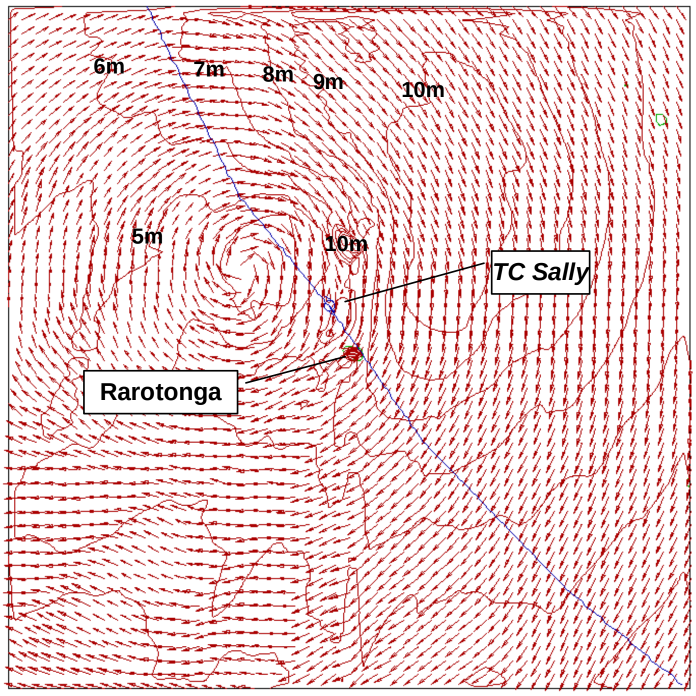


Figure 4.9 Spectral wave field after Cyclone Sally

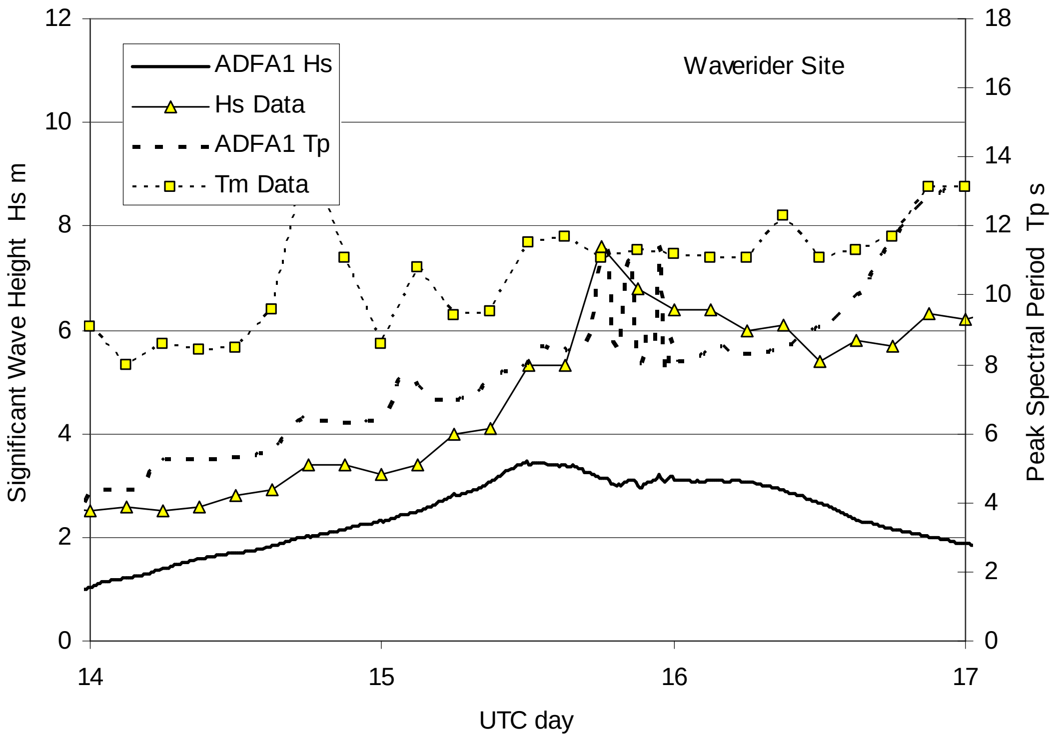


Figure 4.10 Wave model calibration for TC *Peni*.

This in turn is probably responsible for the observed phase lead of the H_s data compared with the model. After the eye passes, the model maintains a higher T_p. It is interesting to note that Croad also presents H_{max} data, which appears inconsistent with the H_s record near and after the peak. This suggests that the buoy may have been experiencing either flotation problems or, consistent with the presence of some data gaps, transmission difficulties. This leads some support to the possibility that the true peak might have been higher, similar to the modelled result. Overall, however, this is a very good comparison, which is consistent with the earlier reasonable matching of the peak winds and pressures of this direct-hit event. The modelled H_s data also appears reasonably consistent with the graphical satellite altimeter data for *Sally* presented in Barstow and Haug (1994), although it is difficult to discern detail in their figures.

Figure 4.9 presents a two-dimensional view of the spectral wave model field shortly before Sally passes over Rarotonga. It can be seen that the highest waves are generally predicted to have been further east of the island but there is a smaller peak region closer to the centre of the storm and directly north of the island at this time that ultimately creates the modelled 8.8m value at the waverider site. It can be seen that the wave pattern around a tropical cyclone is very asymmetric and can be very complex.

Figure 4.10 then compares the modelled wave height and period for *Peni* at the location of the waverider buoy offshore of Muri on the south-east coast. The modelled H_s of 3.5 m can be seen to be much lower than the measured value of 7.6 m although the wave periods (T_m from the buoy and T_p from the model) eventually begin to approach each other. This is a very poor comparison, which prompted a further investigation here into the meteorology of TC *Peni*.

Firstly, the original wind and pressure calibration was revisited, together with the possibility that the anemometer may have experienced shielding from the southerly winds. Also, the possibility that *Peni* may have been more intense than estimated was compared with the Mauke and Rarotonga data. While a more intense storm could be justified by choosing another set of model parameters, this would not significantly increase winds near Rarotonga above those that were measured.

By examining NOAA NCEP numerically hindcast surface wind and pressure fields for this region at the time, it is clear that *Peni* was imbedded within a very large monsoonal low pressure system Figure 4.11. This resulted in the formation of an extensive area of gale force winds to the south and east of Rarotonga during the 24 to 48 h period prior to closest approach of the storm. The evidence for this can be seen in the waverider mean period being already as high as 9 s on the 14th, when the model was still predicting local T_p values as low as 5 s. The measured periods then peaked even higher as the wave height increased. Some attempt to reproduce these features within the available time was attempted but without significant success, partly due to the difficulty of reliably co-locating the NCEP analysis fields with the actual track of *Peni*. The data sparseness of this region means that even the NCEP reanalysis is approximate only. Nevertheless, it can be concluded that the measured waves are due largely to the monsoon low and not the cyclonic vortex. This is evident by considering a theoretical wave growth scenario (e.g. Young and Verhagen 1996), which yields H_s in the range 4.5 to 5.0 m and T_p of about 10 s for a fetch of 600 km and duration of 24 h.

In conclusion, the presently modelled waves from *Peni* are 50% lower due to the non-representation of the monsoon winds and are probably an overestimate of the actual vortex-induced wind conditions. For sites on the southern side of the island, this type of influence could be significant. However, the present study is considering the northern coast, which will experience its peak wave conditions during close approach of cyclones from the north. Hence, non-inclusion of monsoonal low effects is not considered detrimental to the present analysis.

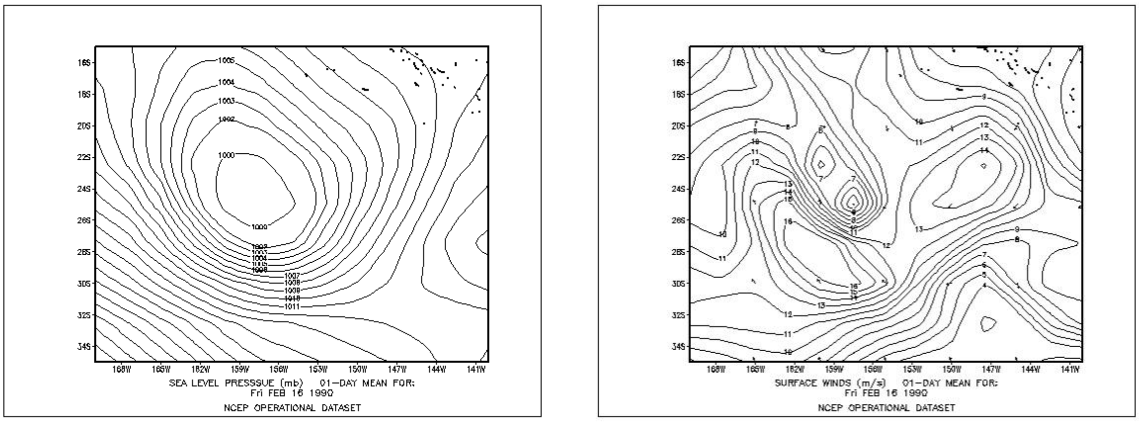


Figure 4.11 NCEP Reanalysis MSLP and winds on 16/02/1990.

4.4 Parametric Wave Model Performance

The statistical prediction of wave setup is based on wave height and period obtained from the simplified SATSIM parametric wave model. While it is expected that the SATSIM model will closely follow predictions from the ADFA1 model upon which it is based, individual historical storms will produce results at variance with the simplified assumptions of constant intensity and track etc. The issue then is not the absolute accuracy, but whether the model can be considered statistically unbiased.

Figure 4.12 compares the SATSIM model prediction of wave height and period with the earlier example of ADFA1 versus the measured data. In this case, SATSIM predicts a peak H_s of 8.6 m, part way between the measured data and the ADFA1 peak and the T_p values are well matched. The width of the SATSIM hydrographs are greater than the measured data and fall below those levels away from the peaks, but overall the model produces a credible realisation of the actual event.

As there are no other direct comparisons that can be made with actual wave data it has to be assumed that the model will provide similarly credible comparisons in other situations. As a guide to this possible performance, the comparison of the SATSIM parametric model and the ADFA1 predictions for each of the historical storms so far considered is summarised below in Figure 4.13. The top panels compare SATSIM results with ADFA1 AA grid results, which are essentially "spot in the ocean" estimates. These show that the SATSIM model is generally unbiased in H_s and T_p for this small sample, although in T_p space, SATSIM tends to overpredict at short periods. As a result, this suggests that the model may tend towards slightly conservative estimates of wave setup. The bottom panels compare site-specific estimates offshore Avatiu – Avarua using the local shielding algorithm in SATSIM and the ADFA1 AC grid result, where the H_s and T_p bias can be seen to be slightly higher than before.

These comparisons show that the SATSIM model will reasonably reliably estimate the peak wave height and spectral peak period for severe tropical cyclones affecting Rarotonga. When combined with the statistical climatology, which utilises straightline tracks, it can be expected that the model will be reasonably unbiased in H_s but may tend to overestimate T_p for the lower H_s events. This might slightly overestimate wave setup in such conditions.

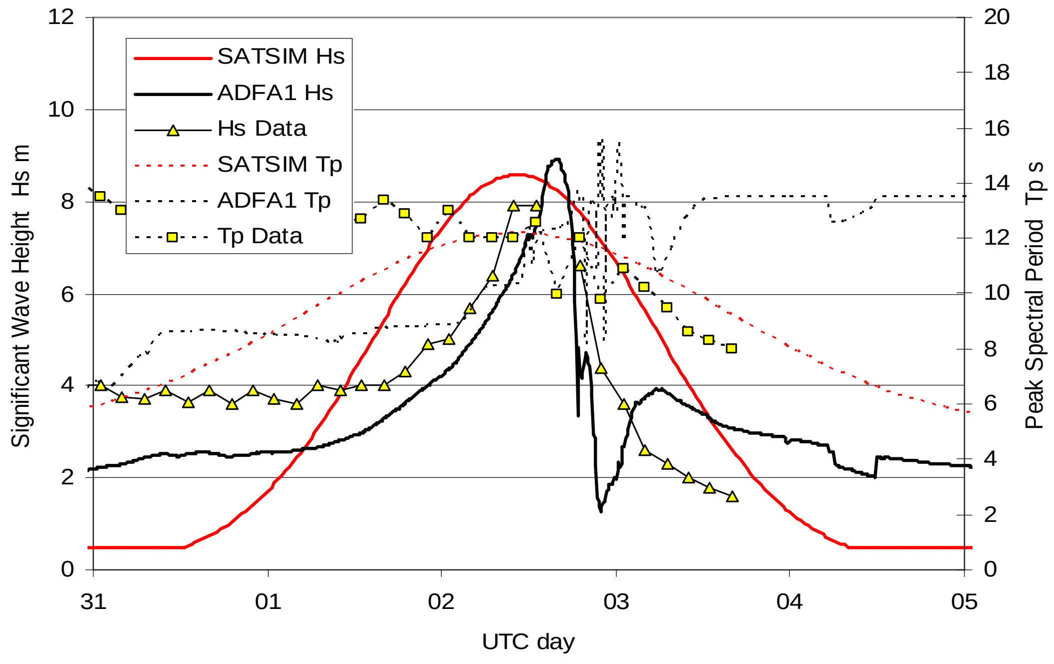


Figure 4.12 Parametric wave model prediction for TC Sally

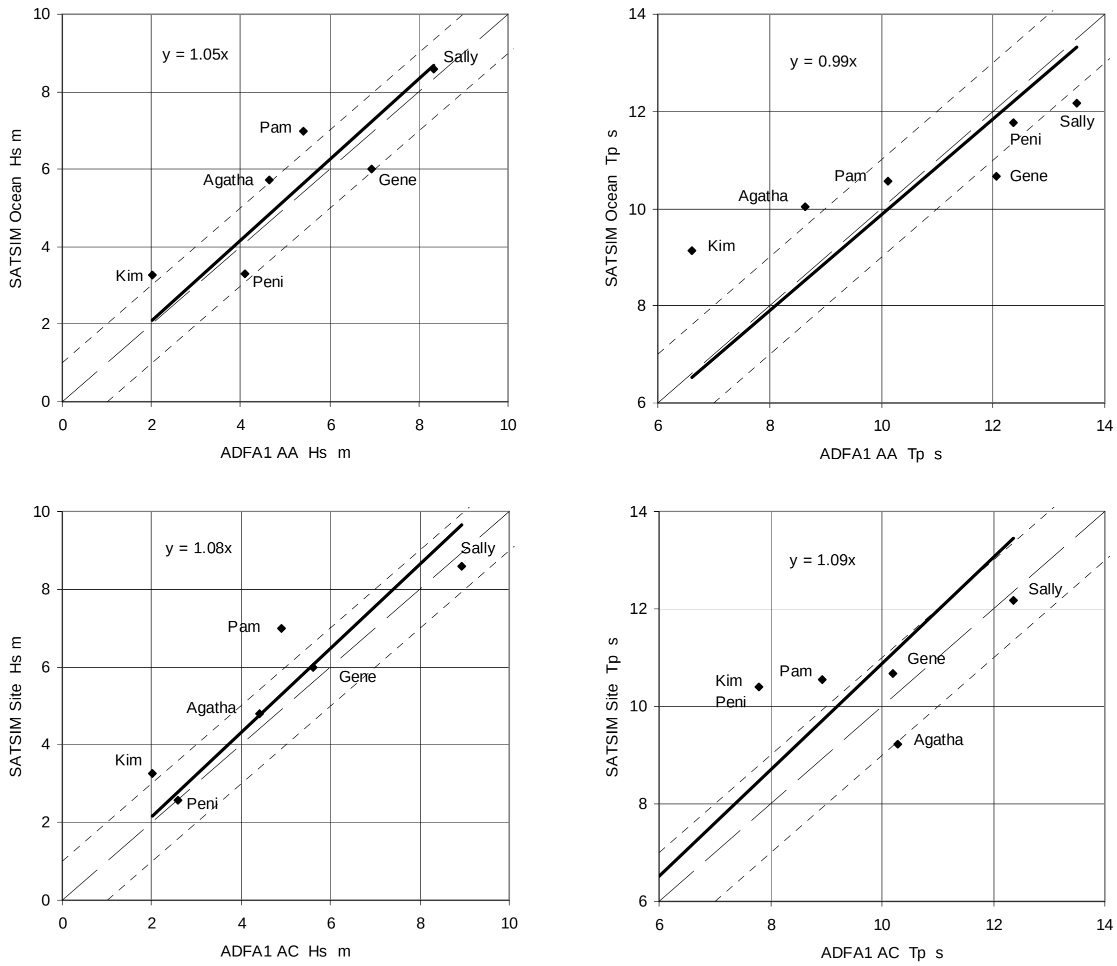


Figure 4.13 Validation of the SATSIM parametric wave model.

4.5 Tide, Surge and Wave Setup Modelling

The SATSIM model is used to represent the combined time history of astronomical tides, deepwater storm surge (Inverted Barometer Effect or IBE) and breaking wave setup. Wind stress induced setup in this deepwater environment is expected to be immeasurably small and is neglected. The data for quantitative comparisons is essentially limited to that available from the SEAFRAME water level recorder during TC *Pam* in December 1997. During TC *Sally* in 1987, the NOAA gauge at Avarua failed before the arrival of the peak conditions. However, because of the location of the water level gauges in Avatui and Avarua harbours, which provide passages to deepwater, the gauge data does not provide an estimate of the wave setup that was likely to have occurred over the adjacent reef flats at the time. This is because the incoming waves at the harbours will dissipate against the vertical walls, and the harbour itself will act as a high velocity return flow path from the adjacent reef flats, that itself will further limit harbour wave effects by “tripping” and spilling incoming waves. Accordingly, the gauge records are not considered reliable indicators of the total water level being experienced nearby on the reef top flats.

Of more value to this analysis then is the anecdotal evidence for actual inundation events that have occurred, even though the actual levels are sketchy. For example, the inundation from *Sally* was undoubtedly very significant and there exists some photographic records that have been utilised here to prove the accuracy of the proposed analysis methods.

4.5.1 TC *Sally* 1987

A detailed hindcast of the water levels during TC *Sally* has been undertaken in an attempt to reconcile the various items of photographic evidence with the recorded winds, waves and tide gauge data. As is common with such extreme events, there is much commentary and opinion regarding what actually occurred and the details are sketchy or inconsistent. For example, Frost (1987) reports "*I visually estimated that 'normal' sea-level at the height of the storm surge peak just as the eye approached was <u>at least 5 metres</u> above normal. At the OIC's house at Tupapa considerable coral rubble reached the front of the house which is 8 metres above sea level*". While there is ample photographic evidence of (small) coral rubble within the streets immediately adjacent to the harbours, the eye actually "approached" around midnight and it is difficult to judge the overall accuracy of such statements, especially in regard to the reported levels. The present study has highlighted difficulty in obtaining accurate vertical elevation data along the foreshore. Notwithstanding this, the modelling suggests that Tupapa may well have experienced greater wave exposure than the Avatiu-Avarua area.

Considering the photographic evidence in and around Avarua, an attempt has been made to estimate the timing of the images relative to the storm passage and the likely combinations of wind, waves and total water level. In this regard the assistance of Mr Don Dorrell has been very helpful. Figure 4.14 presents a detailed view of the events in local time (UTC-10h) with the recorded history of airport winds and waverider measured wave conditions. The direction of waves at this time based on ADFA1 results was at a bearing of about 22˚, which is close to perpendicular to the shore. The estimated timing of the three sets of photographic evidence (Appendix G) has been attributed to (1) around 15:00 on 01/01/87 before winds increased much above gale force, (2) during the calm of the eye at 09:00 02/01/1987, and (3) around 16:00 on 02/01/87 when the eye moved off the island. In (1) the harbour water level at Avarua is approximately 0.7m above MSL although wave overtopping of the wall and adjacent reef top water levels are at about 1.8 – 2.0m above MSL. In (2) where a man is caught by a bore while carrying a gas bottle near Banana Court water is at an elevation of about 2.5m -2.9m MSL. It should be noted that the photographs are taken at the peak of a bore of water overtopping the coastline and road. These bores typically inundate the area for a duration of one to two minutes (pers comm. Don Dorrell) and are not allowed for in the equations developed by Gourlay (pers comm.) The Bore effects were difficult to generate in the laboratory in the studies that were used to provide data for derivation of the equations. Bores have been observed in prototype, notably on Rarotonga during several cyclones.

In photograph (3) taken later in the day there is no water in the township.

In the bottom panel of Figure 4.14, five specific times have been chosen at which to manually calculate estimated total water levels caused by the combination of the tide, IBE and breaking wave setup. Three of these times align with the estimated timing of the photographs. The SATSIM predicted tide level is shown and also the SATSIM predicted IBE, with the combined level labelled as the expected "Ocean Level". Of some interest here is the actual NOAA tide gauge level before the instrument failed. At that time it was indicating around +0.4 m above the estimated ocean level. This is probably not unreasonable given the likely complexity of flows into the harbour at that time as illustrated by the photographs at this site. The calculated 1% water level around this time can be seen to vary on the basis of which reef-top profile is used and ranges from 1.7 m to 2.3 m MSL. Combined with likely bore effects which are expected to have a high probability of creating temporary increases in water depth, at the shoreline, of 70% -100% these values are believed consistent with photographs at this time.

The next two calculations of water level are aligned with the high tide closest to the peak of the storm around 11pm that night and at 3 am the next morning around the time of the peak waves. In the latter case the peak wave estimate has been increased slightly to 8.5 m in view of the earlier reported doubts about the buoy behaviour. The range of resulting water levels at the 1% level is then 2.6 m to 3.5 m. The road levels at Avarua (1.9 m) and Avatiu (2.6 m) are also shown for comparison. It is also reported that water at some time during the night also extended to the highest point on the road (4.8 m) between Avarua and Avatiu. Commentary on this is included later. At the location where the road is 4.8m above MSL the applicable 1% estimate of water level is 2.6 m. Thus to have a limited amount of water crossing the road it is necessary to have a mechanism to temporarily increase water levels by 2.2m.

At the time of the next photograph (2) the wind is calm within the eye but the IBE is high and the tide is rising. Since the waves are still quite high, the estimate 1% total water level is in the range of 2.4 m to 3.1 m MSL, which agrees well with the photographs at this time showing water flowing across the road at Banana Court with a level between 2.4 and 2.9 m MSL.

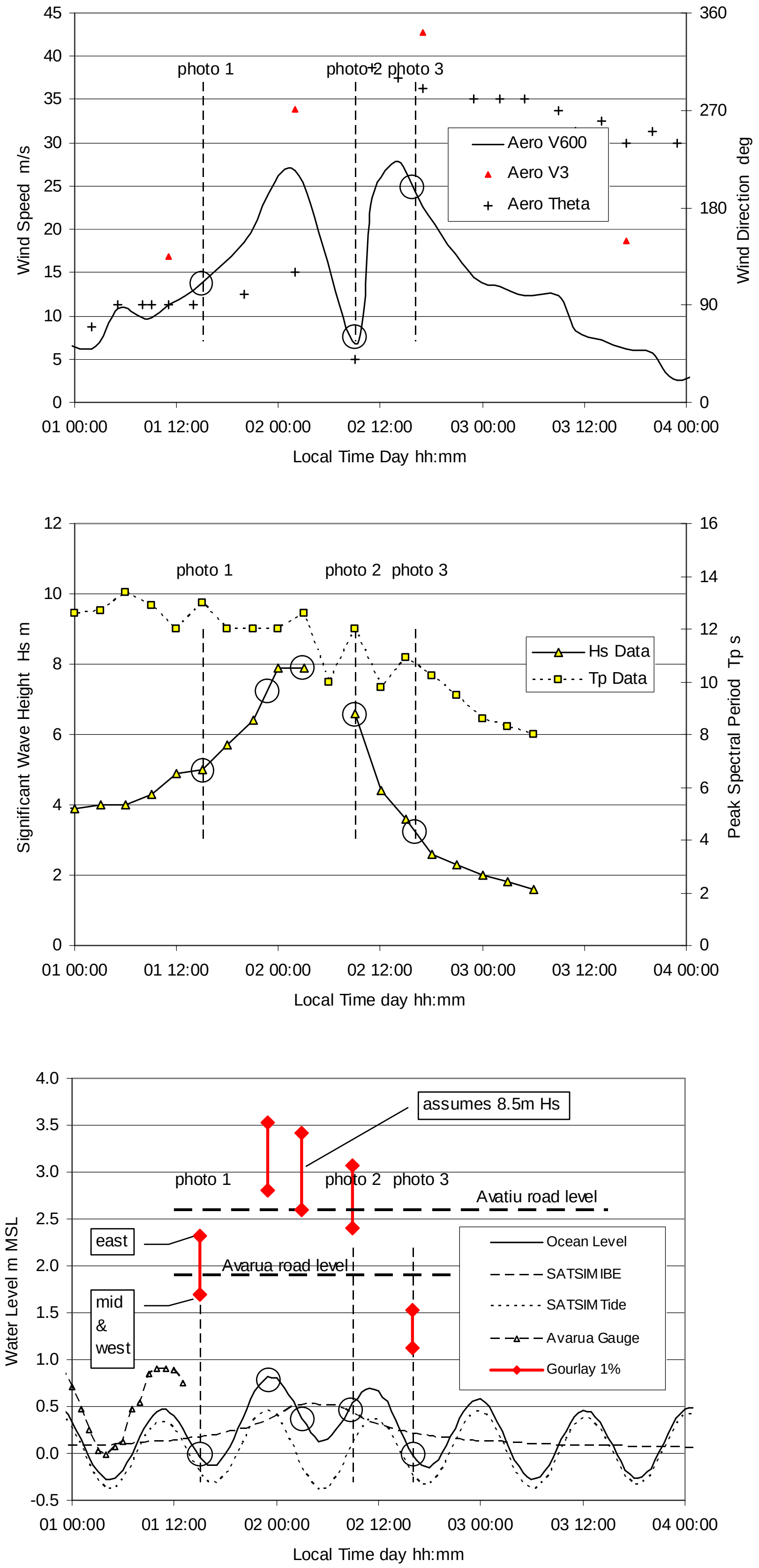


Figure 4.14 Detailed hindcast of TC Sally water levels.

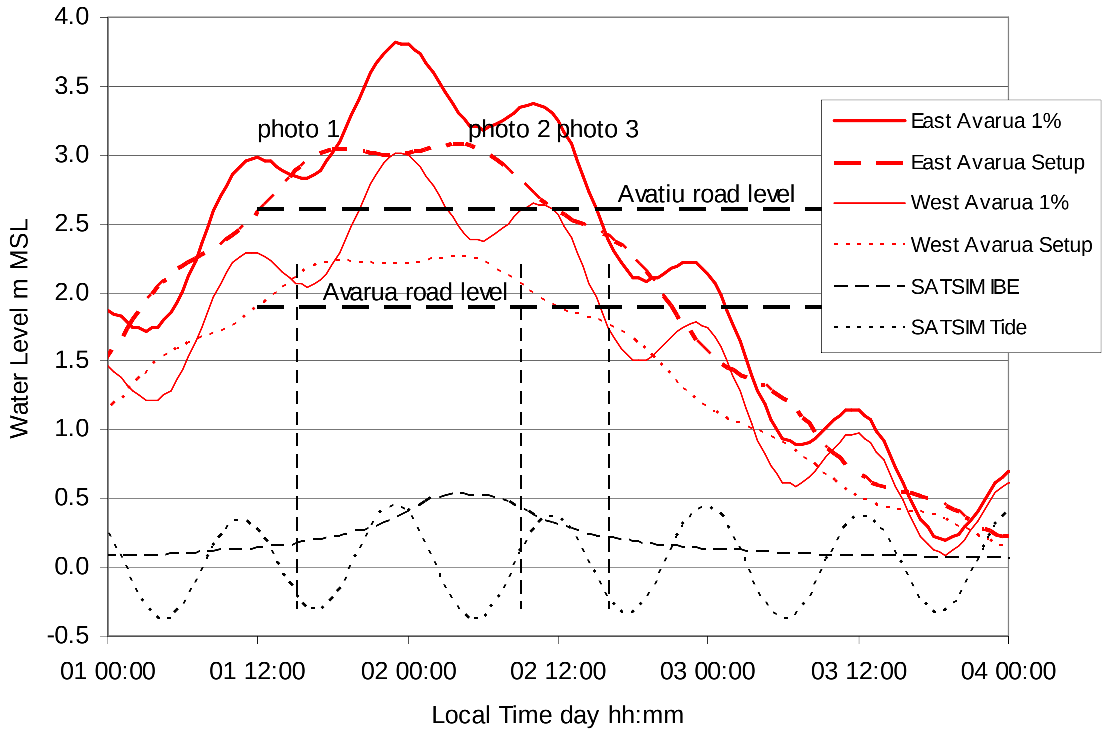


Figure 4.15 SATSIM prediction of TC Sally water levels.

The final photograph time (3) is at 4 pm on 02/01/87 when water levels are estimated to have dropped to between 1.2 and 1.5 m due to the rapid decrease in wave height and the occurrence of the low tide. These levels are consistent with the absence of flooding in the photographic evidence.

Overall, these results demonstrate that the preferred method of calculating reef top wave setup (Gourlay 1997) provides a very realistic description of the water levels experienced during TC *Sally* at Avatiu-Avarua. Figure 4.15 completes this assessment by presenting the full SATSIM prediction of water levels, including the individual wave setup component magnitudes. As expected, the more conservative SATSIM wave hydrograph results in slightly higher levels being experienced but the overall behaviour is similar.

4.5.2 TC *Gene* 1992

A complete water level record from the NOAA gauge at Avarua harbour, as previously discussed in regard to TC *Sally*, is available from the University of Hawaii Sea Level Center (UHSLC). There is no wave data for this event but it is understood that inundation did not occur. Figure 4.16 firstly compares the wave height and period predictions from ADFA1 and SATSIM, where the H_s values agree within 0.8 m and the T_p values within 1 s, with SATSIM being the lower. Next the water level comparison shows that the harbour gauge reached a peak level of 1.3 m MSL on the back of a 0.34 m tide and an estimated IBE of 0.16 m. This results in a “harbour wave setup” component of 0.76 m, while the SATSIM predicted 1% total water levels range between 1.7 m and 2.3 m MSL. Since the ADFA1 wave conditions were more severe, the equivalent predictions based on its peak values are also shown ranging between 1.8 m and 2.5 m MSL. These levels are at least 1 m lower than those predicted during TC *Sally* and consistent with the absence of any significant inundation.

4.5.3 TC *Pam* 1997

A complete harbour water level record was available from the SEAFRAME gauge at Avatiu during TC *Pam* in December 1997 but no wave data exists. It is also understood that water levels also did not rise high enough to create an inundation problem. Accordingly Figure 4.17 presents the comparison between ADFA1 modelled wave conditions and SATSIM, which shows about a 2 m overprediction by the parametric model and also an increased peak wave period. In the bottom panel is a comparison of water level predictions. The actual recorded water level within Avatiu harbour reached 0.92 m MSL, which comprised a 0.27 m tide and a nearly identical IBE component (the SEAFRAME also measures MSL pressure and the NTC calculate the IBE effect). This gives a “harbour wave setup” component of 0.38 m.

The SATSIM predicted tide is identical to the NTC tide but the IBE effect is slightly lower (0.07 m) than the recorded values. This can be traced to the less than perfect wind and pressure calibration for *Pam*, whereby on the basis of the available storm track and intensity values, the pressure model cannot accurately reproduce the measured surface pressure data. Of more interest are the SATSIM predictions of the 1% total water level, which range from 2.2 m to 2.8 m MSL. These would have equalled the lower limit of the predictions during *Sally* and appear to be overestimates. This is confirmed by comparing the 1% total water levels calculated on the basis of the ADFA1 wave parameters at the time, rather than the much higher SATSIM values. This results in a range of 1.6 m to 2.1 m MSL, more consistent with the absence of any extreme inundation.

4.5.4 Commentary

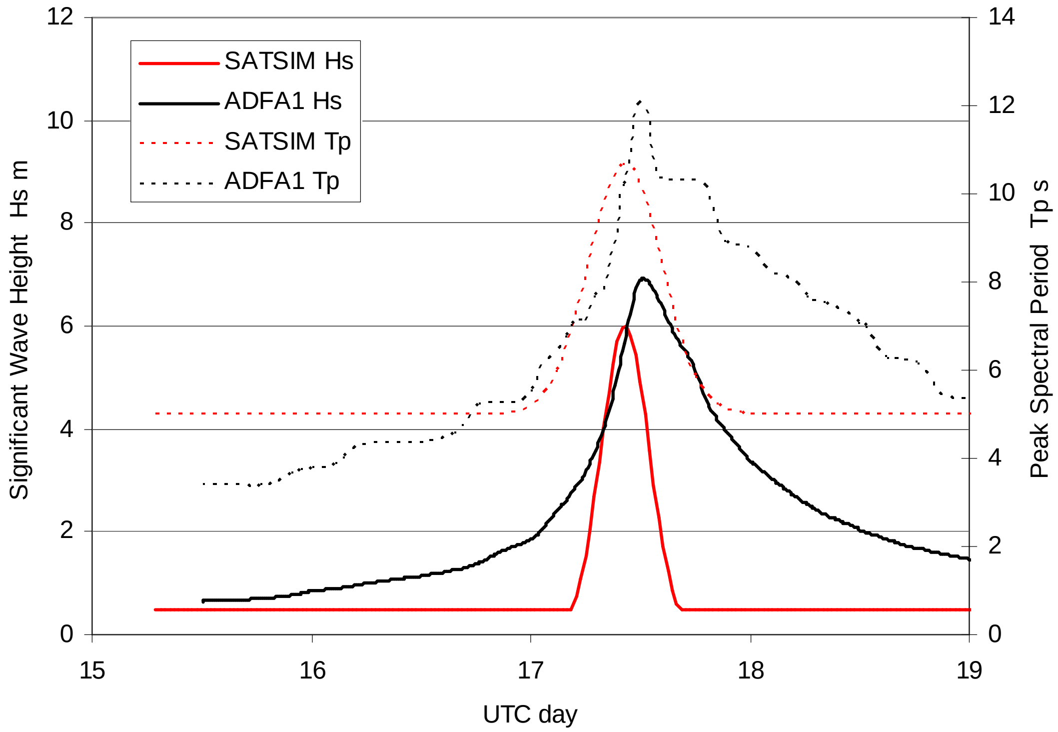
Three historical cyclone events have been considered in assessing the suitability of the analysis procedures where there is some indication of water levels. While TC *Sally* is the only event that caused any significant inundation, the fact that the other two storms did not apparently cause a problem is also useful information.

The results detailed here are very encouraging in terms of the order of magnitude of the predicted impacts and the photographic evidence. Overall, the modelling confirms that during the night passage of TC *Sally*, a significant inundation event occurred. The fact that some water is also reported to have reached a level of about 4.8 m on the road between Avarua and Avatiu is not inconsistent with the peak estimated 1% wave setup water level of approximately 3.5 m MSL. An additional 1 m or more could be attributed to wave runup and intermittent bore effects during the several hours when conditions were at their worst. Also, during the peak period of inundation, onshore winds were at their highest and local wind stress would also enhance the setup and runup effects. During the eye, the photographed inundation is consistent with the modelled 1% water levels in association with intermittent “bore” events.

In contrast, the modelling of the other less intense events *Gene* and *Pam*, suggests that their peak conditions would have only been similar to the *Sally* afternoon scenario on 01/01/1987 (Photo 1).

Table 4.3 Comparison of 1% total water level estimates.

Storm	Sally			Gene		Pam		
Site / Basis	pm	night	am	SATSIM	ADFA1	SATSIM	ADFA1	
East Avarua	1.7	2.6	2.4	1.7	1.8	2.2	1.6	m MSL
West Avarua	2.3	3.5	3.1	2.3	2.5	2.8	2.1	m MSL
Average	2.0	3.1	2.8	2.0	2.2	2.5	1.9	m MSL



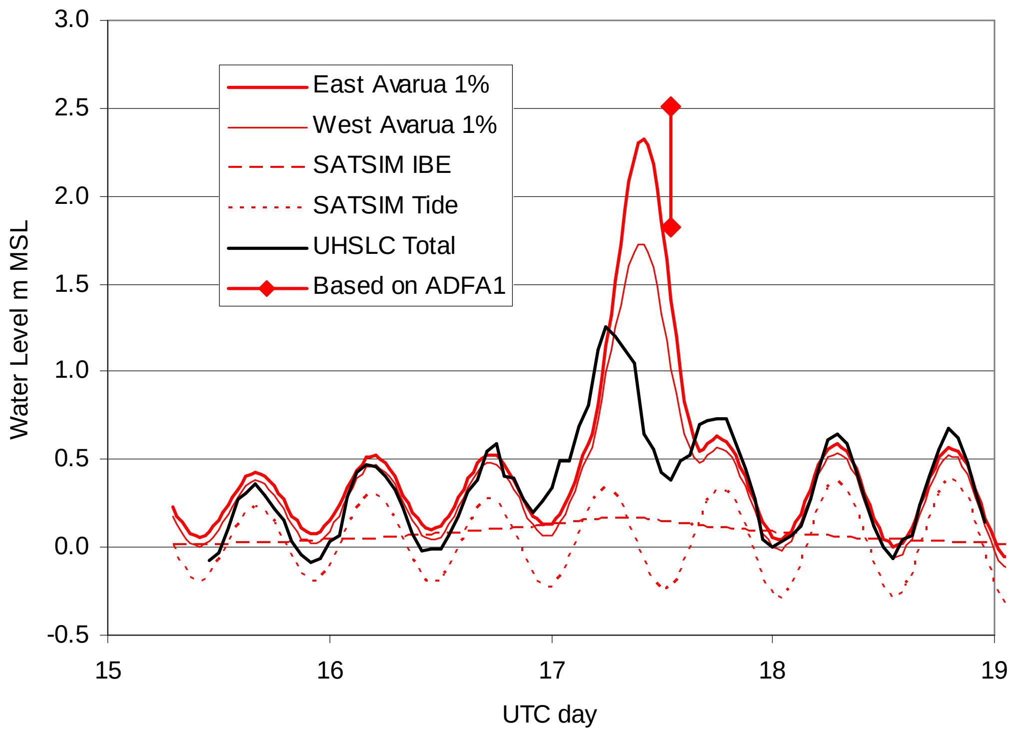


Figure 4.16 Measured and modelled conditions during TC *Gene*.

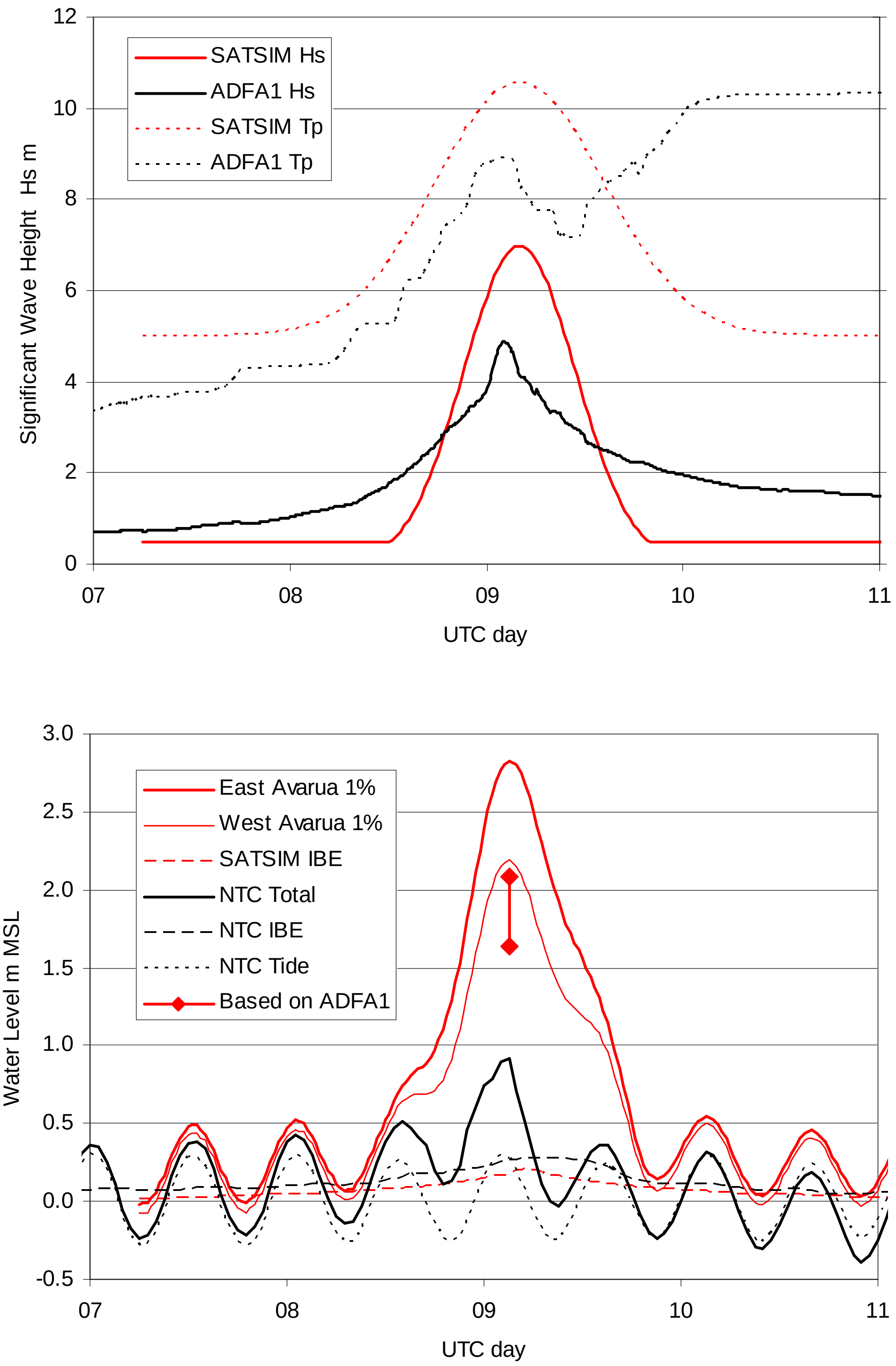


Figure 4.17 Measured and modelled conditions during TC *Pam.*

4.6 Statistical Model Validation

4.6.1 Astronomical Tide

The SATSIM model generates a tidal signal during the occurrence of each simulated tropical cyclone. This is done on the basis of a synthetic real date and time, which is restricted to the summer half-year period only. The accumulated statistics of the highest tidal level generated by the model are then expected to compare favourably with the tidal plane estimates provided by the NTC. The comparison with NTC tidal planes based on simulating 50,000 years of synthetic tropical cyclone occurrences is presented below in Figure 4.18.

In this example, tides have been calculated at a 30 min resolution for periods of 36 h during each cyclone and a 0.05 m interval has been used for the exceedance calculation. The results show that the long term simulated highest tide levels asymptotically approach HAT at around the 100 y recurrence interval. This compares with the expected occurrence of HAT at the 18.6 y cycle if all tides on all days were sampled. The average number of cyclones modelled per year is 1.3 within a 500 km radius (from Table 3.1), hence only about 50 h per year are being sampled, thus creating a time shift in the exceedance calculation. Accordingly the model shows that even though the tidal range is small, the full HAT range will only become fully utilised as part of a total water level at return periods of about 100 y.

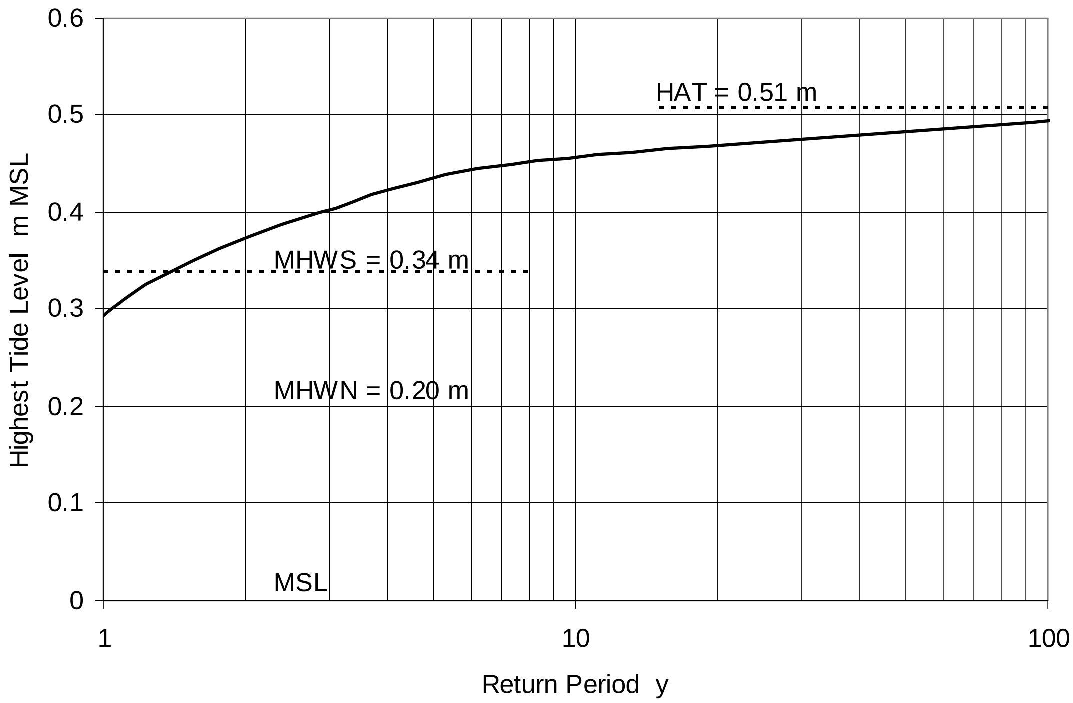


Figure 4.18 Simulated highest astronomical tide levels.

4.6.2 Wind

The statistical storm tide model can only be effectively calibrated on the basis of the long-term wind records, the analysis of which in Section 3.3 represents a completely independent estimate of the intensity and frequency of tropical cyclones affecting Rarotonga. It is assumed that if the modelled wind speeds from tropical cyclones at Rarotonga can be shown to effectively reproduce the trend of measured winds over the past 34 years of available electronic records, then the model can be regarded as reasonably accurate. Given that the modelled waves and hence wave setup are then linked to the modelled winds, it follows that the water level estimates will also be reasonably accurate in a statistical sense.

The principal objective calibration parameter available is the relative phasing of storm lifetimes as they pass through the arbitrary 500 km radius of the statistical model. This controls the proportion of storms which reach their full intensity either before or after passing by Rarotonga. As was evident in the climatology analyses of Section 3.1, the island is situated away from the principal generation region to the west and north, with a significant gradient in occurrence and intensity between a 500 km radius and a 1000 km radius of influence. Sensitivity tests were therefore undertaken to determine the optimum storm spatial distribution, which was found to be consistent with the distribution of storm starting positions within the 500 km radius.

The secondary but subjective calibration control available is to adjust the objectively determined central pressure relationships for each of the two identified tracks (i.e. Figure 3.10). Even some significant changes to the central pressure distributions failed to significantly alter the result. This is likely due to the very low frequency of occurrence of cyclones within the 500 km radius. However, a slightly conservative position has been taken that assumes that it is likely that the intensity of the more intense historical storms might have been underestimated. This is predicated on the difficulty of obtaining good satellite imagery in the region and the sparseness of ground truth data. Accordingly a conservative bias has been applied to the objective analyses of Section 3.1 that reduces the minimum central pressure by about 20 hPa at the 1000-year return period.

The results of the calibration and validation against the measured winds is summarised in Figure 4.19. Here the analyses of Section 3.3 are reproduced again for the separate cases of the mean wind speed V_m and the peak daily gust wind speed V_3 but extended to the 1000-year return period. Overlaid on each panel are the 95% confidence limits of the statistical fit to the data and the independent SATSIM model result.

It can be seen that the model very accurately follows the measured mean wind speed behaviour. The modelled peak gust deviates early from the theoretical data fit but still trends well compared with the higher gust data values. In this case the model assumes a “standard” V_3/V_m ratio of about 1.4 which, as previously mentioned, might be less than that being caused by interaction with the local terrain. Overall, the model performance is considered very good and illustrates the expected physical limiting of extreme winds at low probability levels, rather than the unlimited prediction based on the theoretical statistical fit to the data.

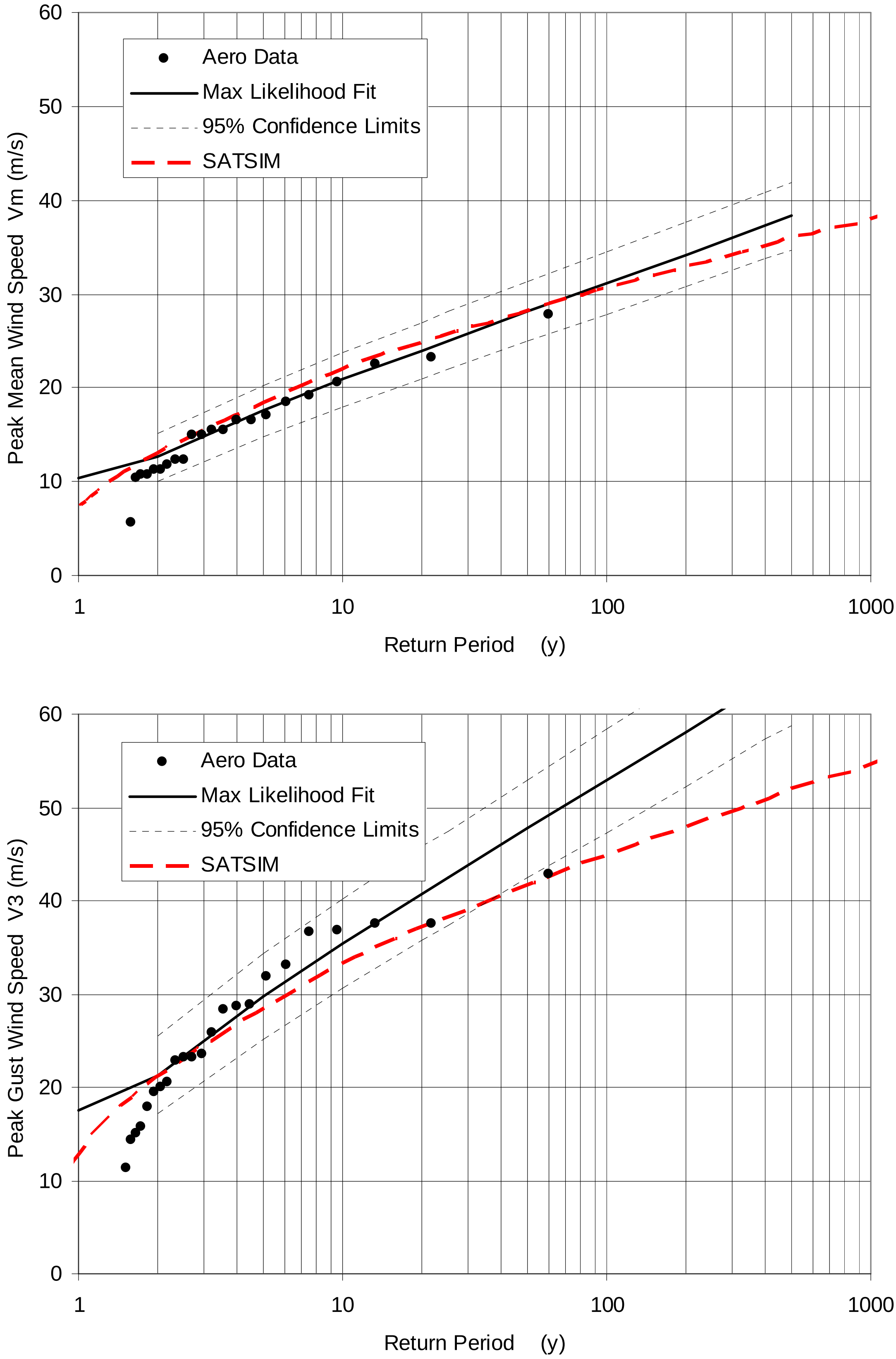


Figure 4.19 Calibration of the statistical wind speed model.

4.7 Statistical Storm Tide Model Estimates

The SATSIM model was initialised with the data descriptions of the astronomical tide, the climatology of tropical cyclones, the parametric models of wave height and period and then used to simulate 50,000 years of synthetic storm behaviour using a *Monte Carlo* sampling technique. This resulted in a total of 65,000 individual storms, each of which were processed to predict a 36 h time history of the combination of tide, inverted barometer effect (IBE), wave height and period and wave setup.

A number of breaking wave setup formula and approaches were considered. Based on the experience from Section 0, the 1% exceedance level after Gourlay (1997) was adopted for the final design estimates. This is the level likely to be equalled or exceeded 1% of the time during a period of constant wave height and period. In practical terms, this could mean about one minute in an hour. Below this level the occurrence of wave-induced superelevation can be expected to increase. As noted earlier, the Gourlay wave setup formula is sensitive to the depth of water over the reef top and hence the stage of the tide also plays a role in determining the wave setup.

The predicted design extreme water levels and significant wave heights are graphed in Figure 4.20 and summarised below in Table 4.4. At the 100 y level, the range of predicted total water levels based on the 1% wave setup criteria ranges from 2.3 m MSL in the east to about 3 m MSL in the west. At the 1000 y level, the range is 2.9 m to 3.8 m MSL. The two separate estimates for total water level and wave setup component magnitudes correspond to the region immediately east of Avarua (the higher predicted water level) and to the west of Avarua (between Avarua and Avatiu and immediately west of Avatiu). The predicted differences between these two sites can be seen to be significant, being about 0.7 m at the 100 y level and about 1m at the 1000 y level. Also shown on the figure are the astronomical tide magnitude and the inverted barometer effect as a function of return period. The predicted significant wave height is also indicated, scaled against the RHS axis, which indicates an incident H_s of 7.3 m at 100 y and 9.7 m at 1000 y.

Based on this prediction, the Cyclone *Sally* peak water level and wave height has a return period of about 200 y.

Table 4.4 Summary of predicted design parameters.

		Return Period					Units
Parameter	**Site**	**10**	**50**	**100**	**500**	**1000**	**y**
1% Wave Setup Total Water Level	East of Avarua	2.20	2.75	2.95	3.54	3.80	m MSL
	West of Avarua	1.70	2.12	2.30	2.72	2.90	m MSL
H_s	Offshore	5.1	6.7	7.3	8.8	9.6	m
T_p	Offshore	9.4	10.3	10.7	11.5	11.9	s

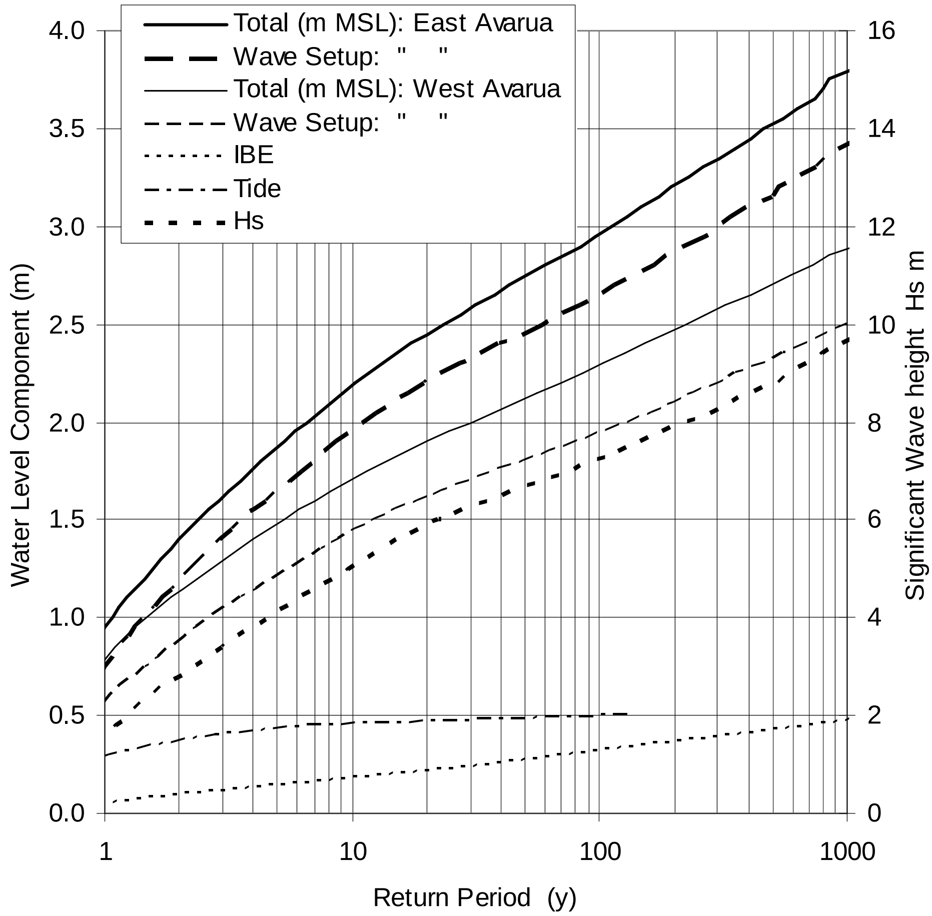


Figure 4.20 Predicted extreme water levels and wave heights.

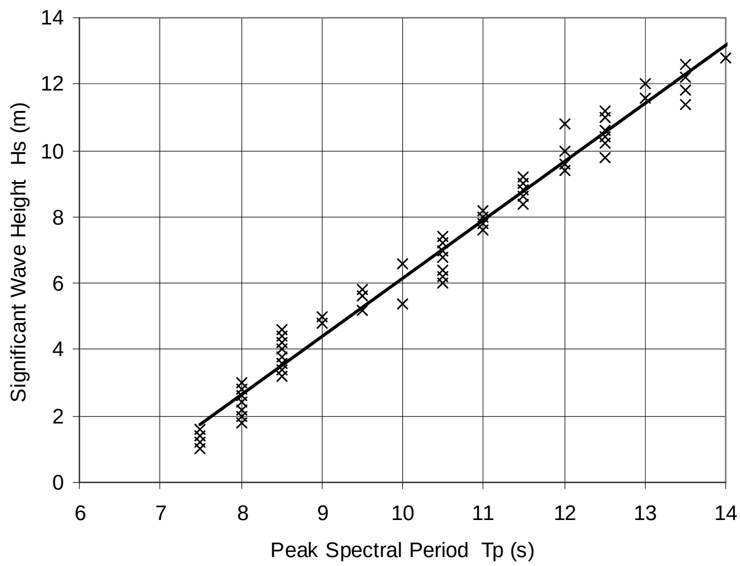


Figure 4.21 Predicted offshore wave steepness.

4.8 Limitations of the Numerical Modelling

The comprehensive numerical modelling and simulation undertaken for this study involves the use of a number of sophisticated models that have previously demonstrated their ability to economically provide reasonably accurate reconstructions of measured tropical cyclone wind, surge, wave and wave setup episodes in a variety of situations. However, the modelling is undertaken here in a necessarily abstract context whereby not all of the complexity of the real situation is represented. Examples that have been previously mentioned in the study report are:

» The wider-scale synoptic wind environment around tropical cyclones is ignored, thus lowering the accuracy of the numerical spectral and parametric wave modelling in certain situations.

» Detailed bathymetry within 1 km of the island has not been fully resolved here by the fine scale wave model (AC) and it is likely there will be some complex localised effects of diffraction/refraction that remain unrepresented and may participate to some extent in extreme inundation episodes. Also, the “sheltering” algorithm that adjusts the open ocean wave height and period estimates to apply at the specific sites of interest is based on a limited number of numerical experiments and is likely to overestimate the effects on the lee side. It is further assumed that waves always propagate perpendicular to the coastline.

» The statistical prediction of the magnitude of extreme water levels is reliant upon the accuracy of the parametric wave model in both the statistical context (validated against the available wind speed records only) and the absolute wave height and period context. The latter can only be compared either with the limited measured wave data or the full spectral wave model ADFA1 representations of the set of more intense cyclones. The comparisons have shown scatter in these comparisons that may impact the overall accuracy of the predictions but the statistical model here appears to be unbiased.

» The reef-top wave setup modelling is necessarily decoupled from the 2D wave modelling and considers an idealised 1D profile in a semi-static context. Some aspects of the complex dynamic effects likely in nature are accounted for by the adoption of estimated 1% statistical water levels but actual conditions will depend on (i) the 2D nature of the reef top, whereby channels and harbours will create localised return flow paths and dissipate and/or concentrate the wave setup effects and (ii) the dynamic wave height and period (wave groupiness) interaction with the reef top geometry, leading to possible long period “surf beat” and stochastic bore formation. Also, reef top wind stress effects are neglected and inshore wave runup is not explicitly treated.

All of the above elements can be improved through more detailed analysis, better data and specific model development and enhancement. However, it is believed that the present study approach has delivered a well-balanced and consistent outcome that captures the essential features of the problem. More sophisticated modelling could lead to increased confidence on some specific aspects if required.

5. Coastal Protection Issues

5.1 Reef Top Conditions

It can be seen from preceding sections that reef top conditions can vary over short distances and time frames.

Figure 5.1 illustrates schematically the quasi static average water levels produced by wave setup in the vicinity of an entrance.

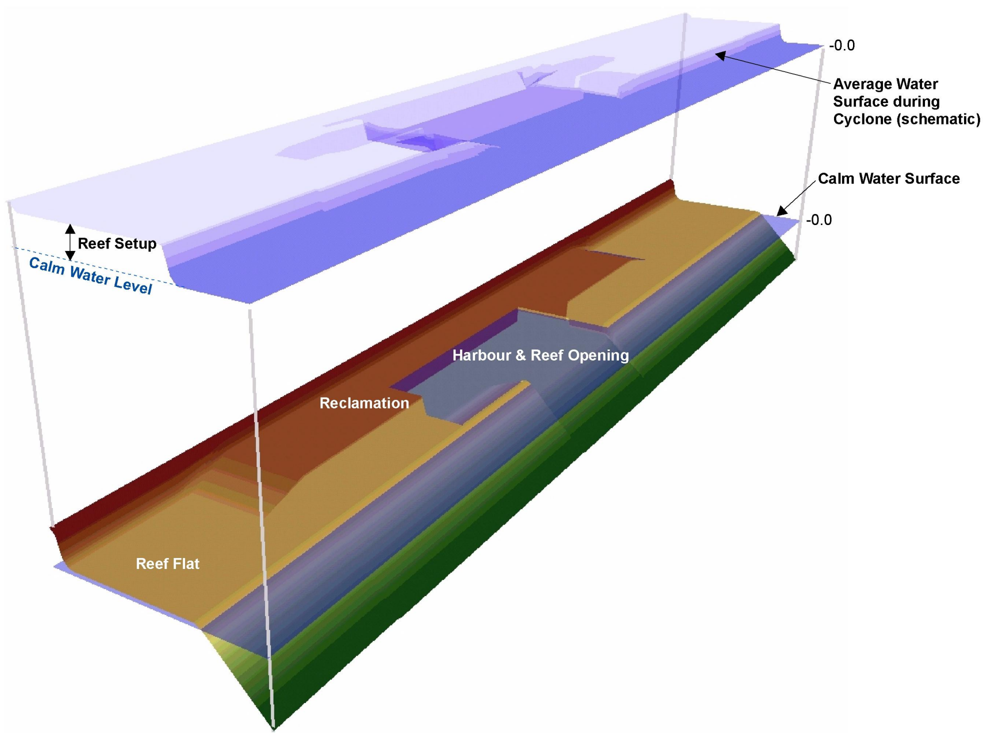


Figure 5.1 Water levels at harbour entrance

For very wide reef flats (>600 m), the quasi static conditions are representative of actual reef top water levels.

Particular processes that require attention in narrower coral reef areas are the occurrence of ‘bore waves’ or surges across the reef flat.

These waves have been observed in the field but are not adequately described theoretically. Causes are postulated as:

» Amplification of natural oscillations in reef top water due to synchronisation between a natural reef top oscillation period and the “surf beat” period.

» Amalgamation of several reformed translatory waves due to increased celerity with water depth on the reef flat.

Amplification of natural oscillations

Amplification of natural oscillations can occur in reef top water due to synchronisation between a natural reef top oscillation period and the "surf beat" period.

For reef flat widths in excess of 120m, the first mode period of oscillation is greater than 105 sec. This corresponds to the typical range of "surf beat" periods and waves may be amplified by the presence of surf beat at a complementary period.

The magnitude of amplification is impossible to quantify, as the "surf beat" characteristics of any particular wave field cannot be calculated theoretically in advance. However, for planning purposes it is possible to set an upper bound by considering that the maximum water level without run up effects would be double the reef top water level due to set up at the time of amplification. As this is at the peak of a "surf beat" event, a 1% exceedance level is considered a reasonable for this event.

Thus, during a storm it can be expected that there is a reasonably high probability that wave groups or "surf beat" will coincide with the natural oscillation period of the reef flat water and produce amplification to some level.

Amplification is reduced by reef top friction and the reflection coefficient of shoreline structures and increased by run up on shore beaches and revetments. In the absence of more detailed information it is assumed that friction and runup effects will be similar in magnitude and their impacts have been discounted.

Amalgamation of reformed translatory waves

For wider reef flats individual wave (groups) propagate at a speed proportional to the water depth on the reef flat. As the reef top waves are often translatory in nature for low relative submergence values the water behind the leading wave in a group is deeper than in front. Consequently, subsequent waves that are formed at the reef edge can travel faster than the original wave and overtake it to form a bore.

This amalgamation effect is most likely to occur on wider reef crests (say >300m). In the Avatiu - Avarua area, the reef crests are in the range of 120 – 200m and the amalgamation effect is not considered to be as likely as amplification of natural oscillations.

Adopted Amplification

The reflection coefficient of shoreline coastal structures for longer period waves is typically of the order of 70%. Thus, the theoretical maximum water level based on a reflection coefficient of 100% for a vertical wall, would be reduced from double the water depth to 1.7 times the water depth.

A "likely bore elevation" of approximately 1.7 times the water depth has been adopted. This value is selected to incorporate the effects of amalgamation of translatory waves and amplification of natural oscillations. This amplification is considered to be applicable to a calculated 1% exceedence level of reef setup.

This adopted amplification factor is consistent with the observation that all of the road between Avatiu and Avarua was submerged at some stage during cyclone *Sally*.

The calculated 1% reef top water levels at the peak of the storm in Figure 4.14 of approximately 2.6m gives a "likely bore elevation" of 4.7m above MSL and the peak 1% reef top water level of 3.0m shown in Figure 4.15 gives a “likely bore elevation” of 5.3m. Allowing for a depth of flow of 0.2 – 0.3m, and losses due to friction between the shoreline and road, would require a maximum water level due to all effects in the range 5.0 – 5.3m above MSL.

At this time the theoretical investigation and research into these phenomenon is not sufficiently advanced to include it in the modelling process or further define water levels and a sensitivity approach will be used with the “likely bore elevation” being predominantly used for calculations of stability etc and the “maximum water level” being used to check for ultimate stability.

In considering flood flow volumes capable of penetrating further inland, which would have to be maintained over a longer length of time, the calculated mean setup levels should be used.

5.2 Reef Opening Conditions

The harbours of Avatiu and Avarua are natural reef openings where fresh water outflow from the streams has inhibited coral growth. The openings are relatively deep in the entrance and become shallower inshore. The natural shape of both harbours has been modified by construction of wharves, quay walls and a harbour basin extension for fishing vessels and yachts at Avatiu.

Tide gauges to monitor ocean sea level are located in the protected water within the harbours and it is generally accepted that the harbour level is directly linked to the ocean level as the reef openings are connected to the ocean by very efficient deep channels. However, during cyclone conditions, reef flat water levels will be several metres higher than the ocean.

The harbour entrances will form the prime drainage path for the adjoining reef flats during cyclonic events so that the edges will have water pouring over them at high velocity and depths in the order of metres similar to the crest of a dam in flood. Velocities will be sufficient to carry the large quantities of rubble that are torn of the reef face by wave action.

This returning flow out of the entrance can flow at speeds in the order of 5-6 m/s (10-12kts) (Jones Pers. Obs) for large reef systems and this implies large water surface slopes. Although the velocities at Avarua and Avatiu may be lower than this due to the limited width of the adjoining reef tops and relatively inefficient drainage paths they will still be substantial. This implies a still water level in the harbour at the tide gauge sites of the order of 0.3m to 0.5m above ocean level and constantly varying.

Thus, during a cyclone causing elevated reef flat water levels, the tide gauges located within the harbour do not provide an accurate representation of either the reef flat water level or ocean water level.

Oscillations can also occur in these reef entrances and they will be less damped by friction than reef top oscillations. The period will be similar to the reef top oscillation period with variations due to the relative positioning of the harbour wall and the positioning of the effective intersection with the reef face seaward of the reef crest.

Overall the combination of lower still water levels and larger oscillations due to wave groups and water depth can produce peak water levels of similar magnitude to peak reef top water levels. Waves will reflect off the vertical inshore wall and may significantly increase overtopping volumes.

5.3 Factors affecting Reef Setup and their influence on coastal protection choices.

5.3.1 Drainage

Reef set up levels are particularly sensitive to the efficiency of drainage from the reef top. Drainage should be enhanced wherever possible and confining sections of reef flat between breakwaters, which reduce drainage, should be avoided.

A good example of the application of this concept is the removal of outer section of breakwaters at Avarua in 1993, after it was noted that their construction exacerbated buildup of water on the adjacent reef top.

Because water depths on the reef top are relatively shallow minor increases in water depth will result in significant reduction in friction. Thus relatively shallow channels can be used to improve drainage without seriously affecting reef top amenity and environmental conditions.

5.3.2 Barriers

The presence of physical barriers can confine the reef setup effects to the outer reef crest. The presence of barriers conflicts with the requirement for drainage and to be totally effective the barrier height would severely affect the visual amenity of the area by blocking all view of the horizon from the shoreline. An optimum solution is to construct barriers (breakwaters) at a lower height allowing some wave energy to pass over the top. Improvements to reef top drainage would also be required to drain the excess water off the reef top behind the barriers.

In addition, the barriers would effectively reduce the width of the reef flat and its natural period of water oscillation to a value well below the surfbeat period and amplification of reef setup effects would be less likely to occur.

Introduction of barriers also reduces the height of the waves reaching the shoreline which has significant impact on the required crest elevation and cost of construction of shoreline protection.

5.3.3 Incremental Protection

Revetments and coastal protection structures will be expensive if they are constructed to heights sufficient to prevent significant overtopping. Much of the wave action related damage during cyclone *Sally* probably resulted from bore waves and sudden massive flows of water penetrating to a considerable distance from the shoreline. If the primary line of defence is overtopped a secondary structure can divert the resulting surface flow back to a return path to the sea. This secondary structure does not have to be a formal seawall and can be incorporated into other features. An obvious opportunity for provision of a secondary structure is the coastal road. Road levels may be increased so that a wide shallow drain exists between the shoreline protection structures and the road allowing overtopping flows to be diverted to a return path to the sea.

5.4 Influence of Sea Level Rise and Climate Change

5.4.1 Sea Level Rise

Sea level rise will initially have very little effect on extreme conditions on the reef flat as water levels are governed by wave setup and effectively isolated from the ocean level.

When sea level rise reaches a point where low tides are above reef flat level, the levels of reef setup will drop and there is potential for a slight reduction in reef top water levels, but it is expected that bores will be more prevalent.

In the event of significant sea level rise, the harbour areas will be subjected to increased harbour oscillation and overtopping due to reduction in return flows from the reef flat and deeper water depths.

5.4.2 Climate Change

The main concern with climate change will be the potential for increased severity of tropical cyclones in the area of influence of Rarotonga and a potential for increased cyclone frequency.

No reliable information on these hypothetical effects is available. Conservative design is required and solutions with more gradual failure modes are favoured to reduce impact of increased storm intensity. Storm frequency changes may favour selection of more stringent design criteria to reduce the encounter probability of events requiring significant reconstruction or repairs.

6. Design Criteria for Shore Protection Structures

6.1 General Notes

The previous section shows that the design parameters for individual elements of a shore protection scheme are intimately linked to the type of protection scheme envisaged. The level of reef top drainage and protection from barriers impacts both reef top water levels and wave effects at the shoreline. Barriers will also reduce significantly any allowance required for bore waves.

Nevertheless a starting point is required, and design conditions for two scenarios will be presented. It should be noted that these are "experienced assessments" that should be verified by physical modelling because the theoretical basis of handling the complex reef top issues is not sufficiently advanced to numerically model the processes at this stage.

The scenarios are:

» Shoreline or reef top structures without barriers or drainage improvements.

» Shoreline structures with barriers at 1.5, 2.0 & 3.0m above Mean Sea Level.

It should also be kept in mind that these design conditions relate only to the local Avatiu – Avarua area and specific modelling of cyclone effects will be required for other areas on Rarotonga.

Coastal engineering formulae for coastal protection structures are generally derived on the basis of deep water at the face of the structure and oscillatory waves. This is clearly not the case for a fringing reef, with a hydraulic jump separating elevated reef setup water levels from the ocean, and a horizontal shelf extending towards shore. Therefore some prudence in the application of standard formulae is warranted and design should be carried out on a sensitivity basis. Structures that exhibit gradual or non critical failure modes should be favoured so that if design conditions are exceeded the consequences will be reduced.

A particular problem area is in the vicinity of the two harbour openings. Although harbour water levels will be close to open sea water levels, higher waves will be able to reach the landward quay wall and harbour resonance may occur. Overtopping discharges at the quay face will be massive and separate protection structures landward of the harbour working area will be required to direct the flow of this water back to a return path to the sea.

6.2 Reef Top Water Levels

Table 4.4 and Figure 4.20 outline the basic results of the cyclone modelling to produce design reef top water levels.

A 1% exceedance level has been selected as appropriate for coastal protection design to capture the general effects of wave groups.

In addition an allowance for bore waves due to either reef top oscillation or wave combination needs to be considered where there is greater than 100m of reef width in front of the structure. It is considered that the "likely" bore elevation due to reef top oscillation will correspond to 1.7 times the water depth and the "maximum" bore elevation will be 2 times the water depth.

Thus for unprotected structures the design water level should include allowance for bore waves. If a shoreline is protected by offshore barriers and adequate drainage paths are available, the adopted shoreline design water level should be the 1% exceedance level without allowance for bore waves.

6.3 Reef Top Waves

Reef top waves will be depth limited reformed waves. As such they will not be directly linked to the ocean waves, although obviously there will be a primary forcing function in their generation at the incident wave period.

Because of the depth limitations the waves will tend to behave as solitons or solitary waves and the wave velocity (celerity) will be directly related to depth. An equivalent lowerbound wave period can be derived from this celerity. Secondary wave generation effects will not be able to produce larger waves with shorter periods. Although the primary forcing function is at the incident wave period it is considered prudent to allow for double this period as an upper bound in assessing period sensitivity. Thus the most adverse wave period for the particular design application would be selected from within this band for structural design.

For overtopping and runup considerations it is considered appropriate to cap the period at the incident wave period as the effects of these can be mitigated by appropriate drainage design of the area behind the structure. It is also considered appropriate to consider bore waves separately to wave runup on top off the quasi static water level due to wave action.

As the waves are crossing a horizontal reef with a closed shore boundary they behave as solitary waves rather than oscillatory waves. In this case it is considered prudent to allow a wave height of 0.78 times the water depth (SPM 1984) rather than a wave height of 0.55 times the water depth (Nelson 1994)

Where reef top waves encounter a reef top barrier and break, transmission coefficients have been calculated in accordance with the methodology of Van der Meer (1990)

Table 6.1 outlines the design parameters appropriate for design of structures without barriers and Table 6.2 outlines the design parameters appropriate for design of structures with barriers.

Table 6.1 Design conditions for exposed structures greater than 20m from reef edge.

East of Avarua

Average Return Interval (y)	1%WL (m MSL)	Water Depth (m)	Max Bore Elevation (m MSL)	Likely Bore Elevation (m MSL)	Offshore Wave Hs (m)	Offshore Wave Tp (s)	Reeftop Wave (m)	Celerity (m/s)	Minimum Period (s)	Maximum Period (s)
10	2.20	2.60	4.8	4.0	5.1	9.4	2.0	5.1	3.2	18.8
50	2.75	3.15	5.9	5.0	6.7	10.3	2.5	5.6	3.6	20.6
100	2.95	3.35	6.3	5.3	7.3	10.7	2.6	5.7	3.7	21.4
500	3.54	3.94	7.5	6.3	8.8	11.5	3.1	6.2	4.0	23.0
1000	3.80	4.20	8.0	6.7	9.5	11.9	3.3	6.4	4.1	23.8

West of Avarua

Average Return Interval (y)	1%WL (m MSL)	Water Depth (m)	Max Bore Elevation (m MSL)	Likely Bore Elevation (m MSL)	Offshore Wave Hs (m)	Offshore Wave Tp (s)	Reeftop Wave Height (m)	Celerity (m/s)	Minimum Period (s)	Maximum Period (s)
10	1.70	2.10	3.8	3.2	5.1	9.4	1.6	4.5	2.9	18.8
50	2.12	2.52	4.6	3.9	6.7	10.3	2.0	5.0	3.2	20.6
100	2.30	2.70	5.0	4.2	7.3	10.7	2.1	5.1	3.3	21.4
500	2.72	3.12	5.8	4.9	8.8	11.5	2.4	5.5	3.5	23.0
1000	2.90	3.30	6.2	5.2	9.5	11.9	2.6	5.7	3.6	23.8

Table 6.2 Design Conditions for shore structures protected by barriers.

East of Avarua

Average Return Interval (y)	1%WL (m MSL)	Water Depth (m)	Reeftop Wave (m)	Celerity (m/s)	Minimum Period (s)	Maximum Period (s)	Transmitted Wave RL 2.5 Breakwater	Transmitted Wave RL 2.0 Breakwater	Transmitted Wave RL 1.5 Breakwater
10	2.20	2.60	2.0	5.1	3.2	18.8	1.0	1.2	1.4
50	2.75	3.15	2.5	5.6	3.6	20.6	1.4	1.6	1.8
100	2.95	3.35	2.6	5.7	3.7	21.4	1.6	1.8	2.0
500	3.54	3.94	3.1	6.2	4.0	23.0	2.1	2.3	2.4
1000	3.80	4.20	3.3	6.4	4.1	23.8	2.3	2.5	2.6

West of Avarua

Average Return Interval (y)	1%WL (m MSL)	Water Depth (m)	Reeftop Wave (m)	Celerity (m/s)	Minimum Period (s)	Maximum Period (s)	Transmitted Wave RL 2.5 Breakwater	Transmitted Wave RL 2.0 Breakwater	Transmitted Wave RL 1.5 Breakwater
10	1.70	2.10	1.6	4.5	2.9	18.8	0.6	0.8	1.0
50	2.12	2.52	2.0	5.0	3.2	20.6	0.9	1.1	1.3
100	2.30	2.70	2.1	5.1	3.3	21.4	1.1	1.3	1.5
500	2.72	3.12	2.4	5.5	3.5	23.0	1.4	1.6	1.8
1000	2.90	3.30	2.6	5.7	3.6	23.8	1.6	1.7	1.9

7. Structure Options

7.1 Revetments Without Reef Top Barriers

Revetments without reef top barriers should be designed to meet the following criteria:

- Rock or armour sizing to withstand the design reef top wave height at the worst wave period between the minimum and maximum wave period
- Crown elevation to limit wave overtopping to an acceptable level with reef top water levels at the 1% exceedance level.
- Crown elevation or other barriers (eg roads or road barriers >30m behind the crown) be above the likely bore wave elevation.
- It is considered appropriate that structurally revetments be designed to withstand a 50 yr ARI event with little or no damage and only relatively minor and repairable damage in a 100 yr ARI event.

For wave overtopping it is considered appropriate to design for overtopping that:

- poses little danger to pedestrians during a 20 yr ARI event,
- does not endanger buildings or vehicle traffic close to the wall for a 50 yr ARI event
- has a potential to damage buildings close (within 20m) to the wall during greater than 100 yr ARI events.

For this study crown elevations corresponding to a 2% exceedance level for a 50 yr ARI event will be used. This approximately corresponds to the second criteria and is calculated in accordance with Van der Meer and Stam's (1992) formulae for rock structures.

Effectively, all options will be bought to a common performance baseline for comparison. Further refinement of design will be possible in the implementation phase.

7.2 Options for Shore Revetments and Nearshore Structures

Shore revetments can be constructed of rock or concrete armour units.

Suitable concrete armour unit options are:

- Hollow blocks or Seabees
- Copeds
- "A Jacks"

The cost of concrete armour units will be a significant factor. This will favour the use of rock in all areas where practical designs can be effected using local rock.

Where design conditions are more severe or where increased energy absorption is required, use of concrete unit revetments may be cost effective.

Coped units are seen as being particularly suited to areas where deeper water is required in conjunction with energy absorption such as surrounding marinas and on the landward side of harbour basins.

In general the most economic rock wall design involves use of the steepest practical slope to reduce the overall rock volume. Rock sizing will be based on a slope of 1:1.5 for comparison purposes. During detailed design other slopes may be considered in concession to availability and cost of local rock.

Alternative revetment design sections using rock and Seabees are presented here for comparison of overall effects and cost. The other armour units (Copeds and Ajacks) are protected by patents and designs should be prepared by the licensed distributors or patent owners.

Details of typical sections examined are included in Appendix H.

Significant aspects of the rock revetment requirements for shoreline protection without barriers are the large rock size requirements and the high crown level required to provide adequate protection.

Rock sizes of 5.2t median size and a crown elevation of 8.5m above MSL would be required east of Avarua and 2.7t median size with a crown elevation of 6.8m above MSL would be required west of Avarua. It is considered that rocks of 5.2t median size would be difficult to obtain in sufficient quantities for practical construction.

For Seabee construction a seabee height of 510mm is required east of Avarua and 410mm is required west of Avarua. Individual units can be configured to weigh between 140kg and 250kg depending on handling and construction considerations. If a variable height construction is used, with 200mm height differences to increase surface roughness, crown levels would be similar or lower than rock construction. (Jones, 2003). A recurved crown wall can be used to deflect sheet flow back seawards.

7.3 Reef Top Breakwaters

To limit wave set up on the shoreline and reduce susceptibility to reef top oscillation, the presence of reef top breakwaters near the seaward edge is desirable from several perspectives:

» Low breakwaters will limit reef setup to the seaward edge of the reef and allow time for overtopping water to drain off the reef flat near shore.

» Low breakwaters near the reef edge will break down the linkage between sudden change in local set up levels due to “surf beat” and the reef top oscillation. This would reduce the occurrence of “bore waves” reaching the shoreline.

» Returning flows from between island breakwaters would induce wave breaking further offshore and reduce reef set-up.

» Low breakwaters near the reef edge can be overtopped with little risk of damage to public infrastructure and the shoreline. They will reduce energy reaching the shore by wave breaking and spread energy peaks to a more consistent level.

7.4 Options for Construction of Reef Top Breakwaters

7.4.1 Rock

Rock sizing has to be large to avoid movement in cyclone conditions. Adequate restraint must be provided to prevent rocks rolling on the reef crest.

As rocks would be subjected to bores moving at approximately 5m/s the rock can be sized using the Isbash (1935) equation for very turbulent steady flow. This gives a rock size of 7t. It is considered impractical to source and handle large quantities of rock of this size on Rarotonga.

7.4.2 Precast Concrete Armour Units.

There are many precast concrete armour units available. Most rely on absorbing wave energy within voids in the units before it contacts a rock substrate. On the reef flat at Rarotonga these would not be placed against a rock backing and would allow wave energy to pass through the breakwaters. This would have to be accounted for in the calculation of transmitted waves for shoreline revetment design.

JICA (1994) suggests 25t tetrapods as a suitable armour unit for reef top breakwater construction. Other units which may be suitable are Hanbars (no patent applicable).

7.4.3 Energy Absorbing Blocks

Several energy-absorbing units are available. Cook Island residents will be familiar with the Coped device, which has been trialled near the end of the airport. Other energy absorbing devices include “A” Jacks and Cobs.

These devices tend to have higher aspect ratios in members and may not be as robust as necessary for the reef edge environment where overtopping waves may also be carrying coral boulders ripped from the reef edge.

7.4.4 Precast Mass Concrete Blocks

In Tonga, several reef top breakwaters have been constructed using precast concrete blocks pinned to the reef top with steel pins and grouted where possible.

These are locally called ‘Eua blocks from the Nafanua Harbour Project in ‘Eua where the moulds were developed. A sketch of the typical block is shown in Appendix H.

Photographs of their use are shown in Appendix I.

The southern breakwater at Nafanua harbour experiences very similar exposure condition to that proposed for the reef top breakwaters in the Avarua – Avatiu area. In 1993 this breakwater survived direct attack from a cyclone with offshore wave heights in excess of 8m without damage. (There was significant damage to areas surrounding the harbour from wave setup induced flow back into the harbour.)

Due to the cost effective, robust and simple construction using these blocks this type of construction is recommended for the reef top breakwaters.

7.5 Options for Shore Revetments with Reef top Breakwaters

Shore revetments can be constructed of rock or concrete armour units. Suitable options are:

- Hollow blocks or Seabees
- Copeds
- "A Jacks"

The cost of concrete armour units will be a significant factor. This will favour the use of rock in all areas where practical designs can be affected using local rock. Where design conditions are more severe or where increased energy absorption is required, concrete unit revetments will be an advantage.

Coped units are seen as being particularly suited to areas where deeper water is required in conjunction with energy absorption such as surrounding marinas and on the landward side of harbour basins where the wave climate will not be modified by the reef top breakwaters.

In general the most economic rock wall design involves use of the steepest practical slope to reduce the overall rock volume. Rock sizing will be based on a slope of 1:1.5 for comparison purposes. During detail design other slopes may be considered to optimise availability and cost of local rock. With the reduced design wave heights due to the reef top breakwaters rock revetments will in general be suitable. In fact the revetments already constructed east of Avatiu will be adequate structurally although it may be necessary to review the crest elevation in some locations.

Alternative revetment design sections using Rock and Seabees are presented here for comparison of overall effects and cost. The other armour units (Copeds and Ajacks) are protected by patents and designs should be prepared by the licensed distributors or patent owners.

Details of Typical sections examined are included in Appendix H.

Significant aspects of the rock revetment requirements for shoreline protection without barriers are the large rock size requirements and the high crown level required to provide adequate overtopping protection.

With a reef top breakwater elevation of 2.5m above MSL Rock sizes of 1t median size and a crown elevation of 6.1m above MSL would be required east of Avarua. West of Avarua the reef top breakwater height can be reduced to 1.5m above MSL and 0.8t median size rock used in shore revetments with a crown elevation of 5.2m above MSL.

For Seabee construction a seabee height of 290 mm is required east of Avarua and this would also be used west of Avarua. Individual units can be configured to weigh approximately 27kg and allow manual handling and construction. If a variable height construction is used with 150mm height differences crown levels would be similar or lower than rock construction. (Jones, 2003). A recurved crown wall can be used to deflect sheet flow back seawards.

7.6 Avatiu and Avarua Harbour Reef Openings

The two reef openings present particular problems related to:

- The presence of the coastal streams and the flow path they present

- Requirement for the reef openings to be maintained as a primary reef top flow drainage path
- Commercial usage of the area
- High wave overtopping rates at vertical landward harbour walls

Essentially these areas have to be isolated as a flow path inland to reduce the volume of water crossing the road to the commercial and residential areas. In recognition of the operational constraints the following structures are recommended to achieve this:

- A barrier between the road and the harbour areas with a crown level near the "Likely Bore Elevation" (say RL +4.6) (refer Figure H6, Appendix H)
- Flood gates across the streams with a crown at road level and gate openings sized to meet local flooding requirements (refer Figure H8, Appendix H)
- Provision for energy absorption at the landward quay walls of the harbours. (refer Figure H1, Appendix H)
- Reduction and dissipation of wave energy entering the opening by installation of vertical ends on entrance breakwaters and enlarging the basin area inshore (similar to the fishing boat / yacht basin in Avatiu) (refer Figure H2, Appendix H)

Implementation of these has to fit in with the commercial and social usage of these areas. It is envisaged that the barrier would take the form of a concrete traffic barrier on the seaward side of the road. This would preserve a maximum area for port operations and other uses on the harbour side. Access to the harbour area would be by limited openings in the barriers of sufficient width for large vehicles. It is preferable that portable barriers be provided to close the gap in the event of a cyclone, however, this is not critical as the volume of water that can flow through the opening during a one to two minute bore event will be limited and it can dissipate into the adjacent areas that are sheltered by the barrier. (refer Figure H6, Appendix H)

Similarly floodgates would not have to be fitted with rubber seals. Consideration would have to be given to the rare operation of the flood gates in their design and low maintenance concepts incorporated. Theses are expected to include:

- Stainless steel construction of gates
- Concrete discharge apron to prevent scour and reduce vegetation growth in the opening range of the gate. (refer Figure H8, Appendix H)
- Upstream gross pollutant trapping by an enlarged basin.

It is envisaged that the barrier and floodgates would be incorporated into the seaward side of the road crossings. The pollutant trapping and screening would be on the landward side of the road crossing.

Energy absorption at the landward quay wall edge of the harbour basin will reduce reflection and overtopping.

Limiting energy entering the harbour by vertical ends on breakwaters has proved beneficial at Pangai Harbour on Lifuka in the H'apai group in Tonga. This is an expensive exercise and would logically be combined with any port upgrade proposals that require establishment of specialised construction equipment. Previous breakwaters of steel sheet piling at Avatiu were not successful in the long term and massive concrete construction is preferred.

At Avarua, limiting wave energy entering the harbour is favoured to reduce overtopping and flooding problems on this low area of coast. The reef top breakwater would extend into deeper water, requiring preparation of a suitable foundation. (refer Figure H2, Appendix H)

In consideration of the large volume of water that will be flowing into the fishing boat / yacht basin in Avatiu consideration should be given to a spillway construction and a debris trap west of this to catch some of the debris carried by the water. The spillway should be at reef level and constructed of concrete blocks imbedded into the reef flat at the edge of the basin. (refer Figure H1, Appendix H)

7.7 Design Verification

The detailed design of structures on the reef top cannot currently be carried out by theoretical or numerical modelling approaches. To assess the effectiveness of reef top structures and evaluate overtopping flows, it will be necessary to carry out 3D physical model testing of the area. It is preferable that this be carried out at a large scale of approx. 1:10 – 1:30 with random wave generation. Specific sensitivity of surf beat induced phenomena should be investigated by using varied spectra and control signals for determining the final layouts.

At present there is insufficient reef top and reef crest survey data to construct such a model. Multi-beam survey of deeper water has been carried out by SOPAC and will assist in providing data for the reef face. The area that is normally in the breaker zone is particularly difficult to survey. For this study snorkelling observations from seaward showed that in general the reef face is a gentle slope from the reef crest to approximately 10m below MSL. The exception to this is the area immediately east of the study area where there is a steep drop-off to approximately 3m below MSL.

A suitably constructed physical model will be able to reproduce the flow patterns and set up that will most likely exist in prototype and allow evaluation of key areas, such as:

- Susceptibility to reef top water oscillation
- Formation of bore waves
- Influence of reef top drainage on landward margin water levels
- Influence of reef top structures on reef set-up and drainage
- Influence of reverse flow between structures on reef set-up

8. Cost Effectiveness

Construction costs of various elements have been examined to determine the cost effective solutions.

Basic revetment and breakwater unit costs used in this evaluation were estimated based on comparable costs in other locations with a knowledge of the plant resources required for this work. They do not include Contractor's establishment costs as this will not necessarily vary in proportion to the scope of works. Thus the rates can be used to compare structure types but cannot be directly extrapolated to prepare an overall budget.

Table 8.1 and Table 8.2 set out the costs for East of Avarua and West of Avarua based on rock construction.

Comparison costs were also derived for Seabee units and these were found to be similar to rock construction generally. They may have an advantage for the areas East of Avarua if no reef top breakwaters are built as it will be difficult to source sufficient quantities of large rock economically.

Table 8.1 Construction costs for options – East of Avarua

Case	**W/O Reef Breakwaters 1:1.5 Slope**	**W/O Reef Breakwaters 1:3 Slope**	**With Reef Breakwaters @ RL 2.5**	**With Reef Breakwaters @ RL 2.0**	**With Reef Breakwaters @ RL 1.5**
Rock Size (t)	5.2	1.8	1.0	1.4	2.0
Crown RL @ 2% Runup (mMSL)	8.5	7.0	6.1	6.5	7.0
Revetment Cost per m (NZ$)	8,400	9,000	4,000	4,620	5,300
Breakwater Cost per m (NZ$)	-	-	2,850	2,200	1,700
Total Cost per m (NZ$)	8,400	9,000	6,850	6,820	7,000

Table 8.2 Construction costs for options – West of Avarua

Case	**W/O Reef Breakwaters 1:1.5 Slope**	**W/O Reef Breakwaters 1:3 Slope**	**With Reef Breakwaters @ RL 2.5**	**With Reef Breakwaters @ RL 2.0**	**With Reef Breakwaters @ RL 1.5**
Rock Size (t)	2.7	0.9	0.3	0.5	0.8
Crown RL @ 2% Runup (mMSL)	6.8	5.5	4.3	4.7	5.2
Revetment Cost per m (NZ$)	6,100	4,300	2,500	3,000	3,600
Breakwater Cost per m (NZ$)	-	-	2,850	2,200	1,700
Total Cost per m (NZ$)	6,100	4,300	5,350	5,200	5,300

Construction costs for the reef top breakwaters was based on a breakwater length of 80m with a 20m gap between breakwaters and a shallow channel excavated between the breakwaters for drainage.

The area between Avarua and Avatiu is already protected with a seawall that meets the requirements with reef top breakwaters at RL +2.5, thus only reef top breakwaters are the preferred option for this area rather than reconstructing the wall to a higher standard.

The area West of Avatiu is not currently protected by a seawall and the cheapest solution may be to construct a new seawall without reef top breakwaters. The crown level would be higher than existing walls and this would have to be taken into consideration from a social amenity perspective as it would block the view of the ocean from ground level/

The area east of Avarua has an existing seawall, however it is doubtful that it is of sufficient standard even with reef top breakwaters at RL + 2.5. This area is also under consideration for a road realignment project that would involve some reclamation. Reef top breakwaters at RL + 2.5 offer the most cost effective solution for new construction.

At the actual harbour areas in both Avatiu and Avarua logistical constraints preclude construction of rock revetments and concrete barrier walls should be constructed as outlined in section 7.6. These would be designed to resist a water head of approximately 2m above the barrier crest. L section reinforced concrete retaining walls with toes seawards are probably the most economical. These could be incorporated into traffic and port security barriers.

9. Social and Environmental Issues

9.1 Background

The Cook Islands is a self-governing territory in free association with New Zealand. All Cook Islanders are eligible for New Zealand passports. Rarotonga has a total land area of 5,719 hectares and is the largest, most populous island and the economic centre of the Cook Islands group.

Since 1926, the population of Rarotonga has been increasing at twice the rate of the Cook Islands in total. The 2001 census has shown that more than 60 percent of the resident population live on Rarotonga.

Rarotonga is made up of three main districts or Vaka's. They are Te Au O Tonga, which is the main centre and business district on the northern side of the island; Puaikura, located on the western side, and Takitumu on the south eastern side.

Cook Islanders have long lived in close harmony with their environment. Careful husbandry of resources and clean household environments are fundamental to survival. Traditional systems for the conservation of resources (Raui) has been introduced in the lagoon areas of Rarotonga and is supported by the entire community.

9.2 Project

The project is to examine coastal protection options for the main commercial area of Rarotonga encompassing Avaitu and Avarua port areas in the Te Au O Tonga district of Rarotinga. Part XII of the Cook Islands Act 1915 provides for customary land. The limits of customary native land, which relates to land on Rarotonga, lies at the high water mark (i.e., the line of medium high tide between the spring and neap tides). Any land below the high water mark is Crown land (s417).

Therefore, although the project will be constructed on Crown land, consultation with traditional as well as elected leaders of the district and Vaka will be necessary.

The project attempts to demonstrate how technical and economic assessments can provide the basis for low risk development of the foreshore, including reducing risks to moored vessels, coastal infrastructure and inhabitants.

9.3 Social Characteristics

There are three important ideals for Cook Islanders. They are land, culture and family. Each of the Vaka's (districts) practices their own form of community control and social organization and there is limited uniformity in the application of customary practices. Despite the introduction of a monetary system and high mobility of Cook Islanders between Rarotonga and overseas (such as New Zealand and Australia), the basic fabric of family and community is treasured and remains an influencing force for Cook Islanders.

The family continues to provide a social safety net for most Cook Islanders despite recent changes, including economic reforms and community and Government recognize that the family must remain the foundation in the pursuit of social and economic development. The family remains the basic economic and social unit in the Cook Islands economy.

Land ownership and usage is related to family and extended family units. Most land on Rarotonga is in the hands of the people. Every Cook Islander has a right to inherit from their parents a share of native land. The sale of land is prohibited and the only permitted transfers other than by inheritance are by lease or occupation right. The complex tenure arrangements make it difficult to impose comprehensive land use controls in a western sense. This underscores the need and importance of consultation and cooperation with traditional landowners and leaders in seeking to purchase or change existing patterns of land use.

9.4 Economic Characteristics

The total number of households on Rarotonga in 2001 stood at over 3,000. Compared to other islands, Rarotonga has the smallest household size of approx 4 persons per household.

Since 1996 there has been more of the population engaged in commercial agricultural activity as well as subsistence agricultural activity. This is consistent with the increase in tourism, accommodation and larger number of restaurants requiring local produce. Travel around the island and a visit to the local Punanga Nui market is indicative of the increase in local produce available for sale.

The total labour force (economically active population) was over 4,000 in 2001, and trends show a decline in public service employment since 1996. Rarotonga, as the commercial centre of the Cook Islands, contains a larger proportion of the higher income groups. Less than 100 people were reported as earning no income with less than 1,000 people earning less than the minimum wage.

The majority of households on Rarotonga were reported as having water piped to inside their dwelling, while less than 20% had water piped to outside their dwelling. This means that very few households do not have access to water

9.5 Assessing Target Populations Needs and Demands For This Project

No qualitative or statistical surveys were undertaken for this project, therefore experience and advice from local experts was used to obtain information.

The Avatiu port in Avarua is the only major port of entry for the Cook Islands providing for ships to unload freight, fuel and gas. It also provides for smaller inter island shipping services as well as visiting yachts. More recently the port has become very important to the fishing industry, servicing an increasing number of fishing vessels to meet the growing demand for Cook Islands fish exports.

There are also increasing emphasis on tourist activity with fishing charters and diving operations requiring improved port facilities and regular visits (approximately every 10 days) by international cruise liners. Local shop owners typically set up small markets during the visits of these ships to cater for the tourist trade.

In addition, the local population, including large numbers of children, use the port for recreational activities including swimming and fishing.

The port is an important economic asset for the Cook Islands and improvements to the port including an improved sea wall will benefit the immediate population of Rarotonga as well as the entire Cook Islands.

9.6 Community Concerns

9.6.1 General Concerns

It is important for communities and residents to agree on technical initiatives if they are to succeed.

There is considerable interest and knowledge about the environment in the community and Cook Islanders are well aware of the delicate ecological balance of their islands and the vital importance of a continuing and flourishing tourist industry to the economy and to their own livelihoods. There is an acknowledgment of the trade-off between the need to protect the central business district from storm damage and the visual pollution and possible destruction of the village atmosphere that is attractive to tourists.

Communities and residents have raised three concerns:

» the environmental impact in visual pollution terms (Visual Amenity)

» the perceived negative side effects of flood control gates (especially if anticipated rainfall expected with a ‘super cyclone’ materializes)

» the loss of recreational and food source areas, albeit minimal.

9.6.2 Specific Concerns

These concerns were raised and have been addressed below.

9.6.2.1 Visual Amenity

Although Rarotonga has to survive severe cyclone attack, the day to day economy revolves around tourism and recreational / food gathering activities centred on the reef flats. The overall visual amenity of any protection option must complement these activities or have minimal impact.

For this study, it is taken that structures that severely impede a view of the horizon for a person walking on shore, will be unacceptable. Where possible, people should also be able to view the reef edge and shore waves.

With as assumed height of eye of 1.5m above ground level, this places a limit of between 3.5m and 6.0m for the top elevation of shore structures, depending on the height of the land. The value of 3.5m has serious implications in that it is not sufficient to provide a reasonable level of protection against overtopping bores. These conditions only occur in a relatively small area near Avarua harbour. It is understood that road reconstruction and alignment works are proposed for this area and it would be possible to raise the level of the road to accommodate the requirements for visual amenity.

Where reef front structures are proposed, it is desirable that they be at a lower level to allow a general view of the ocean and discontinuous to allow a partial view of the reef edge waves. For this study, it is assumed that a maximum acceptable level will be 2.5m, which will allow a view of the distant ocean and only the tops of larger shore waves at lower water levels. At higher water levels, the waves would overtop the reef edge structures, creating an alternative view of wave conditions.

The construction of shallow depressions between reef top barriers to enhance drainage in cyclone conditions will increase the extent of visible shallow tidal pools while maintaining an unchanged reef edge. As the construction process would expose raw coral, it would be appropriate to seed these depressions with coral pieces and whole corals to encourage rapid colonization and coverage of the raw surface.

9.6.2.2 Flood Gates

Flood gates are flap valves where a flat plate is hinged at the top allowing it to swing up and permit flow in one direction only. The purpose of flood gates is to prevent flooding of low lying areas by surges moving up the streams. In proposed application, they would allow flow towards the ocean.

As such they are a secondary barrier and can be omitted if the risk of saltwater flooding is considered acceptable, or detailed physical modelling indicates that the flows are relatively minor.

Flood gates have to be sized to handle the expected flows without exacerbating upstream conditions. It is not part of the scope of this study to examine flooding, however Hay et al (2005) have identified that improvements to the streams and the road crossings are desirable to reduce flooding in Avarua. The gates would be relatively light weight stainless steel construction and should not impede flood flows in a correctly sized road culvert. The potential presence of the gates should be taken into account in any proposed reconstruction of the road crossings.

9.6.2.3 Loss of Recreational and Food Source Areas

Construction of shore revetments without reeftop breakwaters avoids any negative impacts on the recreational and food gathering amenity of the reef flat. This solution is generally not acceptable form considerations of availability of construction materials, cost and visual amenity.

Construction of a reef edge barrier system will impact on the amenity of the reef flat in the following ways:

- » The barriers will reduce wave energy reaching the shore in normal conditions, allowing survival of slightly less robust species.
- » The sheltered area behind the barriers may allow coral rubble to accumulate as demonstrated by the COPED Breakwater near the airport.
- » Construction of drainage depressions between the breakwaters will introduce more intertidal pools into the reef flat providing a site for fauna and flora that inhabit the pools.

On balance, it is considered that in the longer term, the increase in availability of tidal pools will improve recreational and food gathering opportunities on the reef flat during calm weather.

Short term impacts will have to be considered carefully and minimised. This will mainly be the loss of areas of productive reef and replacement by raw coral edges. Another issue to be addressed is the possible temporary increase in ciguatoxin levels reported in some areas where large raw coral surfaces have been exposed. It is understood that where this is an issue, the raw surfaces have been rapidly colonised by marine flora and as a consequence, the effect is relatively short lived.

It is recommended that the design of coastal protection works is tested in a physical model and the modifications to reef top flow patterns established in detail. When this is carried out, it would be appropriate to have the environmental issues examined by an ecologist with experience in coral reef ecology and discussed in community workshops. This will ensure the technical solution to be implemented is appropriate and ensure protection for infrastructure and inhabitants alike.

9.7 Sediment Processes

As set out in previous sections, and above, there will be modification to reef top hydraulics by the proposed measures. This implies changes to sediment transport on the reef top in both ambient conditions and cyclones.

9.7.1 Ambient Conditions

Provision of reef top barriers where required together with associated depressions to improve drainage can have the following effects.

- » General water circulation on the inshore section of the reef will be less vigorous and there may be a slight accumulation of sand and rubble on the shore and on the rear edge of the barriers.
- » Flow paths will be modified and the depressions will become the main exit paths from the reef flat. To minimise impact, it is desirable that the depressions are not continuous through the outer reef edge and do not lower water levels in existing reef pools.

9.7.2 Cyclone Conditions

Sediment processes in cyclones are dramatic as evidenced by the rubble accumulations observed in cyclones. Near the reef edge “sediment” can include boulders weighing several tonnes that can damage non-robust structures.

Reef setup induced flows during a cyclone are capable of transporting large rubble. This is often deposited in shingle banks inshore or carried into outlet channels (including harbour basins).

It is considered that during a cyclone, flows and turbulence will generally prevent a buildup of rubble in front of barriers, although it may be stripped from the outer reef edge. Also, flow velocities in the depressions would be high enough to transport rubble back to the sea.

As flows on the landward side of the reef flat will be reduced, rubble accumulations can be expected along the shoreline and possibly behind the barriers.

It is anticipated that provision of reef edge barriers, together with protection of the edges of harbour basins, will reduce sedimentation in the harbours.

9.8 Summary of Social and Environmental Issues

Table 9.1, below, outlines issues associated with the various structure options examined for the reef flats and Table 9.2 outlines issues associated with the harbour area.

Table 9.1 Summary table of reef flat aspects

Wall Option	Crown Level	Visual	Environmental
East of Avarua			
Revetment at land margin of reef flat		Blocks view of ocean from existing ground level	No change to existing reef flat conditions
– 1:1.5 slope	8.5 (4m above max ground level)		
– 1:3 slope	7.0 (2.5m above max ground level)		
Reef top Breakwater @ RL +2.5	6.1 Shore 2.5 Offshore	Marginal view of the ocean at highest points	Modified reef top conditions – More reef pools – Initial bare coral areas to be re-colonised by corals and algae – Minor risk of ciguatoxin during construction
Reef top Breakwater @ RL +2.0	6.5 Shore 2.0 Offshore	View of ocean obstructed from shore by shore wall	
Reef top Breakwater @ RL +1.5	7.0 Shore 1.5 Offshore	View obstructed by shore wall	
Avarua / Avatiu			
Revetment at land margin of reef flat			No change to existing reef flat conditions
– 1:1.5 slope	6.8 (2.2m above max ground level)	Blocks view from existing ground level	
– 1:3 slope	5.5 (2.2m above max ground level)	Marginal view of ocean from existing ground level, acceptable in some areas	
Reef top Breakwater @ RL +2.5	4.3 Shore 2.5 Offshore	Acceptable view of ocean from areas higher than RL +2.8 (most of section)	Modified reef top conditions – More reef pools – Initial bare coral areas to be re-colonised by corals and algae – Minor risk of ciguatoxin during construction
Reef top Breakwater @ RL +2.0	4.7 Shore 2.0 Offshore	Acceptable view of ocean from areas higher than RL +3.2	
Reef top Breakwater @ RL +1.5	5.2 Shore 1.5 Offshore	Acceptable view of ocean from areas higher than RL +3.7 (approx 50% of area)	
West of Avatiu			
Revetment at land margin of reef flat			No change to existing reef flat conditions
– 1:1.5 slope	6.8 (2m above max ground level)	Blocks view of ocean from existing ground level	
– 1:3 slope	5.5 (2m above max ground level)	Marginal view of ocean from existing ground level	
Reef top Breakwater @ RL +2.5	4.3 Shore 2.5 Offshore	Acceptable view	Modified reef top conditions – More reef pools – Initial bare coral areas to be re-colonised by corals and algae – Minor risk of ciguatoxin during construction
Reef top Breakwater @ RL +2.0	4.7 Shore 2.0 Offshore	Acceptable view	
Reef top Breakwater @ RL +1.5	5.2 Shore 1.5 Offshore	Acceptable view due to higher land levels	

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Table 9.2 Summary table of Harbour area aspects

Aspect	Comments
Water Levels	Water levels elevated above ocean but lower than reef flats.
Entrance Channel	Entrance channel is a major drainage path and prone to sedimentation from material carried off reef flat.
Harbour Oscillation	Deeper channel through reef promotes oscillation and this is exacerbated by highly reflective vertical inshore harbour wall. Energy absorbing wall required to reduce level of oscillation and overtopping.
Overtopping	Vertical harbour wall (necessary for operation) increases overtopping levels and flows. Overtopping flows can be impeded by wall structure behind port area and seaward of main road to reduce risk of damage to buildings. Buildings in port area to be preferably elevated and robust to reduce damage from flows.
Public Amenity	Provision of flow barriers behind harbour areas will reduce view of ocean and impede pedestrian access. Such barriers may be required for Port security considerations.
Streams	Streams have potential to convey large volumes of seawater into low lying areas and floodgates to prevent this are desirable. Community concern over possible impact on flood levels requires a transparent design and construction process.

10. Conclusions and Recommendations

10.1 Determination of Design Criteria

Rigorous modelling procedures have been applied to the modelling of cyclone waves and water levels. There are some limitations due to the length of the available reliable data set on cyclones (34 years), however, results are considered reliable for wave conditions outside the reef edge.

Current research on 2-D reef setup has been used to provide conservative estimates of reef top conditions. As this work is still at a basic stage, further improvement may be possible and conservative design procedures are recommended, including physical modelling.

10.2 Physical Model Testing

It is important that any scheme for coastal protection in Rarotonga be tested by physical modelling with specific testing programs to produce the overtopping and surge conditions observed in cyclones.

This cannot be emphasised strongly enough as the coastal processes on coral fringing reefs are poorly researched and documented.

10.3 Preferred Coastal Protection Options

10.3.1 East of Avarua

Although some cost savings may be possible with a single seawall on the shore alignment, this solution is not preferred on social and visual amenity basis.

It is understood that the road is to be re-aligned seaward of buildings in the area to alleviate traffic congestion.

Reef top barriers, in combination with a lower shore wall, will preserve sight lines to the sea from the road.

10.3.2 Avarua Harbour

Avarua Harbour is the source of major overtopping flows into the low area between streams near Banana Court.

Reduction of wave energy entering the harbour can be achieved by extending reef top barriers into the side of the channel on the west side and as close as practicable on the east side.

Reduction in harbour oscillation and overtopping can be achieved by energy absorbing construction at the existing vertical southern harbour wall.

Barriers landward of the quay wall are proposed to limit overtopping flows across the road.

Floodgates on coastal streams are proposed to limit seawater intrusion upstream. These should be allowed for in upgrading of road crossings to improve flooding characteristics.

10.3.3 Avarua / Avatiu Area

This area is already protected by a seawall that is adequate if reef top barriers are introduced. Reef top barriers at RL +2.5 are recommended.

10.3.4 Avatiu Harbour

This is the main commercial harbour for the Cook Islands.

Harbour oscillations should be reduced by provision of energy absorbing facing to existing vertical quay walls.

Penetration of overtopping water inland should be reduced by construction of barriers to the south of the harbour along the road alignment. These can be incorporated into port security arrangements.

If physical modelling indicates that substantial flow up the stream can occur, floodgates should be installed.

10.3.5 West of Avatiu

The most effective coastal protection in this area from amenity and cost perspectives is a combination of reef top barriers at RL +2.5 and a shore revetment using rock armour.

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Appendix A

Summary of Historical Tropical Cyclones within 500km of Rarotonga

1970 to 2003/2004

Table A.1 - Lifetime Summary of Tropical Cyclones: 1969/1970 to 2002/2003 within 500km Rarotonga

		Start				Finish					At Closest Approach						
No.	Name	Date	Time	Lat	Long	Date	Time	Lat	Long	p0	Date	Time	Dist	Bear	Vfm	Theta	Xprox
			hhmm	deg	deg		hhmm	deg	deg	hPa		hhmm	km	deg	m/s	deg	km
196902	DOLLY	11-Feb-70	1200	-14.9	162.4	25-Feb-70	0	-30.0	-139.5	966	22-Feb-70	900	128	219	6.9	126	128
197110	AGATHA	22-Mar-72	0	-15.2	-160.7	28-Mar-72	0	-26.0	-158.0	980	24-Mar-72	1200	1	267	3.0	198	1
197204	FELICITY	14-Jan-73	0	-16.0	-164.0	18-Jan-73	0	-27.0	-156.0	990	16-Jan-73	1830	251	212	6.9	124	251
197206	GLENDA	31-Jan-73	0	-18.9	-164.9	01-Feb-73	0	-22.8	-162.0	990	31-Jan-73	2130	282	245	7.4	155	282
197311	TINA	24-Apr-74	0	-12.9	168.0	28-Apr-74	0	-24.0	-159.0	997	27-Apr-74	2130	271	198	19.6	107	271
197601	KIM	09-Dec-76	1200	-12.5	179.0	13-Dec-76	0	-24.0	-149.0	980	12-Dec-76	0	11	269	12.8	113	5
197602	LAURIE	11-Dec-76	0	-11.0	-178.0	12-Dec-76	1200	-19.0	-159.0	996	12-Dec-76	1130	256	12	15.1	104	-256
197607	C19770219	20-Feb-77	0	-18.5	-166.5	24-Feb-77	1200	-36.5	-140.5	990	21-Feb-77	500	189	240	7.3	150	189
197706	CHARLES	14-Feb-78	1200	-12.8	-179.0	28-Feb-78	1200	-33.0	-151.0	980	26-Feb-78	600	128	61	5.0	151	-128
197901	OFA_1979	09-Dec-79	1200	-12.0	170.0	15-Dec-79	0	-21.0	-156.0	992	13-Dec-79	430	105	184	7.2	98	105
198006	CYC198103_1980	16-Feb-81	0	-18.0	-179.0	21-Feb-81	0	-36.0	-157.0	985	19-Feb-81	1130	107	224	8.9	135	107
198007	DAMAN_1980	20-Feb-81	0	-14.5	-179.0	24-Feb-81	1200	-36.0	-166.0	984	22-Feb-81	1030	62	28	11.4	122	-62
198010	ESAU_1980	01-Mar-81	1200	-11.0	178.0	05-Mar-81	1200	-29.0	-159.0	995	04-Mar-81	2130	297	241	7.3	150	297
198011	TAHMAR_1980	08-Mar-81	1200	-19.0	-161.0	13-Mar-81	0	-27.7	-139.2	997	08-Mar-81	2330	155	28	5.5	117	-155
198012	FRAN_1980	17-Mar-81	1200	-13.3	-161.8	24-Mar-81	0	-30.0	-140.2	985	21-Mar-81	1200	479	57	4.4	138	-478
198203	LISA_1982	10-Dec-82	1200	-8.5	-159.2	18-Dec-82	1200	-33.0	-143.0	980	14-Dec-82	0	447	84	2.7	164	-384

Table A.1 - Lifetime Summary of Tropical Cyclones: 1969/1970 to 2002/2003 within 500km Rarotonga

198403	DRENA_1984	09-Jan-85	0	-11.8	-178.0	16-Jan-85	0	-20.0	-163.0	997	16-Jan-85	0	360	291	8.3	104	-45
198407	FREDA_1984	25-Jan-85	1200	-16.0	-159.0	30-Jan-85	1200	-36.5	-160.0	982	26-Jan-85	1430	257	318	6.5	228	257
198504	IMA_1985	05-Feb-86	1200	-17.2	-171.9	16-Feb-86	0	-41.0	-134.0	965	07-Feb-86	1200	275	15	8.0	101	-275
198604	SALLY_1986	26-Dec-86	600	-13.0	-165.5	05-Jan-87	1800	-34.0	-141.0	967	02-Jan-87	1930	2	23	4.3	146	-2
198605	TUSI_1986	15-Jan-87	1200	-9.2	-173.0	25-Jan-87	600	-30.5	-170.0	987	20-Jan-87	2230	315	218	2.4	129	315
198608	WINI_1986	27-Feb-87	1200	-12.5	175.4	07-Mar-87	1200	-31.0	-143.9	965	04-Mar-87	1330	448	229	6.0	140	447
198609	NONAME_1986	28-Feb-87	1200	-16.6	-161.4	03-Mar-87	0	-32.5	-140.0	985	01-Mar-87	900	88	67	10.4	152	-87
198611	ZUMAN_1986	22-Apr-87	0	-11.0	-176.0	26-Apr-87	0	-23.8	-160.8	997	26-Apr-87	0	306	199	12.2	112	306
198705	CILLA_1987	26-Feb-88	1800	-18.5	-162.0	08-Mar-88	0	-31.7	-154.4	990	29-Feb-88	600	294	34	6.6	117	-294
198808	JUDY_1988	22-Feb-89	0	-14.4	-150.5	28-Feb-89	0	-29.2	-160.3	972	26-Feb-89	2100	165	159	2.8	248	-165
198904	PENI_1989	12-Feb-90	1200	-10.2	-161.0	18-Feb-90	0	-33.4	-142.5	970	15-Feb-90	1730	183	53	7.1	142	-183
199001	SINA_1990	24-Nov-90	1200	-10.0	174.5	04-Dec-90	1800	-53.0	-124.0	985	30-Nov-90	1700	305	225	16.7	134	305
199102	VAL_1991	04-Dec-91	1200	-9.5	-179.5	17-Dec-91	0	-50.0	-163.0	971	12-Dec-91	730	459	248	9.3	159	459
199110	GENE_1991	15-Mar-92	1200	-14.5	-166.0	19-Mar-92	1800	-31.0	-158.5	985	17-Mar-92	1100	151	245	7.9	155	151
199202	KINA	26-Dec-92	600	-9.0	169.0	06-Jan-93	0	-25.0	-160.0	996	05-Jan-93	2200	416	192	9.6	102	416
199203	NINA	29-Dec-92	600	-14.6	150.0	05-Jan-93	600	-25.0	-162.3	997	05-Jan-93	130	417	241	15.4	154	416
199207	NISHA	12-Feb-93	0	-12.5	-166.0	16-Feb-93	1800	-31.0	-146.0	980	14-Feb-93	330	101	29	6.8	121	-101
199402	WILLIAM	30-Dec-94	1800	-12.0	-163.1	05-Jan-95	1200	-40.0	-119.5	983	02-Jan-95	800	211	34	6.4	123	-211
199608	NONAME	23-Feb-97	0	-18.0	-159.2	02-Mar-97	0	-48.5	-164.5	1002	23-Feb-97	2230	311	299	4.8	210	311
199615	KELI	07-Jun-97	0	-7.5	-168.0	17-Jun-97	1200	-42.0	-119.0	996	15-Jun-97	1000	96	357	27.1	79	-95
199705	PAM	05-Dec-97	0	-9.0	-166.0	14-Dec-97	1200	-57.5	-124.5	980	09-Dec-97	230	32	274	4.6	180	32

Table A.1 - Lifetime Summary of Tropical Cyclones: 1969/1970 to 2002/2003 within 500km Rarotonga

199808	GITA	26-Feb-99	600	-21.5	-157.5	04-Mar-99	0	-43.5	-152.5	1005	26-Feb-99	600	239	98	4.1	151	-193
199809	HALI	12-Mar-99	1800	-20.0	-159.6	20-Mar-99	600	-28.0	-153.0	995	12-Mar-99	2330	118	341	3.1	250	118
199905	LEO	04-Mar-00	0	-18.0	-150.0	10-Mar-00	0	-55.0	-134.0	999	05-Mar-00	2000	140	153	10.8	241	-140
200001	OMA	18-Feb-01	0	-17.1	-162.1	24-Feb-01	0	-60.0	-129.0	986	20-Feb-01	2100	289	221	11.1	133	288
200101	TRINA	30-Nov-01	0	-21.5	-159.6	03-Dec-01	1200	-20.5	-158.2	998	30-Nov-01	0	38	149	2.1	166	-11
200201	YOLANDE	03-Dec-02	1800	-17.0	-179.0	11-Dec-02	0	-37.5	-134.0	998	07-Dec-02	930	351	192	12.0	102	351

Appendix B

Tropical Cyclone Wind and Pressure Model

Appendix B Tropical Cyclone Wind and Pressure Model

The following provides an overview of the parametric tropical cyclone wind and pressure model adopted for this study, which is similar to Harper and Holland (1999). Further elaboration is provided here of specific formulations which have been developed over a number of years as a result of extensive wind, wave and current hindcasting, e.g. Harper *et al.* (1989, 1993, 2001) and Harper (1999, 2001).

B.1 Definitions and Background

A tropical cyclone (hurricane or typhoon) is defined as a non-frontal cyclonically rotating (clockwise in the Southern Hemisphere) low pressure system (below 1000 hPa) of tropical origin, in which 10 minute mean wind speeds at +10 m MSL (V_m) exceed gale force (63 km.h^{-1}, 34 kn, or 17.5 ms^{-1}). In view of the complex nature of tropical cyclones and their interaction with surrounding synoptic scale mechanisms, most empirical wind and pressure models (Lovell 1990) represent the surface wind field by considering the storm as a steady axisymmetric vortex which is stationary in a fluid at rest.

The vortex solution is based on the Eulerian equations of motion in a rotating frame of reference (Smith 1968). The analysis begins with a consideration of force balance at the geostrophic, or gradient, wind level above the influence of the planetary boundary layer. The gradient wind speed can be expressed as a function of storm pressure, size, air density and latitude. The gradient wind speed is then reduced to the surface reference level of +10 m MSL (mean sea level) by consideration of gross boundary layer effects, wind inflow (also due to frictional effects) and asymmetric effects due to storm forward motion or surrounding synoptic pressure gradients.

B.2 Radial Pressure Field

A primary assumption of almost all empirical tropical cyclone models is that the radial pressure field at gradient wind speed level can be expressed as:

$$p(r) = p_0 + (p_n - p_0) \exp(-R/r) \qquad \text{(B.1)}$$

where r = radial distance from storm centre

$p(r)$ = pressure at r

p_0 = pressure at the storm centre (central pressure)

p_n = ambient surrounding pressure field

and R = radius to maximum winds

This exponential pressure profile was first proposed by Schloemer (1954). Holland (1980) noted deficiencies in the ability of Eqn B.1 to represent many observed pressure profiles and that the Schloemer base-profiles resembled a family of rectangular hyperbolae, viz:

$$r^B \ln [p/(p_n - p_0)] = A \qquad \textbf{(B.2)}$$

where *A* and *B* are storm-dependent scaling parameters.

This modification leads to the following radial pressure field, which forms the basis of the 'Holland' model:

$$p(r) = p_0 + (p_n - p_0)\ exp\ (-A/r^B) \quad \textbf{(B.3)}$$

B.3 Gradient Wind Speed

The gradient level winds are derived by considering the balance between centrifugal and Coriolis forces acting outwards and the presence gradient force acting inwards, leading to the so-called gradient wind equation:

$$V_g^2(r)/r \ + \ f\ V_g(r) = 1/\ \rho_a\ \ dp(r)/dr \qquad \text{(B.4)}$$

where $V_g\ (r)$ = gradient level wind at distance r from the centre

ρ_a = air density

f = absolute value of the Coriolis parameter

= $|\ 2\omega \sin \phi\ |$

and ω = radial rotational speed of the earth = $2\pi/23.93$h

ϕ = latitude

The pressure gradient term for the Holland model is:

$$dp(r)/dr = (p_n - p_0)\ /\ r\ (AB/r^B)\ exp\ (-A/r^B) \qquad \text{(B.5)}$$

and substituting into Eqn B.4 and solving the quadratic in $V_g\ (r)$ gives

$$V_g\ (r) = -r\ f/2 + [(p_n - p_0)/\ \rho_a\ (AB/r^B)\ exp\ (-A/r^B) + r^2\ f^2/4]^{½} \quad \text{(B.6)}$$

The so-called cyclostrophic wind equation, which neglects the Coriolis components, is then

$$V_c(r) = [(p_n - p_0)/\ \rho_a\ (AB/r^B)\ exp\ (-A/r^B)]^{½} \qquad \text{(B.7)}$$

with $V_c(r)$ attaining its maximum value when $dV_c(r)/dr = 0$ which, after differentiating, is satisfied when

$$-A/r^B + 1 = 0$$

and since, by definition, $r = R$ when $V_c(r)$ is a maximum

$$R \quad = \quad A^{\ 1/B}$$

or $$A \quad = \quad R^B \qquad \text{(B.8)}$$

Back-substituting into the model equations yields:

$$p(r) \quad = p_0 + (p_n - p_0)\ exp\ (-R/r)^{\ B} \qquad \text{(B.9)}$$

$$V_g\ (r) = -r\ f/2 + [(\ p_n - p_0)/\rho_a\ B(R/r)^{\ B}\ exp\ (-R/r)^{\ B} + r^2\ f^2/4]^{½} \quad \text{(B.10)}$$

which, for the particular case of B=1 the basic set of relationships reduces to the Schloemer model.

The influence of B is one of a 'peakedness' parameter which in the region of R causes an increase in pressure gradient as B increases and a corresponding increase in peak wind speed of $B^{½}$ near R and with lower wind speeds at increasing r. Holland (1980) uses conservation of angular momentum concepts and a review of pressure gradient and R data to propose restricting the dynamic range of B as 1.0 to 2.5. Furthermore, based on the climatological work of Atkinson and Holliday (1977) and Dvorak (1975), Holland suggested 'standard' B values might be inferred of the form

$$B = 2.0 - (p_0 - 900)/160 \quad \text{(B.11)}$$

making B a direct function of the storm intensity.

However, due to the inherent scatter in the climatological data it is reasonable to allow further variability whilst still maintaining the identified parameter trend, viz:

$$B = B_0 - p_0/160 \quad \text{(B.12)}$$

where B_0 is the so-called intercept value of B.

B.4 Open Ocean Atmospheric Boundary Layer

Following Powell (1980), a gross simplification of the complex atmospheric boundary later is made by transferring gradient level wind speeds (V_g) to the +10 m MSL reference level (V_m) by way of a boundary layer coefficient (K_m) viz:

$$V_m = K_m V_g \quad \text{(B.13)}$$

Additionally, variation with height above the ground is derived on the basis of a traditional roughness height and logarithmic deficit law approach whereby the near-surface boundary layer profile at any height z is a function of the surface roughness and the reference speed at +10 m MSL, ie:

$$V_m(z) = V_m(10)\ \ln(z/z_0)/\ln(10/z_0) \quad \text{(B.14)}$$

which is terminated at a nominal gradient height z_g such that

$$V_m(z_g) = V_g = V_m(10)\ \ln(z_g/z_0)/\ln(10/z_0) \quad \text{(B.15)}$$

hence

$$V_m(10) = V_g \ln(10/z_0)/\ln(z_g/10) \quad \text{(B.16)}$$

$$K_m = \ln(10/z_0)/\ln(z_g/z_0) \quad \text{(B.17)}$$

requiring a priori selection of z_0 and z_g which are both known to vary; the former as a function of wave height (wind speed and fetch) and the latter as a function of storm energetics.

North West Cape data sets presented by Wilson (1979) give a lower limit estimate of z_g as 60 m for the open ocean environment, yielding a typical z_0 of 0.3 m for wind speeds of the order of 30 m s^{-1}. Garratt (1977) provides a functional form for z_0 at lower wind speeds (generally agreed to around 20 m s^{-1}) and nominal z_g values form Standards Australia (1989) allow the following representation of the variation of z_0 and z_g after Woodside (1992):

$\ln(z_0) = 0.367\ V_m - 12$ $\quad 0 < V_m < 30$ (B.18)

$\ln(z_0) = -1.204$ $\quad V_m \geq 30$

$z_g = 228 - 5.6\ V_m$ $\quad 0 < V_m < 30$ (B.19)

$z_g = 60$ $\quad V_m \geq 30$

which, when combined into Eqn B.17 and referenced to the V_g level, yield

$K_m = 0.81 \quad 0<V_g<6 \quad$ (B.20)

$K_m = 0.81 - 2.96 \times 10^{-3} (V_g - 6) \quad 6 \leq V_g < 19.5$

$K_m = 0.77 - 4.31 \times 10^{-3} (V_g - 19.5) \quad 19.5 \leq V_g < 45$

$K_m = 0.66 \quad V_g \geq 45$

The above speed-dependent formulation for K_m was devised in an attempt to try to improve wind speed calibrations from a number of tropical cyclones in the North West Shelf region of Australia where measured wind, wave and current data was available. It embodies the observation that winds from more remote storms and/or winds on the "weak" side of storms was generally underpredicted using a constant K_m. This can also be interpreted as an attempt to devise a spatially varying K_m formulation, which has some similarity with, for example, the findings of Kepert and Wang (2000). For practical purposes in strong winds, this Eqn B.20 yields a K_m of about 0.7, which is in the range observed by Powell (1980) and subsequently, for a number of US hurricanes. In Australia, McConochie *et al.* (1999) report favourable results using the above formulation on the east coast of Queensland.

B.5 Inflow Angle and Windfield Asymmetry

In addition to direct boundary layer attenuation, frictional effects cause the inflow of winds across the line of the isobars, towards the centre of the storm. This inflow (β) is typically of the order of 25º but decreases towards the storm centre, viz:

$$\beta = \begin{cases} 10\,(r/R) & 0 \leq r < R \\ 10 + 75\,(r/R - 1) & R \leq r < 1.2\,R \\ 25 & r \geq 1.2\,R \end{cases} \quad \text{(B.21)}$$

following Sobey *et al.* (1977).

The observed gross features of moving storms is accounted for by including an asymmetry effect which, on one side of the storm adds the forward speed of the storm centre (V_{fm}) and subtracts it from the other side, relative to an assumed line of maximum wind θ_{max}, ie

$$V_m (r,\theta) = K_m V_g (r) + V_{fm} \cos (\theta_{max} - \theta) \quad \text{(B.22)}$$

Where θ_{max} is commonly taken to be in the range of either 65^{o} to 70^{o} (left forward quadrant for Southern Hemisphere) or as 115^{o} (left rear quadrant for Southern Hemisphere) measured upwind from the line of V_{fm} to which θ is referenced.

Figure B.1 presents the geometry of the wind field model in detail, including consideration of north point references for θ_{fm} and V_b (the bearing of V_m).

B.6 Wind Gust Formulae

The wind speed gust factor, G, is defined as the largest value of the average peak gust speed, of a given duration, to the mean wind speed averaged over a specified period. It is related to the longitudinal turbulence intensity Iu as follows:

$$G = 1 + g\, I_u \quad \text{(B.23)}$$

where g is a 'peak' factor normally determined from the power spectral density of the wind speed record. However, in the absence of measured data the following empirical formula after Ishizaki (1983) are used:

$$G = 1 + 0.5\, I_u \ln (T_m/T_g) \quad \text{(B.24)}$$

where T_m = mean speed reference time

T_g = gust speed reference time

such that

$$V(T_g) = G\, V(T_m) \qquad \text{(B.25)}$$

and

$$I_u = I_u\,'/\ln(V_m) \qquad \text{(B.26)}$$

where I_u ' = 0.6 for "peak gusts" and 0.4 for "mean gusts" based on the assessment of over-water wind gusts on the North West Shelf.

B.8 Radius to Maximum Wind Estimates

Estimates of R are rarely available for storms which are remote from measurement sites and outside radar range but this parameter can have an important influence on, for example, the fetch available for wind-wave generation. As an aid in determining suitable *R* values in the absence of any direct information, an empirical relationship has been developed based on available data from Australian and US sources.

The *R* hypothesis is based on the proposition (Myers 1954) that the storm spatial scale and the central pressure differential are related throughout the life of a given storm. The evidence for this appears reasonably substantial but the physical basis is by no means established. Myers presented an argument based on conservation of kinetic energy within a nominal radius of the storm centre which showed a hyperbolic relationship linking radius to maximum winds and the central pressure deficit viz:

$$R = F\,[p_n\text{-}p_0] \qquad \text{(B.27)}$$

An analysis of over 20 separate tropical cyclones in the north-west Australian sector was undertaken using the time history of *R* values throughout each storm for both the intensifying and decaying legs and a series of best fit relationships were developed of the form:

$$R(t) = R_c\,/\,(p_n\text{-}p_0)(t) \qquad \text{(B.28)}$$

where R_c represents a scaling parameter with units of hPa.km and *t* is time.

Based on the Australian experience R_c values for the intensifying leg are likely to be in the range of 650 to 3000, with a mean value around 1850 hPa.km. Using US Gulf Coast data from NOAA (1979) a range of 900 to 4300 is indicated with a mean of 2100 hPa.km. Other regions may exhibit slightly different characteristics.

It should be noted that no relationship between R_c and $(p_n\text{-}p_0)$ is itself proposed but rather that for any given storm intensity it is reasonable to ascribe a particular trend in spatial variation over time. On this basis storms of vastly different intensities might still share a common R_c value. In the model the R_c value is applied only to the intensifying leg and is made monatonically decreasing in *R* towards minimum p_0 such that any minor fluctuations in pressure are ignored. Also, based on Holland (1990), *R* is held constant in the decaying leg and is always limited to a practical maximum value in the range of 80 to 100 km.

Where radar eye data is available, the radar radius to the eyewall echo is taken and a constant 5 km added to estimate the position of the radius to maximum winds. This is based on experience and is consistent with available data from historical storms, e.g. Hurricane *Andrew* in 1992.

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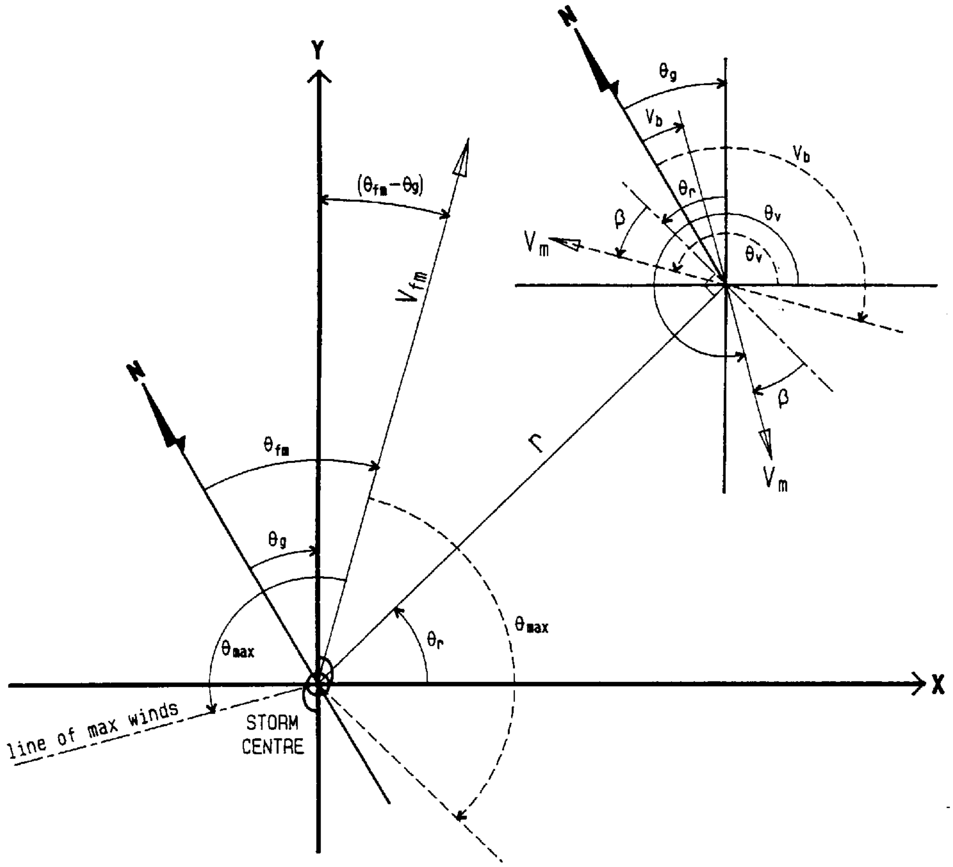

	SOUTHERN HEMISPHERE	NORTHERN HEMISPHERE
θ_{max}	$+ 115$	$- 115$
θ_v	$270 + \theta_r - \beta$	$90 + \theta_r + \beta$
V_b	$- \theta_r + \beta + \theta_g$	$180 - \theta_r - \beta + \theta_g$

TROPICAL CYCLONE MODEL
WIND FIELD GEOMETRY

Figure B.1

Appendix C

Tropical Cyclone Spectral Wave Model – ADFA1W

Appendix C Tropical Cyclone Spectral Wave Model - ADFA1W

C.1 Overview of the Model

A comprehensive description of the numerical spectral wave model ADFA1 can be found in Young (1987a, 1987b). ADFA1 is a further development by the original author of the 2nd generation model SPECT (Sobey and Young 1986), originally from James Cook University, having enhanced shallow water and non-linear source terms.

The complex sea state is described by the model in terms of the directional wave energy spectrum $E(f,\theta,x,y,t)$. At each position x,y and time t, E represents the superposition of free linear wave components of all frequencies f and all directions θ. The evolution of the energy spectrum is then described by the *Radiative Transfer Equation*:

$$\frac{\partial}{\partial t}(C C_g E) + C_g \cos\theta \frac{\partial}{\partial x}(C C_g E) + C_g \sin\theta \frac{\partial}{\partial y}(C C_g E) + \frac{C_g}{C}\left[\sin\theta \frac{\partial C}{\partial x} - \cos\theta \frac{\partial C}{\partial y}\right]\frac{\partial}{\partial \theta}(C C_g E) = C C_g S \qquad \text{(C.1)}$$

Where $C(x,y,f)$ = the individual wave phase speed

$C_g(x,y,f,\theta)$ = the wave group speed

$S(f,\theta,x,y,t)$ = a source term representing the net transfer of energy to, from or within the spectrum

The kinematics of wave propagation are described in the model by ray theory, neglecting the effects of currents. This allows wave propagation to be represented by characteristic equations.

The net source term S is represented as the summation of a number of separate influences:

5. atmospheric input
6. non-linear wave-wave interactions
7. white cap energy dissipation
8. bottom friction
9. shallow water wave breaking

Atmospheric forcing is provided by specification of the 10 minute average wind speed and direction at the standard reference height of +10 m SWL (V_m). In the present investigation, this is provided by the Holland (1980) tropical cyclone wind field model. This was incorporated into ADFA1 and updates wind speed and direction at each x, y location and at each time step t based on the position of the storm centre, and the various storm parameters, central pressure, radius to maximum winds and ambient pressure.

Eqn C.1 is solved numerically using a fractional step method consisting of separation of propagation and forcing mechanisms. This method avoids the penalty of numerical dispersion in the solution. The propagation solution (which includes refraction and shoaling) is obtained from the method of characteristics, assuming only the influence of bathymetry. A separate wave characteristic is constructed for each frequency and direction component of the discrete representation of the spectrum and at each model point within the computational grid. The set of characteristic paths need be only determined once for each particular computational grid, provided changes in water depth will not be significant throughout a storm simulation.

Boundary conditions are either of the radiation type where there are no significant generation areas beyond the computational limits, or a system of sub-grids may be used to provide greater geographical detail where necessary. Boundary data for the finer sub-grid are provided post-hoc from the coarser parent grid.

Model output can be either the time history of the relevant spectral parameters ($H_s, T_p, T_z, T_m, \theta_m$) at particular computational grid locations, contours of H_s and vector fields of T_p and θ_m over the entire region, one-dimensional spectral energy plots at particular locations and times or full directional energy density contours throughout the simulation.

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Appendix D

Surge and Tide Simulaton Model – SATSIM

Appendix D - Surge and Tide SIMulation Model - SATSIM

D.1 Background

SATSIM is a discrete *Monte-Carlo* statistical model employing tide generation and a parametric tropical cyclone storm surge model, which can be applied to arbitrary coastal or open ocean areas. The early model was based on techniques first described by Stark (1976, 1979) and Harper and Stark (1977) and is similar to Russell (1971) as applied in the Gulf of Mexico. SATSIM was formalised by Harper and McMonagle (1983) and used to establish design water levels along the Queensland coast (Harper 1983, 1985), the Northern Territory (Harper and McMonagle 1983) and parts of Western Australia (Stark and McMonagle 1982). The model was further extensively developed in the late 1980s to include parametric tropical cyclone wave, wind and 3-D current models (Harper *et al.* 1989). More recently, the same basic technique has been further extended to include wind estimation and building damage in an even more complex model (MIRAM) which includes severe thunderstorms as well as tropical cyclone wind and storm surge (Harper 1996ab, 1997, 1999). The latest variant of SATSIM includes breaking wave setup over coral reefs and shallow water bathystrophic storm tide effects (SEA 2001).

D.2 Definitions

The total water level experienced at a coastal, ocean or estuarine site during the passage of a severe meteorological event such as a tropical cyclone, is made up of contributions from some or all of the following components. The combined water level is termed the *storm tide,* refer Figure D.1.

(a) The Astronomical Tide

This is the regular periodic variation in water levels due to the gravitational effects of the Moon and Sun. With a suitably long period of tide measurements at a specific location, combined with harmonic analysis, the tide can be predicted with very high accuracy at any point in time (past and present). The highest expected tide level is termed *Highest Astronomical Tide* (HAT) and occurs once each 18.6 y period, although at some sites tide levels similar to HAT may occur several times per year.

(b) Storm Surge

This is the combined result of the severe atmospheric pressure gradients and wind shear stresses of a significant meteorological event such as a tropical cyclone acting on the underlying water body. The storm surge is a long period wave capable of sustaining above-normal water levels over a number of hours. The wave travels with and ahead of the storm and may be amplified as it progresses into shallow waters or is confined by coastal features. Typically the length of coastline which is severely affected by a tropical cyclone storm surge is of order 100 km either side of the track although some influences may extend many hundreds of kilometres. The magnitude of the surge is affected by many factors such as storm intensity, size, speed and angle of approach to the coast and the coastal bathymetry.

(c) Breaking Wave Setup

Severe wind fields create abnormally high sea conditions and extreme waves may propagate large distances from the centre of the storm as ocean swell. These waves experience little or no attenuation in deepwater regions and an offshore storm can impact several hundred kilometres of coastline. As the waves enter shallower waters they refract and steepen under the action of shoaling until their stored energy is dissipated by wave breaking either offshore or at a beach or reef. Just prior to breaking, a phenomenon known as *wave setdown* occurs where the average stillwater level is slightly lower than the same level further offshore. After breaking, a portion of the wave energy is converted into forward momentum which, through the continuous action of many waves, is capable of sustaining shoreward water levels which are above the stillwater level further offshore. This quasi-steady increase in stillwater level after breaking is known as *breaking wave setup* and applies to most natural beaches and reefs.

There remain other related phenomena which can also affect the local water level. These may include long period shelf waves, unsteady surf beat, wave runup, stormwater and/or river runoff etc. Any phenomenon which can be deterministically described in space and time with respect to the incident storm parameters can be incorporated into the SATSIM methodology.

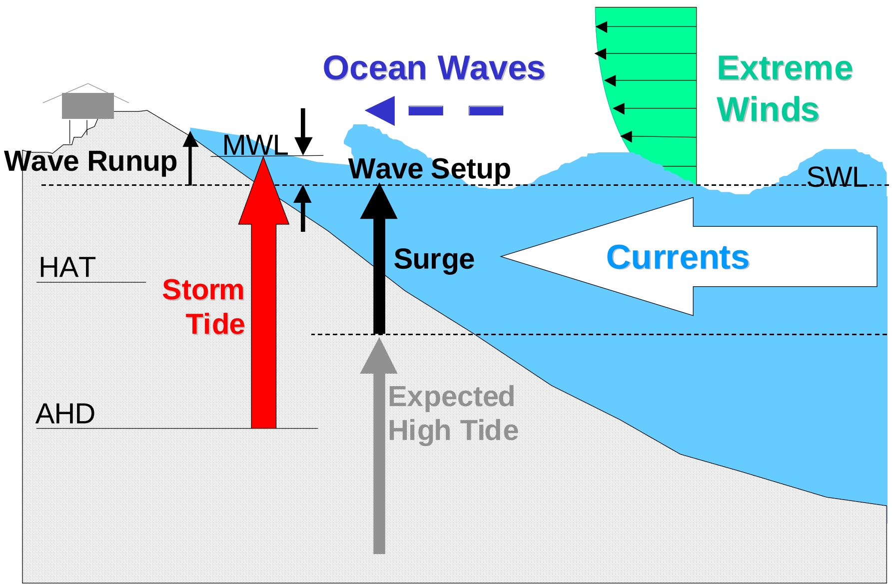


Figure D.1 Components of total water level.

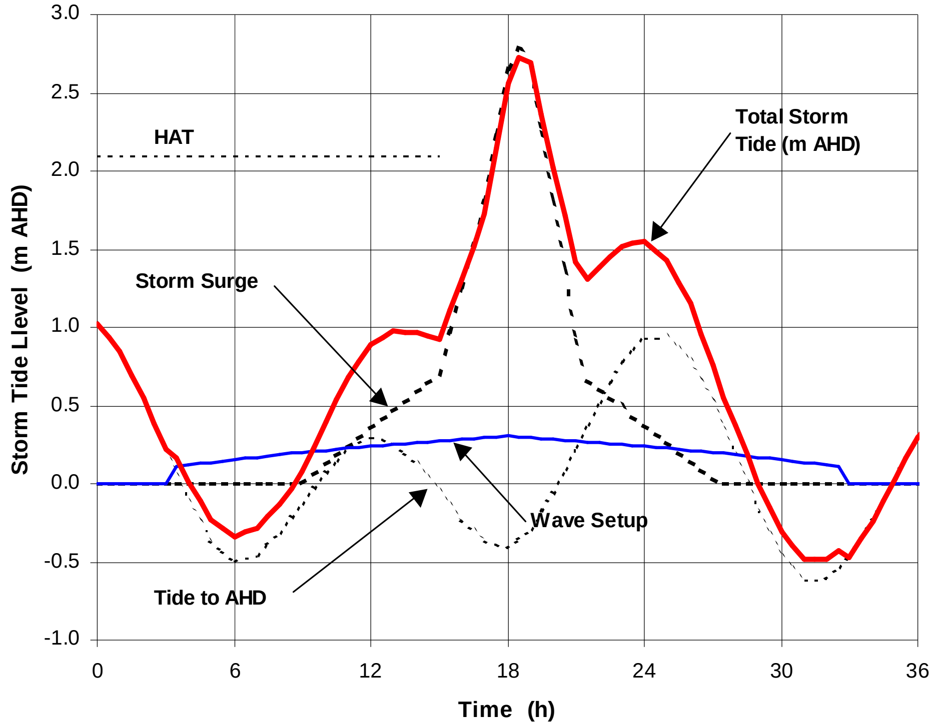
D.2 Basic Methodology

(a) Deterministic Phase

SATSIM consists of a series of water level forcing modules which can provide an estimate of the time history of each of the water level components of interest. In the case of the astronomical tide, the time history of water levels is provided directly from a set of harmonic constituents for the site under consideration and tidal planes (e.g. AHD) provide a base water level datum. The storm surge and breaking wave setup time histories are provided by a series of *parametric* models which describe the likely behaviour of the respective component as a function of the incident storm parameters (e.g. distance of approach, intensity, track, size etc). These parametric models are derived from a combination of complex numerical hydrodynamic models (e.g. SURGE, ADFA1) as well as analytical approximations such as those for breaking wave setup (e.g. Nielsen and Hanslow 1991; Gourlay 1997).

The model typically considers a 36 h "window" for each storm tide event and generates simultaneous and independent estimates of each of the water level components at a time interval of 30 mins. These are then linearly combined using superposition to provide the estimated total *storm tide* level over that time as shown schematically in Figure D.2, which closely approximates the Cyclone *Althea* storm tide at Townsville in 1971 (Stark 1972).

Figure D.2 Example of the superposition process.



(b) Probabilistic Phase

A number of different probabilistic variants of the model have been developed. All approaches are based on the concept of defining a statistical *control volume* around the site of interest. This may be in any geometric form such as a square or rectangular domain or a radius from the site, termed the *target site* (refer Figure D.3). The climatology of the meteorological forcing within that control volume is then determined based on either the analysis and interpretation of historical data or, where no data exists, hypothetical statistical distributions of the parameters of interest.

In Australia, tropical cyclone tracks and estimates of central pressure have been variously recorded and archived by the Bureau of Meteorology since the early 1900s. The quality of the data is quite variable in space and time (e.g. Holland 1981) and as a general rule is only suitable for statistical analysis from around 1959/60 onwards. This marks the commencement of routine satellite imagery and the adoption of objective intensity estimation methods. Individual storms which passed close to recording sites prior to this time are still suitable for inclusion but care must be taken not to bias the overall statistical descriptions.

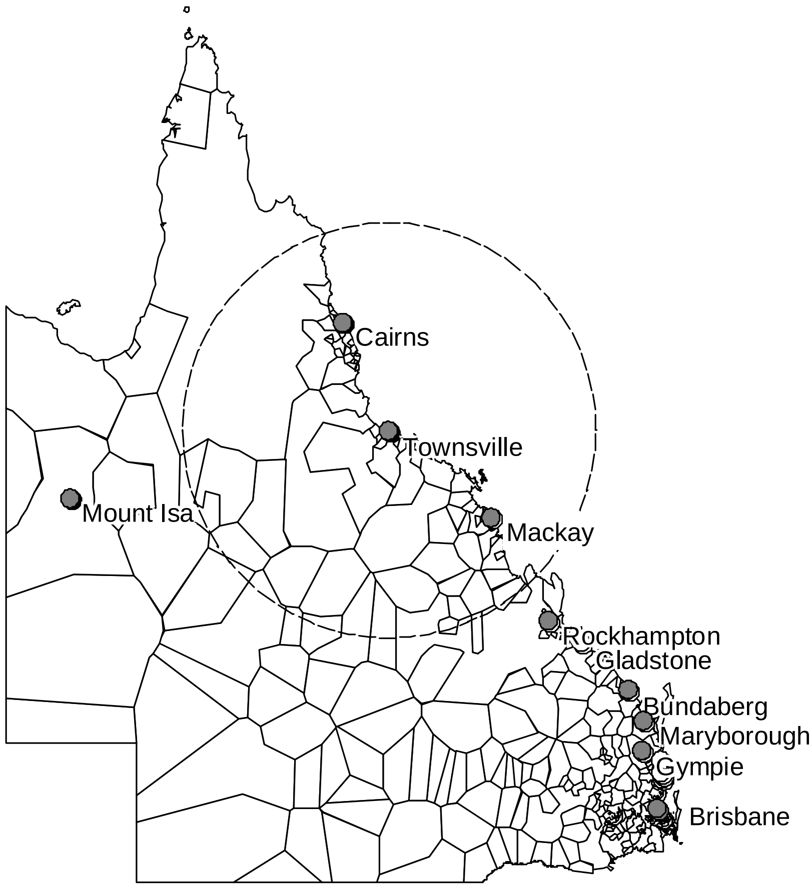


Figure D.3 Example of a 500 km radius statistical control volume with Townsville as the target site.

The climatology of storms within the control volume then is normally expressed in terms of the following major components:

Population class

At any single location it is common for the incidence of tropical cyclones to be due to two or more separate storm populations. These can normally be clearly identified by origin and track but other more complex discriminators may be required.

Frequency of occurrence

The relative frequency of occurrence between populations is often a further discriminator.

Intensity

Different populations often exhibit varying intensity behaviour which is typically related to the origin and track of the storms relative to the prevailing atmospheric patterns and landmass effects.

Scale

This typically relates to the radius of maximum winds or the radius to gales and influence the extent of storm surge or wave generation fetch etc.

Forward speed and track

The speed of approach to the coast and the angle of crossing, for example, influence the generation of storm surge.

Distance of closest approach

This is one of the principal determinants of impact at any site, the tropical cyclone structure is spatially variable and the region of maximum effect is typically within 2 to 3 radius to maximum winds of the centre.

D.3 Statistical Model

The model utilises a discrete Monte Carlo approach, whereby a random number generator is used to provide a source of unbiased probability, and a series of individual storm events are created based on the climatological description. The deterministic output from each hypothetical storm event is then created, based on the relationships determined between the storm parameters and the impacts of interest (surge, waves, wave setup etc). A 36 h window is typically allowed for each event and simultaneous time histories of each impact at a resolution of 0.5 h are assembled and combined as required to yield the output of interest (e.g. storm tide level). The statistics of each event are then recorded in terms of the frequency of exceedance of a range of given magnitude levels. After many thousands of samples from the control volume, the statistical exceedance function becomes smoothed and simulation ends when the function has converged sufficiently at the desired probability level. For example, to estimate the 100 year return period (or 1% annual exceedance), at least 1000 years of simulation is recommended so that there will be at least 10 estimates of the 100 year magnitude. Figure D.4 illustrates the basic model structure in flowchart format.

The forms of the statistical representations used are typically:

Frequency of Occurrence	Poisson
Storm Intensity	Gumbel (EV Type I)
Forward Speed	Smoothed Data CDF
Track	Smoothed Data CDF
Closest Approach	Smoothed Data CDF
Radius to Maximum Winds	Normal CDF
Windfield Peakedness	Normal CDF

Any of the input statistical distributions may then be altered to test the sensitivity of the model results to the input assumptions.

D.4 Model Variants

SATSIM has been variously developed over a number of years according to the needs of the particular analysis. The following provides an introduction to some of the specific versions which were used in major or landmark studies. Individual study reports should be consulted for further details.

V3 through V4

These versions were used for the series of studies conducted during the early 1980s (e.g. Harper 1983; Harper and McMonagle 1983, 1985). It considers a rectangular control volume of nominally 5º of latitude alongshore (556 km) and 2.5º of longitude offshore (278 km). Tidal constituent data for the target site was provided and extended to up to 10 secondary sites by the use of published range ratios. The coastal storm surge response was parameterised according to intensity, track, closest approach and forward speed based on the results of a series of numerical hydrodynamic model tests (e.g Harper 1977 for each of 10 locations along the Queensland coast). Some versions incorporated breaking wave setup and also coastal wave height, these being derived from a series of model tests using the SPECT model (Sobey and Young 1986).

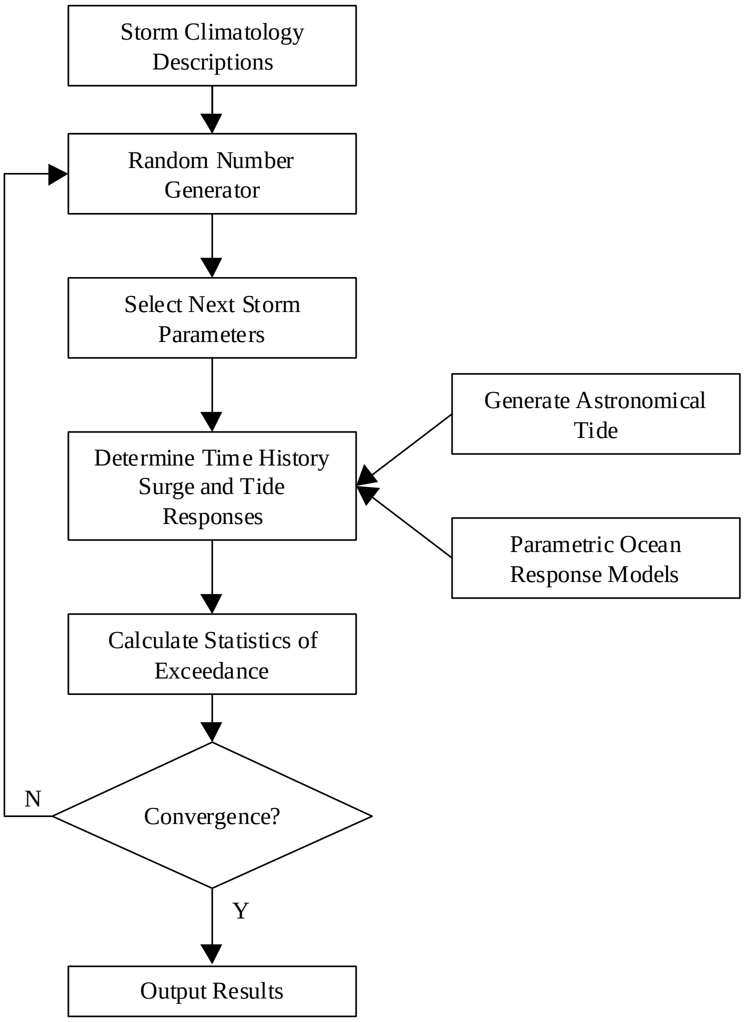


Figure D.4 Flowchart of the model simulation process.

V5 through V8

These versions were developed under licence by Woodside Offshore Petroleum Pty Ltd in the late 1980s to provide design criteria for the Goodwyn 'A' offshore production platform on the North West Shelf of Western Australia (Harper et al. 1989, 1990). A radius of influence of 1000 km was taken to represent the statistical control volume around a single site. These versions provided full (contemporaneous) statistical descriptions of environmental loadings on an offshore platform allowing phase separation at very long return periods (10,000yr). Hurricane wind fields could be specified as NHRP circa 1970 or according to a modified and extended Holland (1980). Each site impact of interest was separately modelled, e.g.

V5 deepwater storm surge (inverted barometer effect) driven directly from the parametric wind and pressure field model dependent upon the relative position of the site and the storm centre.

V6 wind speed and direction (mean and gust) driven directly from the parametric wind and pressure field model, as above.

V7 wave height (H_s, H_{max}), period (T_z, T_m, T_p), direction (θ_m) parameterised based on over 200 separate spectral wave model tests using the ADFA1 model (Young 1987) - an updated version of the SPECT model. A two-stage nested-model domain system was used with resolutions of 54 km and 10.8 km. Results were summarised in terms of a series of complex tabular functions describing the wave conditions of straightline tracks as a function of various storm parameters and position relative to the target site. Long-term directional wave counts were also estimated for structural fatigue considerations. Maximum wave heights and associated periods were determined by numerical integration of the time history of significant wave heights and periods (e.g. Sobey *et al.* 1990).

V8 3D currents (barotropic, baroclinic, pulsed) were similarly parameterised on the basis of a series of sensitivity tests using a hydrodynamic model after Fandry (CSIRO Division of Marine Research), conducted by Steedman Science and Engineering (SSE 1989).

The Woodside developments included significant calibration and verification testing of the various parametric model components against extensive measured wind, wave and current datasets.

V9a

This version was developed to represent storm tide impacts at the Cocos (Keeling) Islands in the Southern Indian Ocean on behalf of GHD Pty Ltd, acting for the Commonwealth Department of Transport and Regional Services (SEA 2001). The selected radius of influence was 500 km. The model combines a number of aspects of previous models, namely:

Astronomical tide

Deepwater inverted barometer effect

Mean and gust wind speed

Parametric open ocean tropical cyclone waves

As well as some additional capabilities:

Ability to represent up to 20 sites around the island by a directionally sensitive wave sub-model, further modifying the V7 open ocean model

Breaking wave setup over the fringing reefs based on Gourlay (1997)

Bathystrophic storm tide effects within the island lagoon

This version of the model simultaneously generates estimates of all impacts for all sites.

D.5 Algorithms

D.5.1 Astronomical Tide

The astronomical tide is specified only for the target site and secondary sites may have an associated range ratio to allow variation from the target site. No phase differences are incorporated, with phase being regarded as a random variable in this context. The target site tide is specified by up to 36 harmonic constituents (amplitudes , phases) together with the relevant datum planes for z_0, MSL and HAT.

D.5.2 Tropical Cyclone Winds and Pressures

The Holland (1980) model formulation is used, as modified and extended by Harper and Holland (1999).

D.5.3 Inverted Barometer Effect (IBE)

This is represented by

$$\Delta B = \frac{(p_n - p_s)}{\rho_w \; g}$$

where p_s is the local MSL atmospheric pressure; p_n is the ambient or surrounding MSL atmospheric pressure; ρ_w is seawater density; g is gravity. The magnitude of B is then typically 10 mm for each 1 hPa pressure difference.

D.5.4 Bathystrophic Storm Tide (BST)

This is the first-order 1D momentum balance for a steady-state wind stress scenario which, considering the *x* direction is given by:

$$0 = -g\,(h+\eta)\frac{\partial \eta}{\partial x} - \frac{(h+\eta)}{\rho_w}\frac{\partial p_s}{\partial x} + \frac{1}{\rho_w}(\tau_{sx} - \tau_{bx})$$

where the *x-y* datum plane is located at the mean water level with the *z* axis directed vertically upwards; the water surface elevation w.r.t. datum is *(x,y,t),* the seabed is *h (x,y)* below datum. The forcing influence of the tropical cyclone is represented through the surface wind shear stress vector component τ_{sx} and the *x* gradient of the MSL atmospheric surface pressure $p_s(x,y,t)$. The effect of bottom stress is represented by the bottom shear stress vector component τ_{bx}.

Following the SURGE model (Sobey *et al.* 1977), the surface stress and bottom stress components are represented parametrically. For example, the surface wind stress forcing is parameterised w.r.t. the 10-minute mean wind speed component W_{x10} at the standard reference height of +10 m MSL by

$$\tau_{sx} = C_{10}\,\rho_a\,W_{x10}^{\;2}$$

where ρ_a is the air density and C_{10} is an empirical coefficient whereby (Wu 1982)

$$10^3\,C_{10} = 0.8 + 0.065\,W_{x10}$$

The effect of bottom stress is parameterised by a Darcy-Weisbach equation with *U* the *x* component of flow and λ the Darcy-Weisbach friction factor, e.g.

$$\tau_{bx} = \frac{\lambda}{8}\,\rho_w\,\frac{U^2}{(h+\eta)^2}$$

with λ assumed depth-dependent according to the hydraulically rough Colebrook-White formula, with roughness height k_b, e.g.

$$\frac{1}{\lambda} = -2\log_{10}\left[\frac{k_b}{14.8\,(h+\eta)}\right]$$

where k_b is typically set at 0.025 m for coastal areas.

However, U remains an unknown in this context and is therefore further parameterised by the surface wind speed

$$U = k_u\, W_{x10}$$

assuming k_u is a fixed nominal value of 0.03 (e.g. Bishop 1979).

The surface elevation is then calculated based on a given fetch and depth profile using a Runge-Kutta integration technique.

D.5.5 Coastal Storm Surge

This follows the method outlined in Harper and McMonagle (1985).

D.5.6 Tropical Cyclone Waves and Currents

This follows the tabular look-up methodology described in Harper *et al.* (1990), which is based on a schematised storm reference system as shown in Figure D.5. Straightline tracks of constant speed are assumed but with a symmetric variation in central pressure based on a Gaussian function. Radius to maximum winds varies as a function of pressure differential for a given R_c constant.

Some examples of the tabular functions which comprise the open ocean tropical cyclone wave model are shown in Figure D.6. Clockwise from top left these are: peak wave height as a function of central pressure; modification due to forward speed; modification due to along-track position; modification due to across-track position. Track "1" paths are fetch limited and apply to the North West Shelf of Western Australia, while Track "2" applies to any open ocean site. Similar functions describe the variation of wave periods, direction and shape of the hydrograph.

The model incorporates a bias adjustment for H_s determined from detailed calibration studies with 23 tropical cyclones (Harper 1992) which identified an apparent cross-track bias in the ADFA1 spectral wave model, thought to be due to non-linear wave-wave interactions in the rotating wind field. The adjustment is implemented as a linear function according to the relative x position within a nominal y domain, as follows:

$$H_s' = \frac{H_s}{E_r}$$

where

$$E_r = 0.00196\,x + 0.92089 \quad -100 < x < 100$$

with x in km and a clipped linear return to unity at -200 and +200 . The applicable y domain is defined by

$y0 < -100$ and $y2 > 0$

D.5.7 Breaking Wave Setup

The method of Nielsen and Hanslow (1991) is applied for plane beaches while that of Gourlay (1997) is applied for reefs.

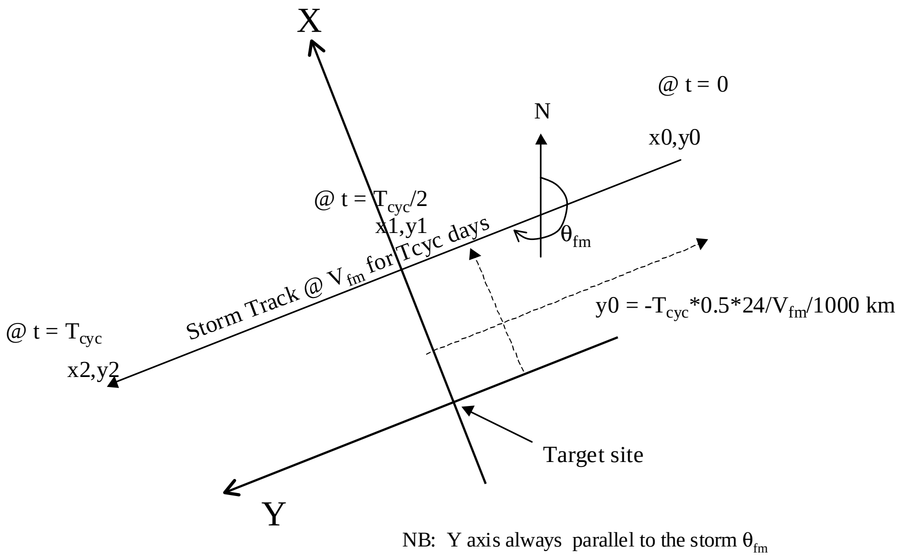


Figure D.5 Model reference system for schematised tropical cyclones.

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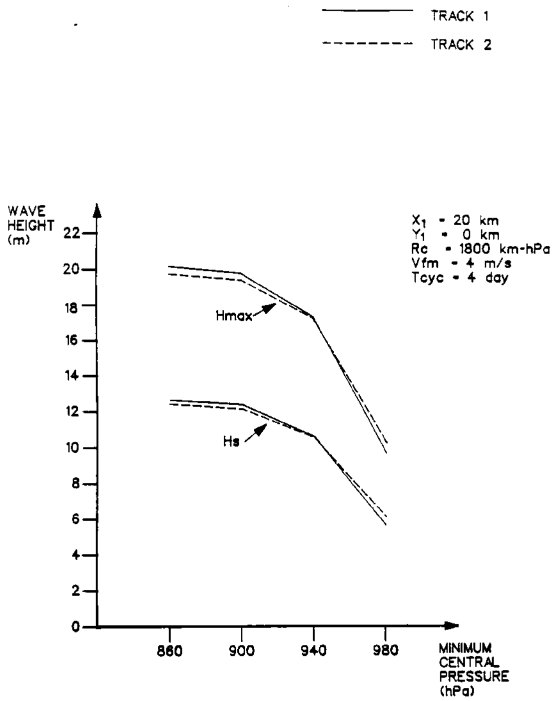


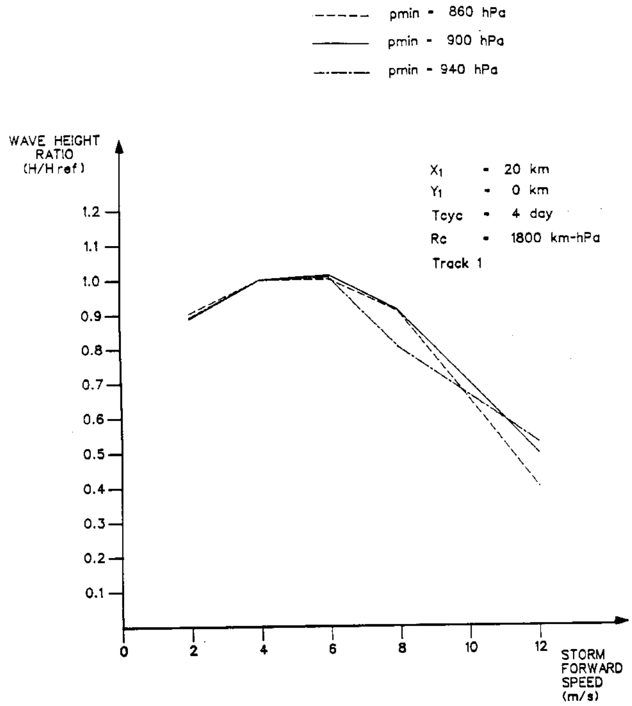
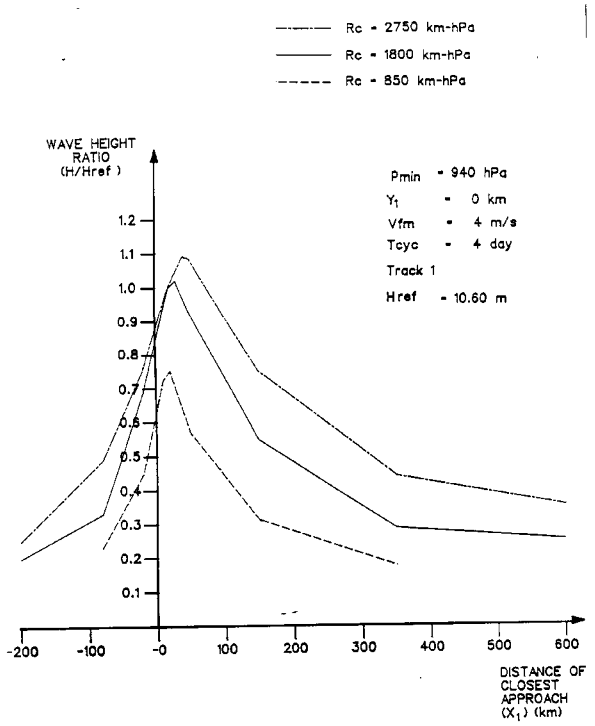
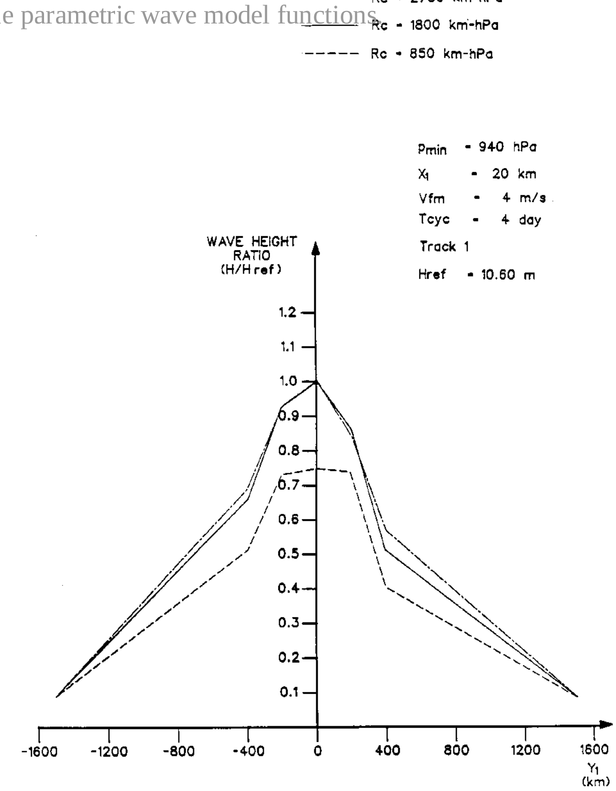


Figure D.6 Example open ocean tropical cyclone parametric wave model functions





Appendix E

Breaking Wave – Induced Setup in Coral Reefs

Appendix E Breaking Wave-Induced Setup on Coral Reefs

Coral reef environments present a specific set of physical characteristics which can often create very significant nearshore wave setup, raising the water levels on the reef-top, and driving reef-top current systems that may control lagoon flushing and sediment transport processes. In extreme situations, wave setup can be responsible for overtopping and flooding of low lying islands and is likely to be the largest component of storm tide at some offshore island sites. In particular, the combination of astronomical tide variation, storm surge and incident wave height and period present a very dynamic and sensitive wave setup environment.

E.1 Definitions

Figure F.1 presents a schematic view of a typical coral reef nearshore environment (Gourlay 1997), where:

Reef-face is the relatively steep seaward facing underwater slope of the reef;

Reef-top the skyward facing surface of the reef, usually submerged except at low tides;

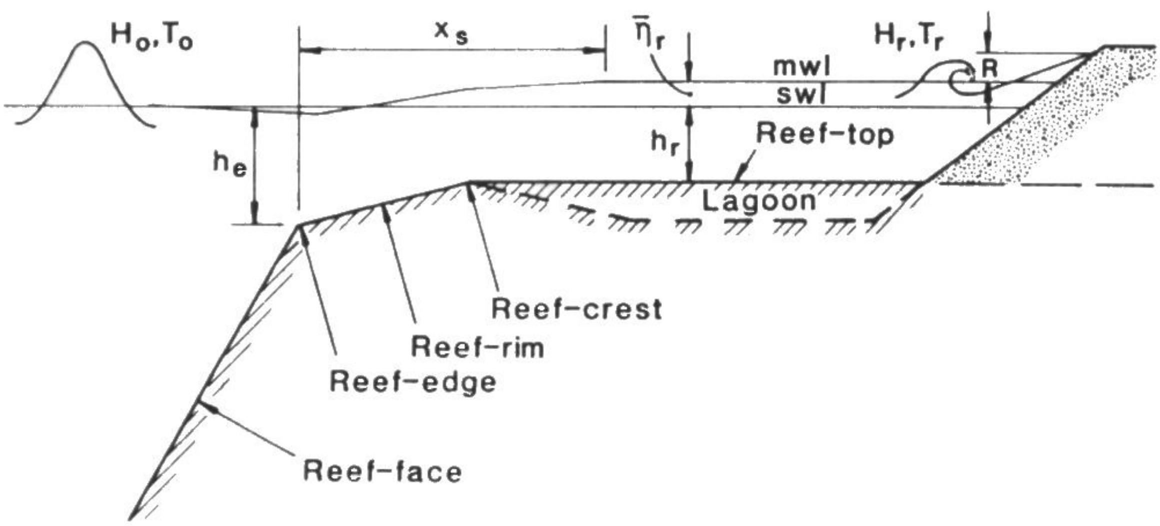
Reef-rim the relatively flat seaward inclined surface between the reef-top and the reef-face;

Reef-edge the intersection between the reef-face and the reef-rim;

Reef-crest the highest part of the reef-rim or the intersection between the reef-rim and the reef-top;

Lagoon a body of water ponded on or enclosed by a reef or by a reef and a continental or island land mass.

Figure E.1 Coral reef wave setup definitions (after Gourlay 1997).



Clearly, reefs and reef platforms represent potentially very complex shorelines which, being living environments, have evolved at any specific location to be in equilibrium with the incident wave and tide conditions. Even relatively small areas of reef platforms may display a myriad of channels, ridges and holes. Any estimate of wave setup must therefore be considered in a very generalised manner and applied with caution to specific locations. In particular, accurate information on reef-top water levels and slopes will be seen to be critical to any accurate assessment of reef setup.

E.2 Estimation of Wave Setup

Using an analysis combining wave energy flux and radiation stress concepts and assuming deepwater conditions offreef and shallow water conditions on the reef-top, Gourlay (1994b) derived the following equation for the wave setup $\overline{\eta}_r$ on a reef-top as a function of the offreef wave conditions H_o and *T* and the stillwater depth h_r on the reef-top:

$$\overline{\eta}_r = \frac{3}{64\pi} K_p \frac{g^{1/2} H_o^2 T}{(\overline{\eta}_r + h_r)^{3/2}} \left[1 - K_r^2 - 4\pi K_R^2 \frac{1}{T} \sqrt{\frac{(\overline{\eta}_r + h_r)}{g}} \right] \quad \text{(E.1)}$$

where K_p is a reef profile factor (varies according to reef profile up to approximately 0.8)

K_R is the reflection coefficient (0 to 1)

K_r is the transmission coefficient (0 to 1)

and the term in [...] is subsequently referred to as the *transmission parameter* P_T .

Laboratory experiments (Gourlay 1996a) indicate that for steep reef-faces $K_R \le 0.3$, whereas for flatter slopes (Gourlay 1994) $K_R \le 0.1$. Hence wave reflection at most decreases the wave setup by about 10% and can be reasonably neglected.

The influence of wave transmission on wave setup however is a function of the relative submergence *S*, viz.

$$S = \frac{(\overline{\eta}_r + h_r)}{H_o} \quad \text{(E.2)}$$

and is also relatively small when S < 1. However it becomes increasingly significant as the submergence increases until, when S 〈 2.5, waves pass over the reef *without* breaking and hence without generating any setup (Gourlay 1996a).

To facilitate analysis, the transmission coefficient K_r $(\equiv H_r / H_o)$ can be expressed in terms of the reef-top wave height to depth ratio γ_r $(\equiv H_r / (\overline{\eta}_r + h_r))$ The resulting form of the transmission parameter P_T , neglecting K_R, is then:

$$P_T = \left[1 - 4\pi\, \gamma_r^2\, \frac{1}{T} \sqrt{\frac{(\overline{\eta}_r + h_r)}{g}} \left(\frac{(\overline{\eta}_r + h_r)}{H_o} \right)^2 \right] \qquad \text{(E.3)}$$

or

$$P_T = \left[1 - 4\pi\, \gamma_r^2\, S^2 / D \right] \qquad \text{(E.4)}$$

where *D* is the inverse of the relative depth of the reef-top waves, i.e.:

$$D = T \sqrt{\frac{g}{(\overline{\eta}_r + h_r)}} \qquad \text{(E.5)}$$

On a horizontal or near-horizontal reef-top $\gamma_r \leq 0.55$ (Gourlay 1994; Nelson 1994),i.e. maximum wave heights never exceed 0.55 times the reef-top water depth $(\overline{\eta}_r + h_r)$. For regular waves with significant dissipation at the reef-edge, laboratory studies show $\gamma_r = 0.4$ (Gourlay 1994) and this value also has been found to apply for significant wave heights in the field (Hardy *et al.* 1991). Hence, when P_T is calculated with typical values of $\gamma_r = 0.4$ and D = 12.5, it can be simplified to the following form:

$$P_T = \left[1 - 0.16\, S^2 \right] \qquad \text{(E.6)}$$

thus correctly representing the observed condition that full transmission (and hence zero setup) occurs when S 〈 2.5. Alternatively, P_T = 1 when S=0 (or rather K_R = 0 and K_r = 0), and the maximum reef-top wave setup is estimated as:

$$\overline{\eta}_{r\,\max} = \frac{3}{64\pi} K_p \frac{g^{1/2} H_o^2 T}{(\overline{\eta}_r + h_r)^{3/2}} \qquad \text{(E.7)}$$

with typically achievable maximum setup values in the range 0.25 to 0.80 m even for average swell conditions at many exposed coral atolls, and potentially increasing above 3 m in extreme wave conditions, being modulated significantly by the tide and/or incident storm surge levels.

The reef profile factor K_p depends upon the roughness, permeability and shape of the reef. Gourlay (1996b) provides a range of K_p values derived from laboratory studies which increase with increasing profile slope tan α. For waves breaking at the reef-edge, the reef-face slope determines the value of K_p, whereas for waves breaking on a seaward sloping reef-rim, the reef-rim slope determines K_p. In the latter case it was found (Gourlay 1997) that an average water depth h_a determined over the reef-rim surf zone, was more appropriate than the reef-top water depth h_r for calculating the wave setup and use of a modified K_p' is recommended. Both relationships with respect to tan α are presented in Figure F.F.

Hence, two situations are possible:

(a) Wave breaking occurs at the reef-edge:

tan α is taken as the reef-face slope (i.e. into deepwater)

- S is calculated using either h_r for a horizontal reef-top or the average depth h_a over the reef-rim (assuming that the surf zone x_S extends over the full width of the reef-rim).
- the appropriate reef profile factor is K_p taken from Figure E.2

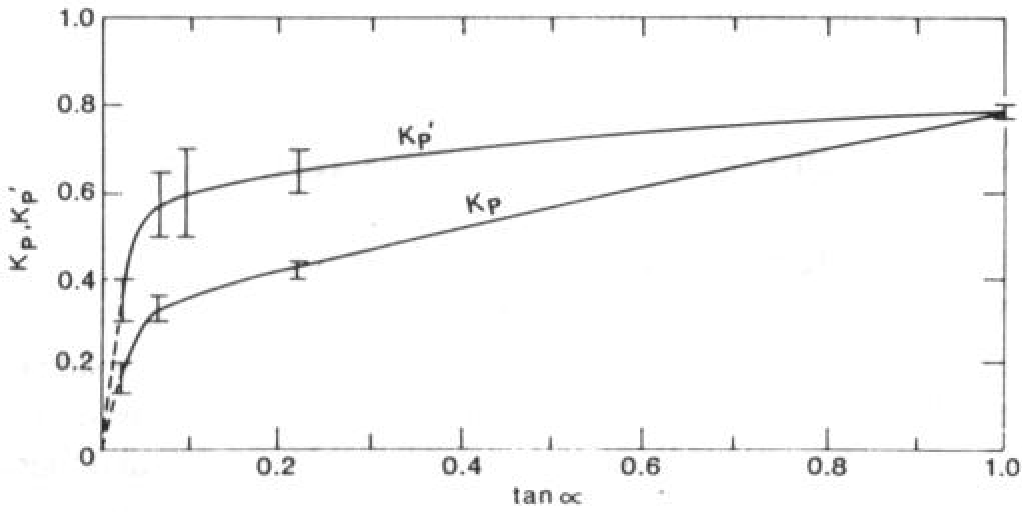


Figure E.2 K_p and K_p' as a function of tan α (after Gourlay 1997).

(b) Wave breaking occurs at the reef-rim:

- tan α is taken as the reef-rim slope
- the breakpoint should be calculated (i.e. the breaker depth related to the reef-rim slope)
- S is calculated using the average depth h_{ba} on the reef-rim between the breakpoint and the reef-crest (for tan α > 0.1, refer Gourlay 1997)
- the appropriate reef profile factor is K_p' taken from Figure E.2

In the latter situation, the breakpoint depth ($d = d_b$) on the reef-rim can be estimated as proposed by Gourlay (1992), ignoring possible wave-setdown, viz:

$$\frac{d_b}{H_o} = 0.259\left[\tan^2\alpha\left(\frac{H_o}{L_o}\right)\right]^{-0.17} \qquad \text{(E.8)}$$

Furthermore, calculation of h_a or h_{ba} implies knowledge of the surf zone width x_S which can be estimated by the following equation from Gourlay (1994):

$$x_S = \left(2 + 1.1\frac{H_o}{h_e}\right)T\sqrt{g\,h_e} \qquad \text{(E.9)}$$

where h_e is the reef-edge depth.

F.3 Irregular Waves

Finally, irregular waves may be considered in an analogous manner, substituting the offreef wave parameters as follows, e.g.

$$H_o = H_{0rms} \equiv \frac{H_{os}}{\sqrt{2}} \qquad \text{(E.10)}$$

where H_{os} is the offreef significant wave height, and

$$T_o = T_{0p} \qquad \text{(E.11)}$$

where T_{op} is the peak spectral wave period, yielding a maximum reef-top setup value from Eqn E.7 of:

$$\overline{\eta}_{r\max} = 0.023\, K_p \frac{H_{os}^2\, T_{op}}{(\overline{\eta}_r + h_r)^{3/2}} \qquad \text{(E.12)}$$

This maximum condition also assumes that wave crests approach normal to the reef profile and that H_{oS} is a measure of the shore-normal wave energy. This will not always be the case since the reefs are often surrounded by very deep water and refraction generally will be limited. Accordingly, in specific situations the direction of wave approach relative to the orientation of the reef-shore should also be considered and, given the overall level of approximations involved, a simple transference is preferred, e.g.

$$H_{oS(\theta=0)} = H_{oS(\theta=\vartheta_o)} \cos\theta_o \qquad \text{(E.13)}$$

where υ_o is the angle between the offshore deepwater wave energy and the shore-normal reef profile.

Gourlay (1996b) also examined the variability in setup values due to the irregular wave condition (or *surf beat* phenomenon), utilising laboratory data from Seelig (1982, 1983) and Nielsen and Rasmussen (1990). These analyses considered the variability in water levels in terms of the standard deviation $\sigma_{\overline{\eta}}$ of setup magnitudes relative to the mean setup and also an estimate of the extreme setup level $\overline{\eta}_{1\%}$, i.e. equaled or exceeded only 1% of the time. Gourlay's analysis of these data sets showed that the relative values of $\sigma_{\overline{\eta}} / \overline{\eta}_r$ and $\overline{\eta}_{1\%} / \overline{\eta}_r$ were functions of the submergence *S* and the reef-top width W_r, and presented the relationships in graphical form, as shown in Figure E.3. This allows estimation of the wave induced water level z_L on a reef-top lagoon, variously:

$$z_L = z_o + \overline{\eta}_r \pm \sigma_{\overline{\eta}} \quad \text{(E.14)}$$

within the expected range of one standard deviation, or

$$z_L = z_o + \overline{\eta}_{1\%} \qquad \text{(E.15)}$$

an estimate of the extreme wave setup level; where z_0 is an offshore SWL datum which includes astronomical tide, inverted barometer effect and/or wind setup.

Depending on the reef-top characteristics, it may also be necessary to consider the possibility of re-formed waves and bores in the lagoon and additional beach setup and runup. Gourlay (1997) provides further advice on such matters.

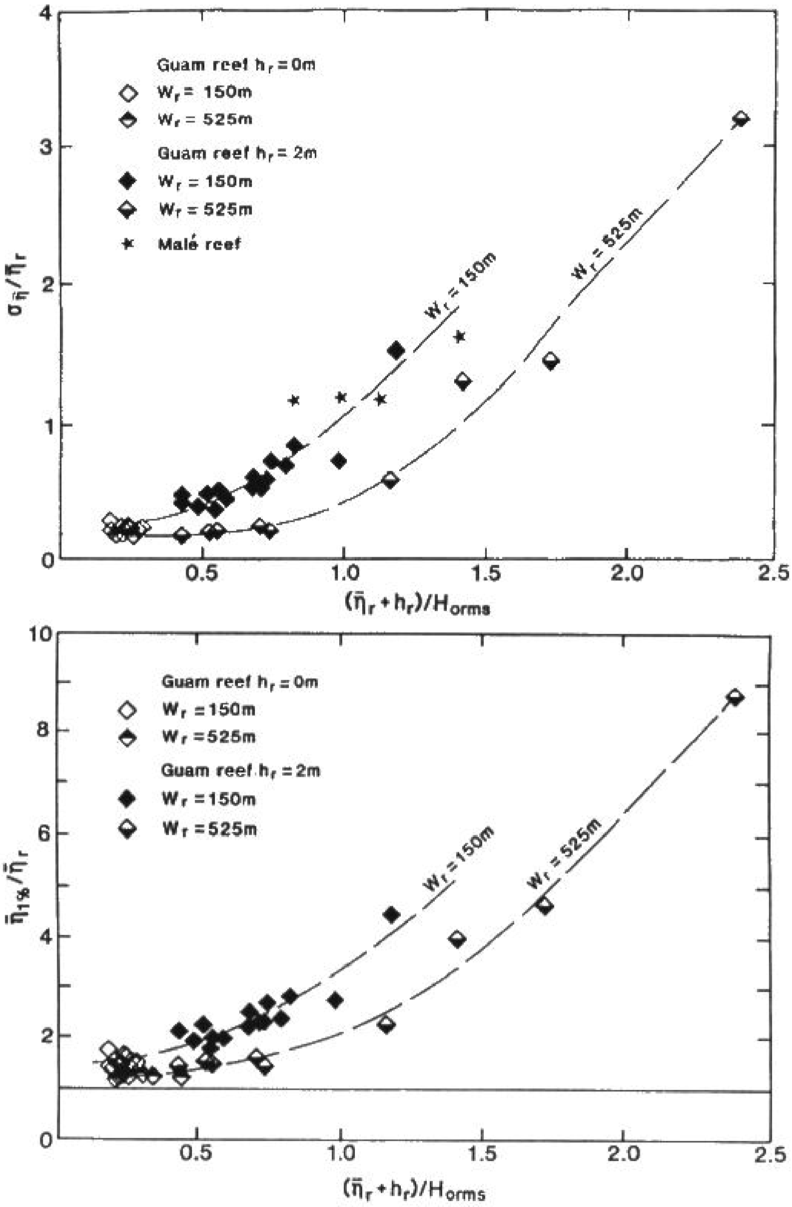


Figure E.3 Effect of irregular waves on reef-top wave setup. (after Gourlay 1996b)

E.4 References

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Appendix F

Tidal Harmonic Data for Rarotonga

Appendix F Tidal Harmonic Data for Rarotonga

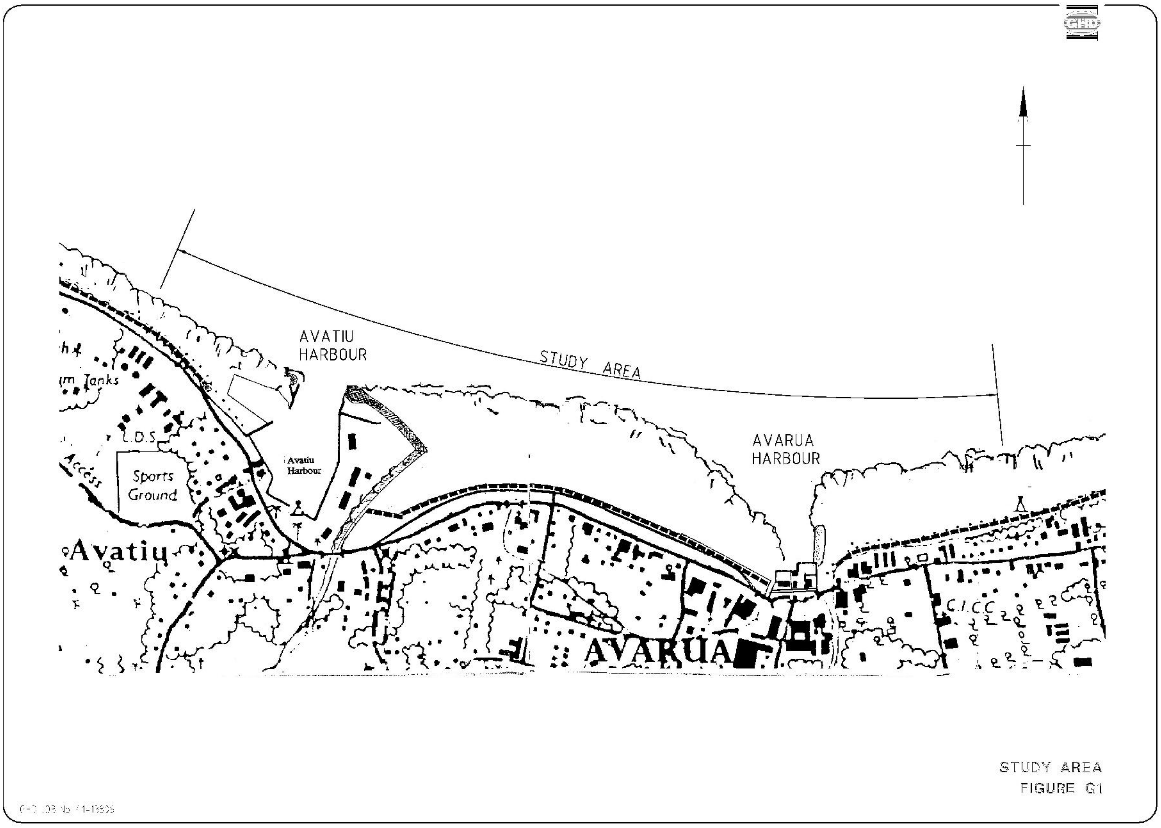
The following tidal harmonic data was supplied by the Australian Bureau of Meteorology National Tidal Centre.

Constituent	Amplitude	Phase
	m	deg
\Sa	0.03010	339.71190
\Ssa	0.02290	259.05430
\Mm	0.00640	357.54110
\MSf	0.00260	301.57110
\Mf	0.00650	54.08280
\Q1	0.00540	65.83510
\O1	0.02580	75.35810
\M1	0.00000	0.00000
\P1	0.00850	61.29870
\S1	0.00050	205.17160
\K1	0.02680	69.83400
\J1	0.00000	0.00000
\OQ2	0.00000	0.00000
\2N2	0.00940	153.77630
\mu2	0.01090	152.45780
\N2	0.06300	178.85890
\nu2	0.01160	183.67830
\M2	0.26770	209.79610
\L2	0.00570	247.18670
\T2	0.00570	283.56140
\S2	0.06980	292.03940
\K2	0.01840	291.26160
\2SM2	0.00000	0.00000
\MO3	0.00000	0.00000
\M3	0.00000	0.00000
\MK3	0.00000	0.00000
\MN4	0.00000	0.00000
\M4	0.00130	5.37720
\SN4	0.00000	0.00000
\MS4	0.00080	50.33380
\MK4	0.00000	0.00000
\S4	0.00000	0.00000
\2MN6	0.00000	0.00000
\M6	0.00000	0.00000
\2MS6	0.00140	167.82390
\2MK6	0.00000	0.00000
\MSK6	0.00000	0.00000

Datums		
Z0	=	0.56
HAT	=	1.07
MHWS	=	0.9
MHWN	=	0.76
MSL	=	0.562
MLWN	=	0.364
MLWS	=	0.225
ISLW	=	0.172
LAT	=	0.051

Appendix G

Site and Historic Photographs



Study Area

Photo 1: Avarua Harbour approx 15:00 January 1, 1987

Photo 2: Near Banana Court during cyclone eye approx 09:00 January 2, 1987

Photo 3: Avarua after cyclone during afternoon January 2, 1987

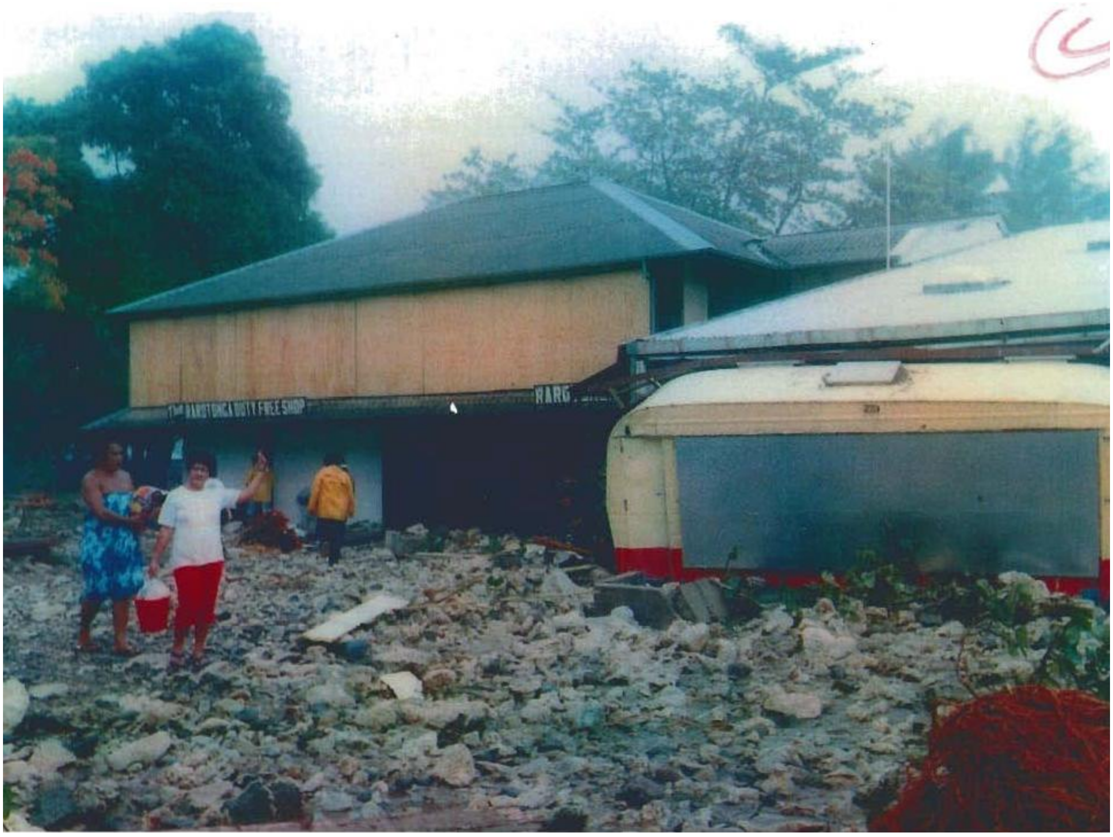
Avarua Hbr at 1500 Jan 1

Knee deep in water near a shop in the CITC / Banana Court area during the cyclone eye.

Shot after cyclone of Avatiu harbour showing a dozer clearing away rubble buildup

Avatiu after cyclone

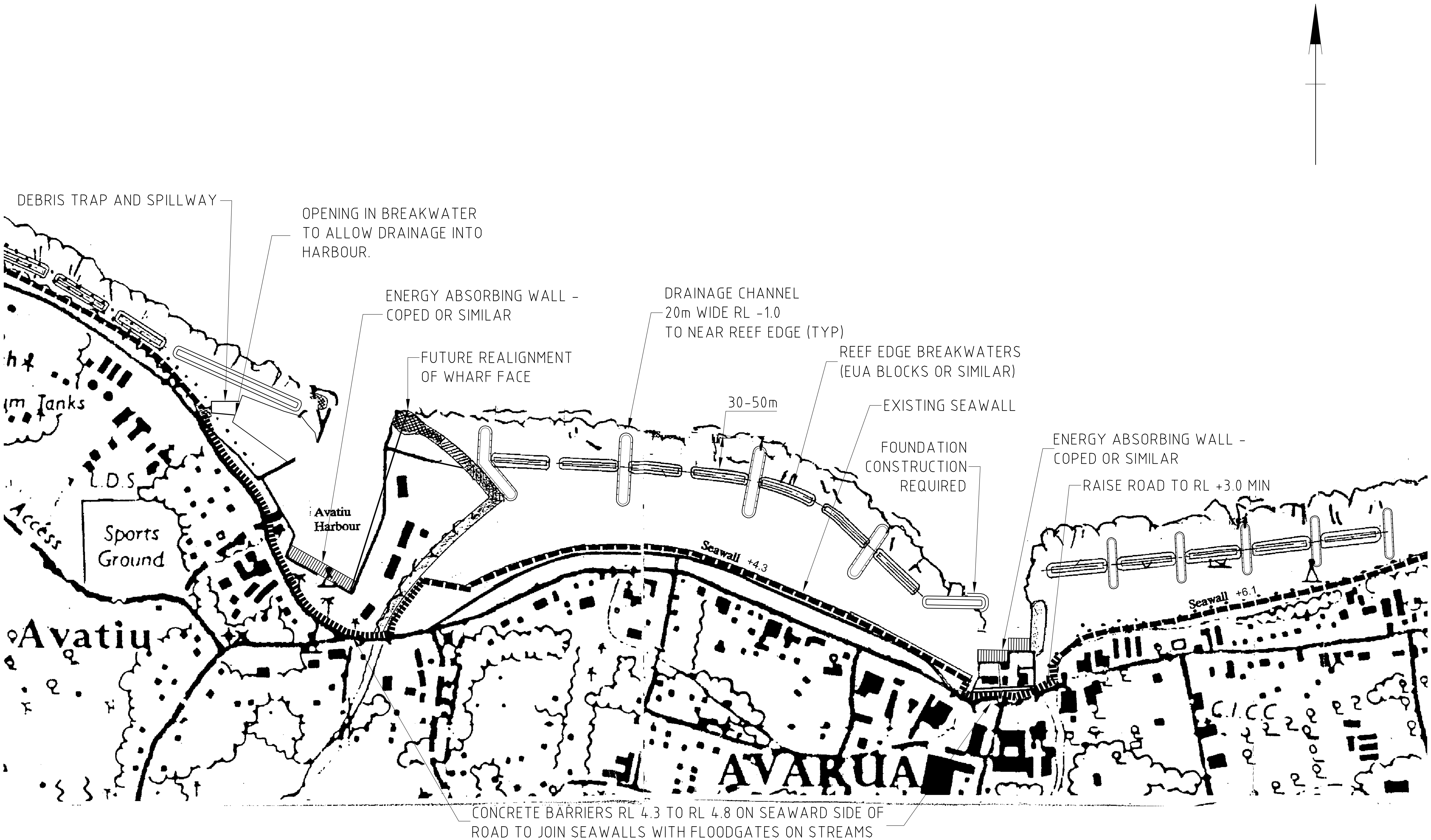
Rubble on higher section of road between Avatiu and Avarua



Typical damage from overtopping flows and rubble

Appendix H

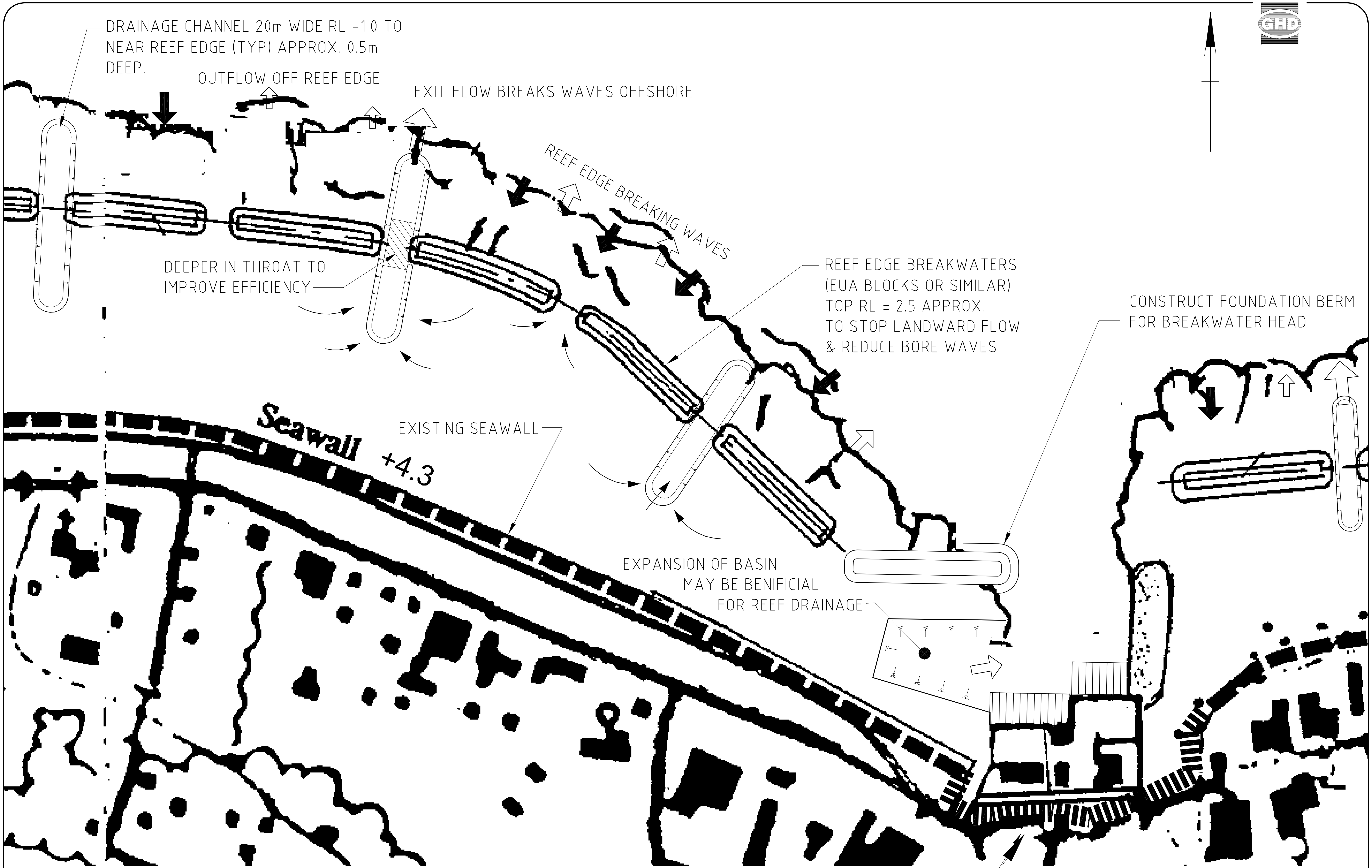
Drawings



TYPICAL LAYOUT OF COASTAL PROTECTION STRUCTURES

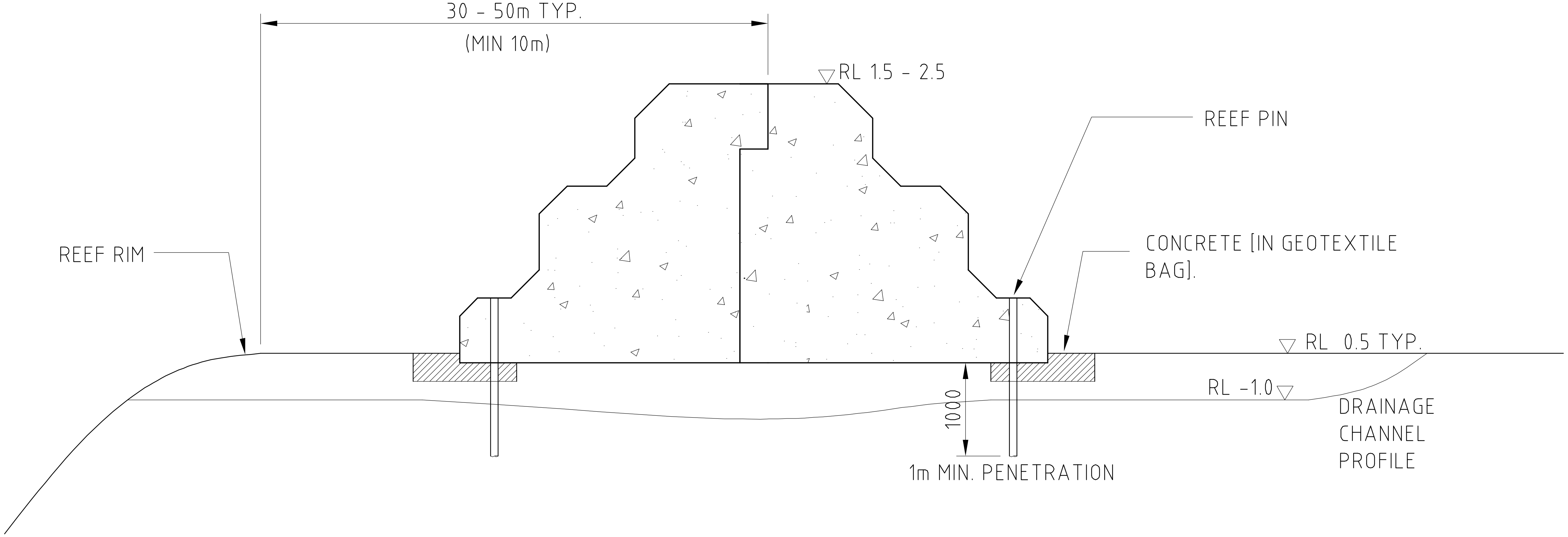
FIGURE H1

GHD JOB No. 41-13806

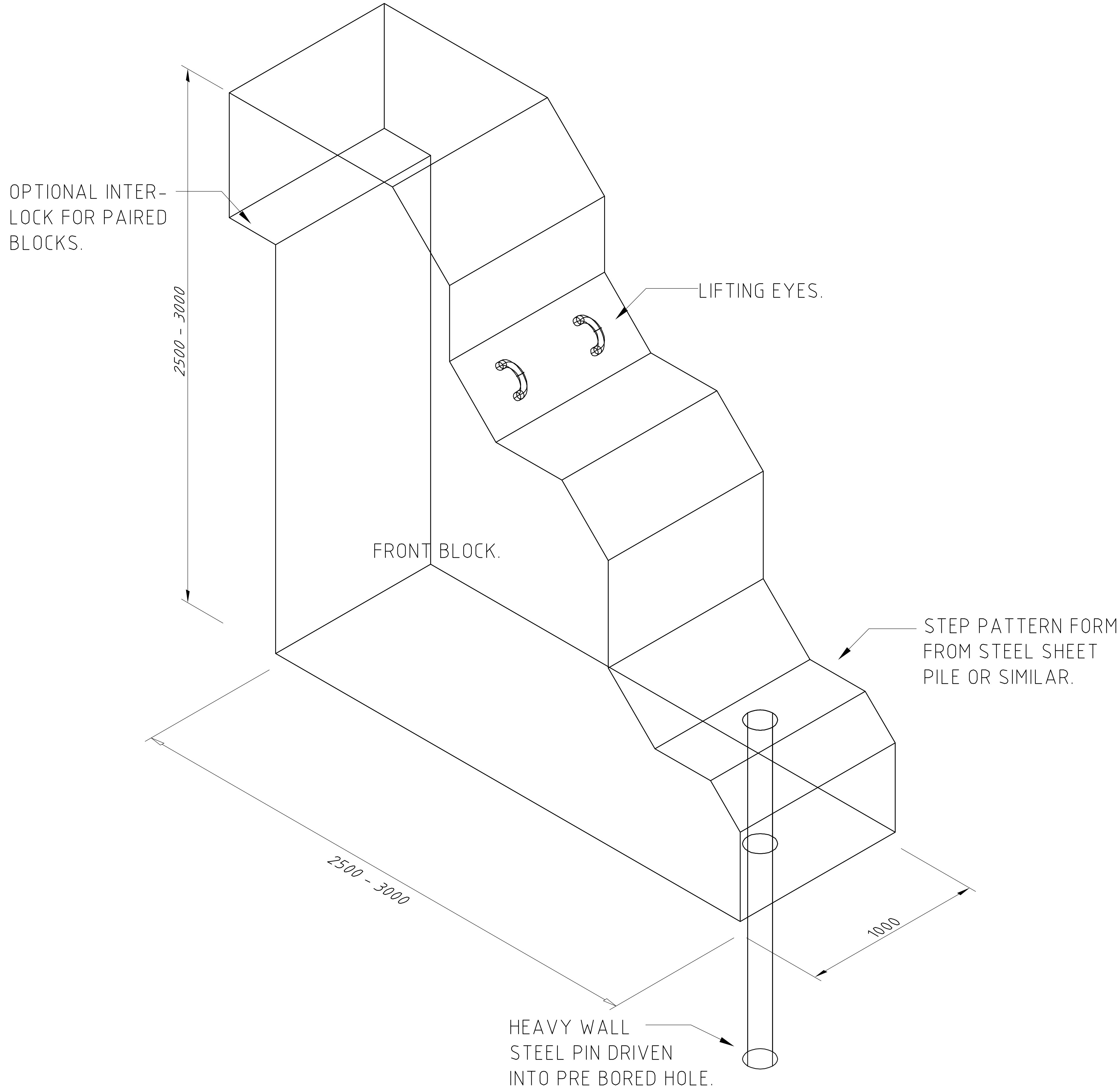


OPERATIONAL CONCEPT
FIGURE H2

GHD JOB No. 41-13806



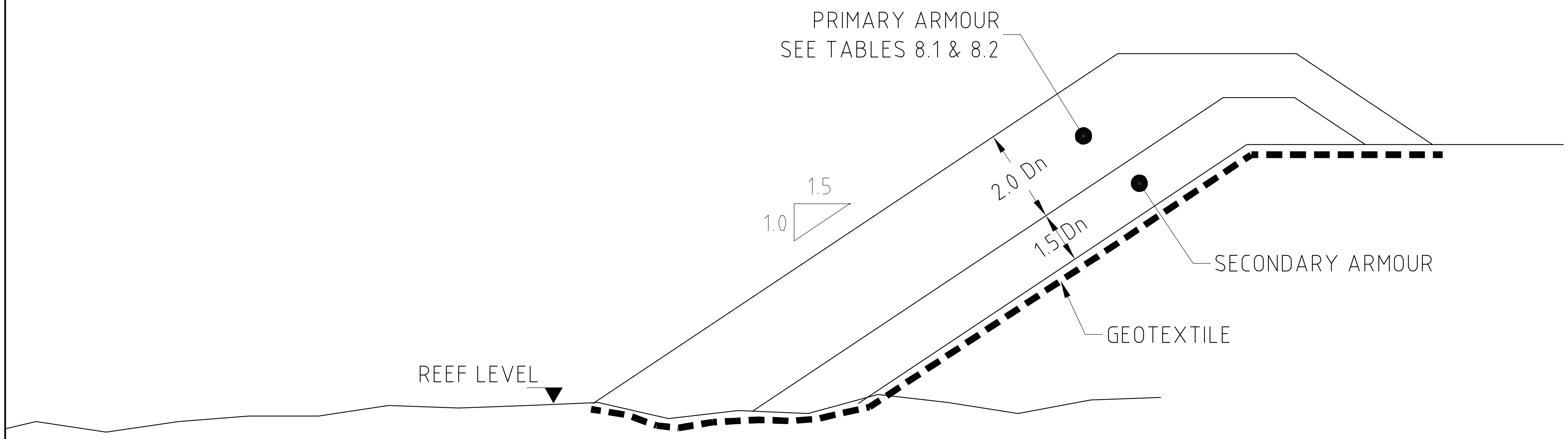
TYPICAL CROSS SECTION THROUGH REEF BREAKWATER.
FIGURE H3



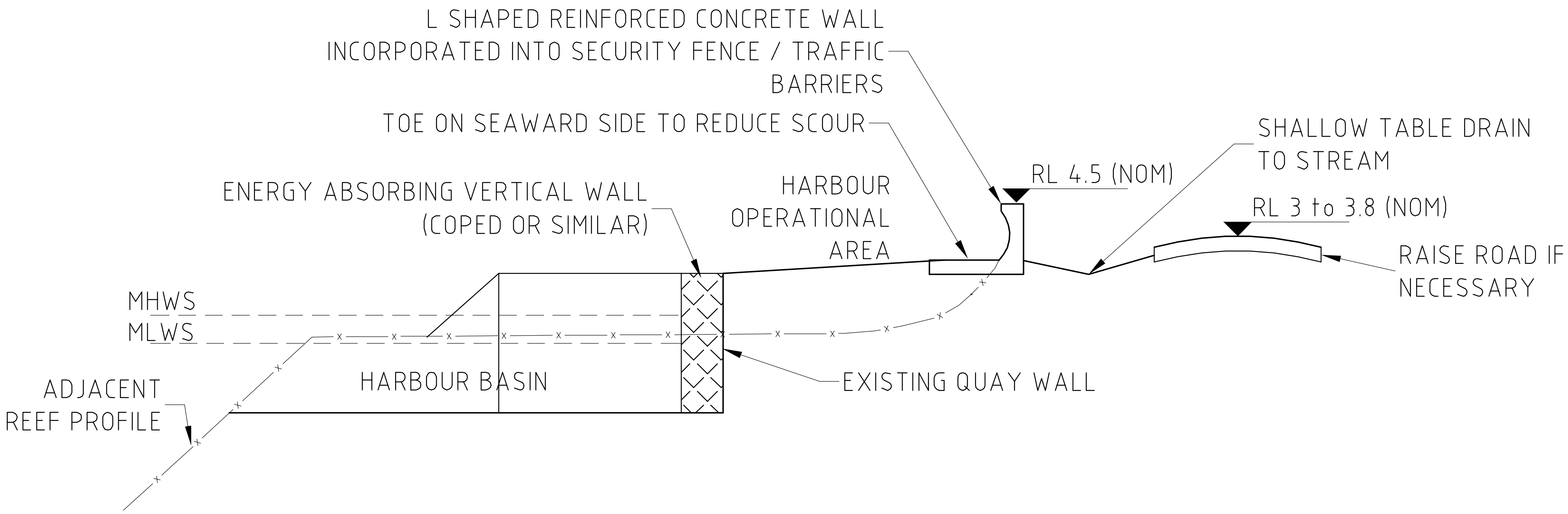
EUA BLOCKS

FIGURE H4

GHD JOB No. 41-13806



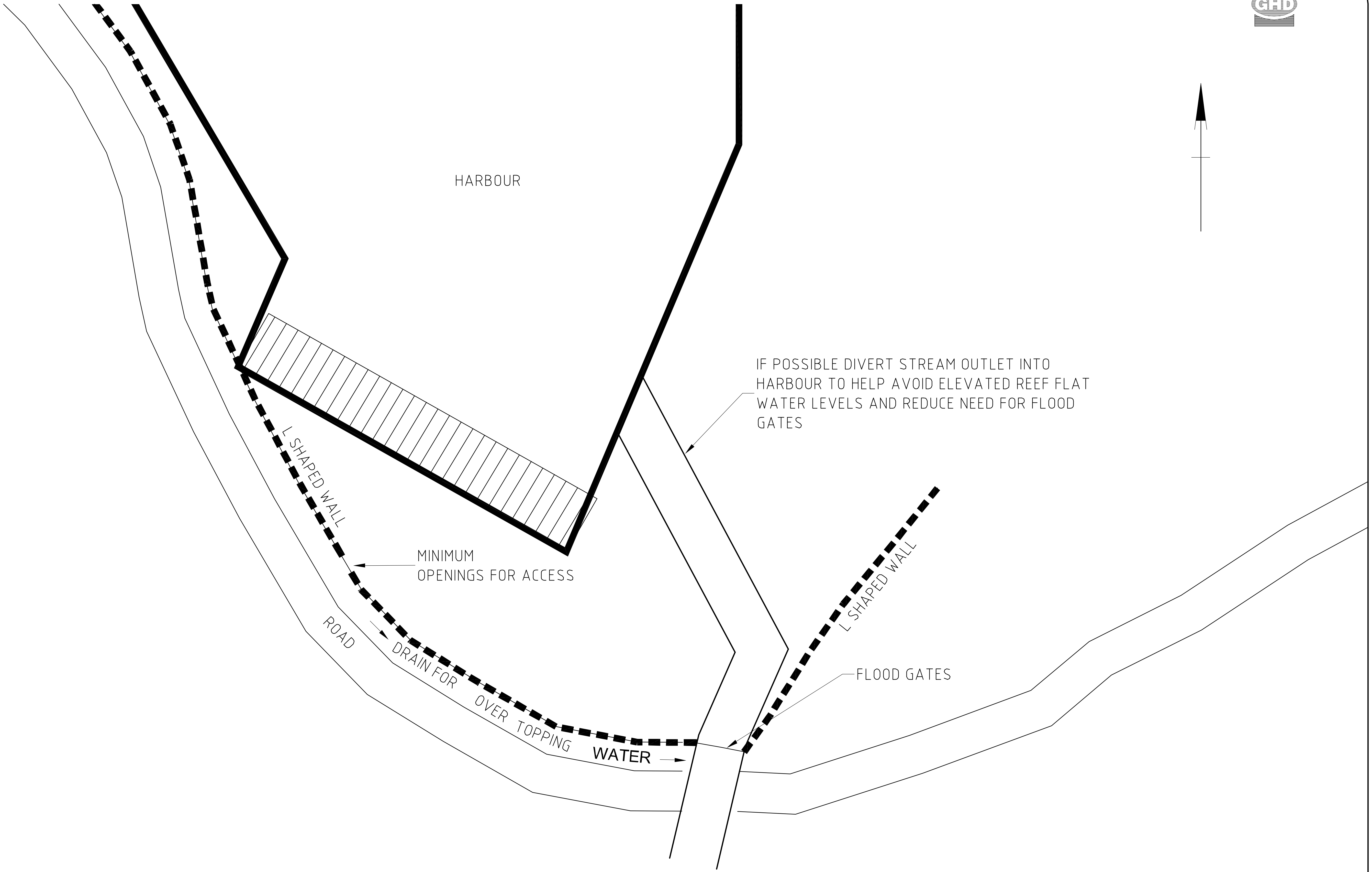
TYPICAL ROCK REVETMENT CROSS SECTION
FIGURE H5



TYPICAL SECTION THROUGH HARBOUR INLET.

FIGURE H6

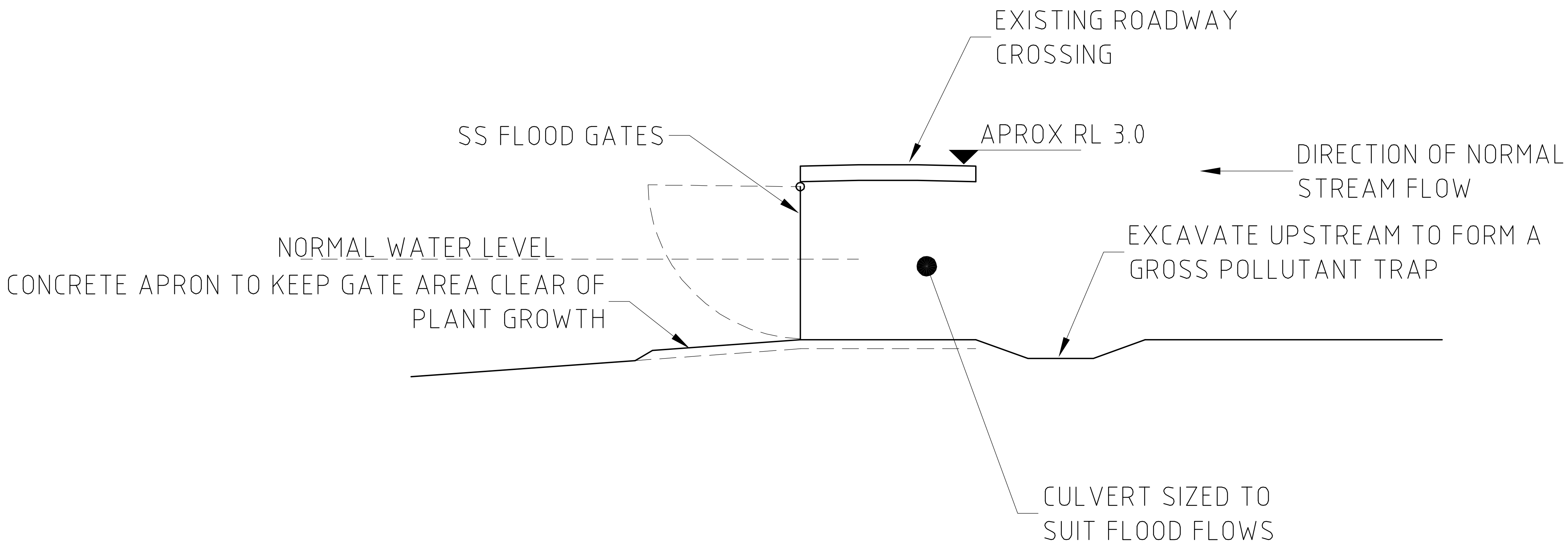
GHD JOB No. 41-13806



SCHEMATIC HARBOUR LAYOUT

FIGURE H7

GHD JOB No. 41-13806



SECTION THROUGH FLOOD GATE.
FIGURE H8

Appendix I

Photographs of Coastal Structures

Nafanua Harbour, 'Eua, Tonga. South Breakwater is on right hand side of image

South Breakwater Nafanua Harbour, 'Eua showing flow of reef setup water between end of breakwater and shore. (Closure of this gap raised reef flat water levels by approx 0.4m during normal weather

West Entrance Breakwater, Touliki Harbour, Nuku'alofa, Tonga, showing front profile of 'Eua Blocks (Tip constructed on rubble foundation replacing weak substrate)

Dozer ripping cemented reef flat in preparation for placing concrete blocks, South Breakwater, Nafanua Harbour, 'Eua

Seabee wall at West Is, Cocos (Keeling) Is

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