

DESIGN SEA LEVELS FOR SOUTHERN AFRICA

A PROBABILISTIC APPROACH

BY

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ABSTRACT

This thesis describes the development of a methodology for the quantitative assessment of design sea levels for southern Africa. In order to achieve this objective it was necessary to establish which ocean processes affected sea level in the sub-continent and develop a probabilistic model for the combination thereof. The methodology, is used to characterize regional design sea levels in terms of the west, south and east coasts. A site specific application is undertaken to demonstrate the model's capabilities with regard to the design of depth limited structures. The ultimate objective of this study is to provide a practical approach to the quantification of the sea level component of loading in the full probabilistic design assessment.

Data analysis considers all available sea level and wave data for three ports around the coast. A three parameter threshold analysis technique is used to define independent identically distributed events. The distinction between the major processes affecting sea levels in southern Africa may be related to the differences in both the time and space scales of their response to the forcing mechanisms. The data analysis procedure is used to defined the primary statistical characteristics of the observed events in each data set as they relate to sea level. A stochastic simulation model is developed which reproduces a synthetic hourly sea level record displaying the same statistical characteristics as the observed data. Annual maximum values are extracted from the model output with a view to estimating extreme sea levels. The model may be run over any number of periods until satisfactory convergence in the results is obtained. The theoretical basis of the model is described and the results compared with the Gumbel method.

A regional assessment of design sea levels for southern Africa indicated that the south coast experienced larger fluctuations in the stochastic component of sea level than the east and west coasts. Sea levels throughout the sub continent are primarily affected by tide, shelf waves, wind waves and edge waves. These processes were found to be statistically independent of one another for the areas evaluated. Design sea level would appear to be determined by a combination of a number of moderate magnitude events rather than one single process. The application of the model illustrates the importance of considering both wave height and sea level conditions as stochastic variables for the design of depth limited structures. The relative influence of stochastic sea level is shown to increase from deep to shallow water.

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ABBREVIATIONS

AFDA	Advanced Full Distribution Approach
ANSI	American National Standard Institute
CDF	Cumulative Density Function
CERC	Coastal Engineering Research Centre
EPM	Exceedence Probability Method
IAEA	International Atomic Energy Agency
JPM	Joint Probability Method
PDF	Probability Density Function
RJPM	Revised Joint Probability Method
SAN	South African Navy
WIS	Wave Information Study

CHAPTER 1

INTRODUCTION

The need for a quantitative approach to design sea level was initiated by work undertaken for the nuclear power industry in South Africa. Coastal nuclear power stations require extensive safety assessments as part of their qualification process. Design sea levels represent an important component in the safety analysis procedure in respect of coastal flooding, safety related cooling water supply and the reliability of the associated coastal structures. Two major limitations were evident in the conventional engineering approach to design sea levels for the sub continent. The first problem related to the lack of definition regarding the natural processes affecting sea level fluctuations in Southern Africa. The second factor concerned the actual techniques used to combine tides and storm surge levels and consequently determine extreme design levels. The primary objective of this study became the development of a quantitative approach or methodology for design sea levels in Southern Africa. This methodology could then be used to undertake full probabilistic design and hence realistic economic assessments for coastal structures.

It became clear as the study progressed that not only safety related structures, such as power stations, but also many conventional coastal engineering structures would be sensitive to design sea level. Of particular interest would be structures designed for depth limited wave conditions. Depth limited conditions relate to the situation where the design wave height for the structure is a function of the water depth at the wave breaking point. Conventional design practice has, in the past, made the assumption that water depth is constant at mean sea level or mean high water spring. It will be shown in this thesis that this assumption can be incorrect resulting in under or over design. The design engineer has as a result had to rely on engineering judgement and past experience to overcome this limitation.

It should be expected that recent advances in our understanding of structural response and failure will, in future, be incorporated into coastal engineering practice. Many of these new techniques require more detailed information pertaining to the statistical nature of the load components. This study should be seen as a first step towards quantifying the loadings related to design sea level.

The natural processes affecting sea levels in Southern Africa can be categorized into astronomical tides, meteorologically induced phenomena and tsunamis. Conventional engineering practice refers to tides, storm surges, wind setup, wave setup and drawdown as phenomena affecting design sea level. It is important within the context of Southern Africa, to differentiate between the different processes in order to gain insight into the statistical characteristics of the individual phenomena. It will be shown that the sub continent is primarily affected by astronomical tides, shelf waves, wind induced or gravity waves and long period edge waves. Up until this point, shelf and edge waves have not been considered in any detail in engineering design.

The thesis has been divided into six broad categories comprising nine chapters. The first four chapters are essentially of an introductory nature. The review of the literature represents an attempt to develop an underlying reference framework for the study. Papers described in this section have been limited to those having a direct bearing on the thesis. Chapter 3 develops the notion of engineering design with a view to illustrating the relative importance of design sea level. The processes affecting sea level fluctuation are described in chapter 4. Attempt is made, in this chapter, to identify the most important or dominant processes of the region with a view to narrowing the scope of the data analysis required.

The analysis of processes affecting design sea level is undertaken in chapter 6. The work concentrates on three ports located on different coastlines around South Africa. Data were collected for Richards Bay, Mossel Bay, and Port Nolloth. A three parameter threshold analysis is used to establish the statistical characteristics of each process for the ports under consideration. Some attention is given to the possible interaction between different processes along the coast.

Chapter 7 describes the development and verification of the stochastic simulation or convolution model. The purpose of the model is to combine the probability functions for the various causative processes into one single distribution function for design sea level. The individual components of the model are tested independently and then in combined form in the convolution model.

The application of the model is described in chapter 8. A regional assessment of maximum and minimum design sea level is undertaken to establish overall trends and characteristics. A more detailed case study for the Mossel Bay breakwater is undertaken to compare the model with conventional engineering approaches.

The thesis is concluded with a discussion of the results emanating from the model and a statement of areas of future research with regard to design sea level in southern Africa.

CHAPTER 2

REVIEW OF THE LITERATURE

This chapter describes all the literature which was found relevant to this thesis. Only papers of major interest will be discussed here, secondary publications will be referred to in subsequent chapters. A brief review of existing historical and process related information pertaining to extreme sea level events on the Southern African coast is given. This is followed by a discussion of literature relating to quantitative approaches for the assessment of design sea level. Finally some attention is given to published methods used to assess sea levels, with particular reference to applying full probabilistic approaches.

The earliest account of flooding refers to an article by Von Buchenroder (1830) where the inundation of parts of Robben Island in 1809 are mentioned. Gill (1883) describes a tsunami measured at Port Elizabeth in August of that year as result of the eruption of Krakatoa. In this instance a tsunami of 760 mm was measured. On the 4th of September 1883 an event at Port St Johns was registered where the water level was measured as being 2 m above the high water mark (Cape Times 1883). An article in the Cape Times on the 25th of May 1960 discusses abnormal measurements at Hermanus relating to the Chilean Earthquake of that same month. Most articles obtained refer to wind induced storm damage on land with little emphasis on the coastal impact.

The phenomena affecting sea level fluctuations have been documented, in respect of Southern Africa, by (AEC (1989)). The most significant can be listed as the astronomical tide, continental shelf waves, wind wave storms, resulting in wave setup and runup, edge waves and tsunamis. The assessment of astronomical tide in Southern Africa is undertaken by the S.A.Navy Hydrographer. Bosman (1989) describes the methods used by this authority. Up until 1957 this function was performed by the British Admiralty. This study makes use of the tidal analysis and prediction model developed for the TOGA Sea Level Center (Cadwell and Kilonsky (1988)). The model is described in more detail by Foreman (1977). Earlier studies have made use of the Doodson tide filter described by Doodson and Warberg (1941) to assess tidal residuals by removing the primary solar and lunar semi-diurnal and diurnal components. This approach presented some problems with regard to the determination of true hourly tidal values and was therefore not used in this thesis.

Continental shelf waves or coastal trapped waves are described by De Cuevas (1985). De Cuevas used the Doodson filter on available tide gauge data from 1980–1985 to assess the nature of these phenomena. This work indicated that the daily mean sea levels would be expected to fluctuate by up to 500 mm over a period of between 2–20 days.

Wind waves generated around the Southern African coastline have been described in numerous publications SAN (1975), Swart and Serdyn (1981), Shillington (1984), Rossouw (1984) but most comprehensively by Rossouw (1989). Rossouw reviewed all existing waverider and directional VOS data up to 1986 with a view to developing a systematic approach to design waves in the region. A significant part of this work was aimed at fitting the available measured data to given distributions in order to extrapolate extreme events. Button (1988) using much of the same data for the Cape South coast produced various stochastic models for significant wave height (H_{m0}) and zero downcrossing waveperiod (T_z). The result was a number of stochastic models which could produce simulated H_{m0} and T_z values with the same characteristics as the original measured data. The significance of wind induced waves with regard to this study relates to the nearshore transformation in terms of wave setup and wave runup. Both these topics are discussed in more detail in chapter 3.

Long period edge waves off Southern Africa were first discussed by Darbyshire (1963) and Darbyshire (1964). These events were called shelf waves in her paper but essentially conform with the description given by Shillington (1985) for edge waves. Both authors relate this phenomenon to observations made by Munk et al (1956) in Southern California. Long period edge waves are described as events with a wave height up to 1 metre and wave periods varying between 5–60 minutes. These events are associated with concurrent air pressure oscillations which take place in association with the sea wave. Shillington (1985) set out in some detail a viable explanation and model for Southern Africa.

The occurrence of tsunamis in Southern Africa would appear to be rare. Gill (1883) describes the tsunami measured at Port Elizabeth in August of that year as a result of the eruption of Krakatoa. The wave height as measured from the marigram was 760mm. Dames and Moore (1979) reviewed tsunami hazard as part of the safety

assessment for Koeberg Nuclear power station. The credible maximum tsunami for design purposes was set at +2.25m GMSL. Due to the similarities between the marigrams for tsunamis and edge waves some confusion arose regarding these two phenomena. Wigen, Murty and Philip (1981) ascribe an event on the 11th of May 1981 on the Cape south coast to submarine slumping whilst Shillington (1985) shows that this event is more readily explained by edge waves activity. Wijnberg (1988) reviewed tsunamis in Southern Africa with a view to assessing their potential coastal impact. The general conclusion emanating from this work was that in spite of their rare occurrence tsunami generating mechanisms do exist, which affect the sub-continent, and these waves should not be ignored when assessing sensitive coastal structures, such as nuclear power stations.

The impact of local wind storms on coastal sea level stand is dealt with briefly by De Cuevas (1985). Due to the relative steep coastal profile wind setup would not appear to be significant for all but a few shallow coastal embayments. Tropical cyclones are mentioned by SAN (1975) as occurring infrequently on the Natal north coast. No published information could be traced relating to an associated sea level rise. Most damage reported in press reports relates to wind induced destruction and not inundation of coastal areas.

A rational approach to sea level requires that all the aforementioned processes be integrated in an unbiased fashion when assessing design levels. WHP (1976) and CSIR (1987) represent typical engineering assessments undertaken in the past to determine an extreme design sea level. Individual processes are identified, data assimilated, fitted to suitable distributions and extrapolated to determine extreme values. These extreme values are then combined as statistically independent or dependent processes to determine combined probabilities of exceedence and associated return periods. CERC (1984), Muir-Wood and Flemming (1981), IAEA (1983), ANSI (1981) and Bruun (1984) provide accepted guidelines for the determination of design sea levels for coastal engineering projects. Whilst these guidelines provide the overall governing principles, they do not discuss the practical problems relating to data assessment and the combination of processes. Several publications in the last fifteen years have discussed some of these aspects. Blackman and Graff (1978) analyzed observed annual sea level for ports in southern England with a view to estimating the probability of exceedence

and return periods for these areas. Problems relating to the method of analysis were discussed, however no consideration was given to the underlying causative processes. In an attempt to improve this situation, Pugh and Vassie (1978) presented the joint probability method (JPM) whereby astronomical tide and storm surge were analyzed separately. Once the probability distribution for both tide and storm surge had been determined, they were combined as statistically independent processes to provide a new joint probability distribution for sea level. The major short coming or limitation with regard to this method was that no account was taken of the dependency structure of the hourly data used. In spite of these limitations this method represented a major advance on the Gumbel (1954) approach used up till then, as different processes were assessed as independent physical phenomena. In a similar vein, Middleton and Thompson (1986) reviewed existing approaches used for defining return period for extreme sea levels and presented, using statistical theory based on Rice (1954), the exceedence probability method (EPM). This method eliminated the problems associated with the dependency structure of hourly surge measurements. Hamon and Middleton (1989) demonstrated a practical application of this method using data from Sydney, Australia. The proposed benefit of using this approach was that short duration records (1 year) could be used to predict 1:50 year return periods.

Tawn (1988) pointed out that the major weakness with the (EPM) approach was that it modelled the whole process and not just the extremes, that it made highly restrictive assumptions and did not give good results in application. Tawn (1988) then refined the joint probability method (JPM) (Pugh and Vassie (1978)) and developed the revised joint probability method (RJPM). The main advantage of this approach was that it accommodated the 1-dependency structure of the storm surge data. Tawn and Vassie (1989) and Tawn and Vassie (1990) discuss the method, as proposed for engineering application and the spatial transfer of extreme data for different ports, respectively.

If design conditions are to be adequately described it is necessary to include the effects of wave setup and runup at the coast. Both these phenomena are driven by short period wind waves. Vrijling, Jansen and Bruinsma (1983) proposed a probabilistic method to predict wave and storm surge conditions using a time series correlation analysis on existing data. A relationship was derived between storm surge and wave

energy which was presented as a conditional probability distribution. Ackers and Ruxton (1976) had followed a similar approach in their study on extreme levels affecting the Essex coastline by calculating the combined probabilities of extreme high sea levels and high waves and their simultaneous occurrence. The treatment of tide and surge as statistically independent processes was in a sense a forerunner to the work by Pugh and Vassie (1979) but contained the same limitations. The incorporation of wave effects make this paper most relevant to the work undertaken later in this thesis.

A number of papers illustrated the use of probability theory in the determination of coastal engineering design parameters. Manoha, Bernier and Graff (1986) present a statistical method for the estimation of extreme wave effects, using the theory of renewal processes, as required by the nuclear power industry for coastal locations. In short their approach uses a partial duration series method to increase the data set of significant events in order to reduce the uncertainties relating to the annual maximum values. This approach shows much promise due to its fundamentally sound statistical basis regarding the analysis of wave heights and storm characteristics. Smith (1988) followed a similar approach when investigating the duration of extreme wave conditions based on 20 years of hindcast data obtained from the US Army Engineer Experiment Station. An important conclusion emanating from this study was that no significant relationship could be found between peak storm intensity (as measured by H_{m0}) and the duration of an event. Ochi, Mesa and Lui (1988) undertook a similar study to estimate extreme sea severity (50 and 100 year H_{m0}) from measured daily maxima. Whilst various extreme type distributions were fitted to the data set, using the maximum likelihood, the skewness and the non-linear multiple regression methods, it would appear that the authors did not account for the dependence structure. Rossouw (1989) undertook similar work using a larger data set and concluded that the effect of dependence was insignificant. Similar to Ochi et al (1988) it was found that the extreme type I distribution best fitted the data. Other relevant works with regard to statistical modelling of wind waves are Burrows and Salih (1986), Deo and Burrows (1986) and Salih, Burrows and Tickell (1988). The last paper illustrates the use of partial duration series analysis on wave climate data to evaluate peak storm wave height and duration above a specified threshold. Of importance, is the testing of a Markov model to characterize storm statistics. Kimura (1988) uses a similar approach to evaluate the maximum run of irregular waves.

Jensen and Klinting (1988) discuss the significance of abnormal storms and their impact on design parameters. This is an important paper in that it reiterates the importance of evaluating the underlying physical processes to ensure that the data being analyzed belongs to the same statistical set. Once again a partial duration series or threshold analysis is used to evaluate the data set, in order to isolate independent storm events. Furthermore it is assumed that the occurrence of wave peaks takes place according to a Poisson process. European data are used to illustrate the importance of separating the data into different distributions based on the underlying processes. An important conclusion is that most abnormal storms do not relate to the normal statistics at a particular location.

More recent work by Goda and Kobune (1990) and Andrew and Hemsley (1990) has focused on methods to select suitable distribution curves for extreme value predictions. The former uses rejection and acceptance criteria for the choice of distributions whilst the latter applies a bootstrap resampling approach. A more generalized paper by Castillo and Sarabia (1992) proposes three methods for selecting limit distributions based on the available data set, namely the classic probability paper method, least-squares method and the curvature method.

As design methodologies have developed it has become necessary to incorporate more information into assessment models. Information such as storm duration, persistence of calms, wave groupiness and the periodicity. Sunder, Angelides and Conor (1979) proposed a stochastic model for the simulation of a non-stationary sea. The model was developed for the assessment of long term degradation in soil and structural properties of offshore structure – foundation systems. The model uses significant wave height, above a pre-defined threshold to define sea state. Duration, an intensity and a non stationary random process for tracing wave height evolution, represented by a Fourier transformation, are used to describe storm events. Similarly Button (1988) considered a number of stochastic models for simulating significant wave height and zero downcrossing wave period for the available data on the South Cape coastline. Whilst different models seemed to perform well in particular cases it would appear that no generalized model which dealt with all wave characteristics could be found. Scheffner and Borgman (1992) developed a stochastic model for the representation of wave height, wave period and direction. The work was developed with a view to assessing

dredging disposal sites. The method uses finite length wave records to compute a matrix of coefficient multipliers which are used to simulate wave data which reflects the primary statistical characteristics of the original data set. The model was demonstrated using the Wave Information Study (WIS) data for the Gulf of Mexico. Apart from the primary statistical properties being modelled, the authors were able to include seasonal patterns and wave sequencing. The principles discussed in the last three papers are used for the development of the simulation model in this study.

The papers discussed in this chapter form the basis of the work considered relevant to this thesis. Whilst the broadest possible spectrum of references were consulted, not all papers relating to the topic could be obtained. Chapter 2 has set out the underlying reference framework of the study in terms of existing published information. Chapter 3, which follows, explores the engineering perspective of this study in more detail.

CHAPTER 3.

DESIGN SEA LEVEL: A COASTAL ENGINEERING PERSPECTIVE.

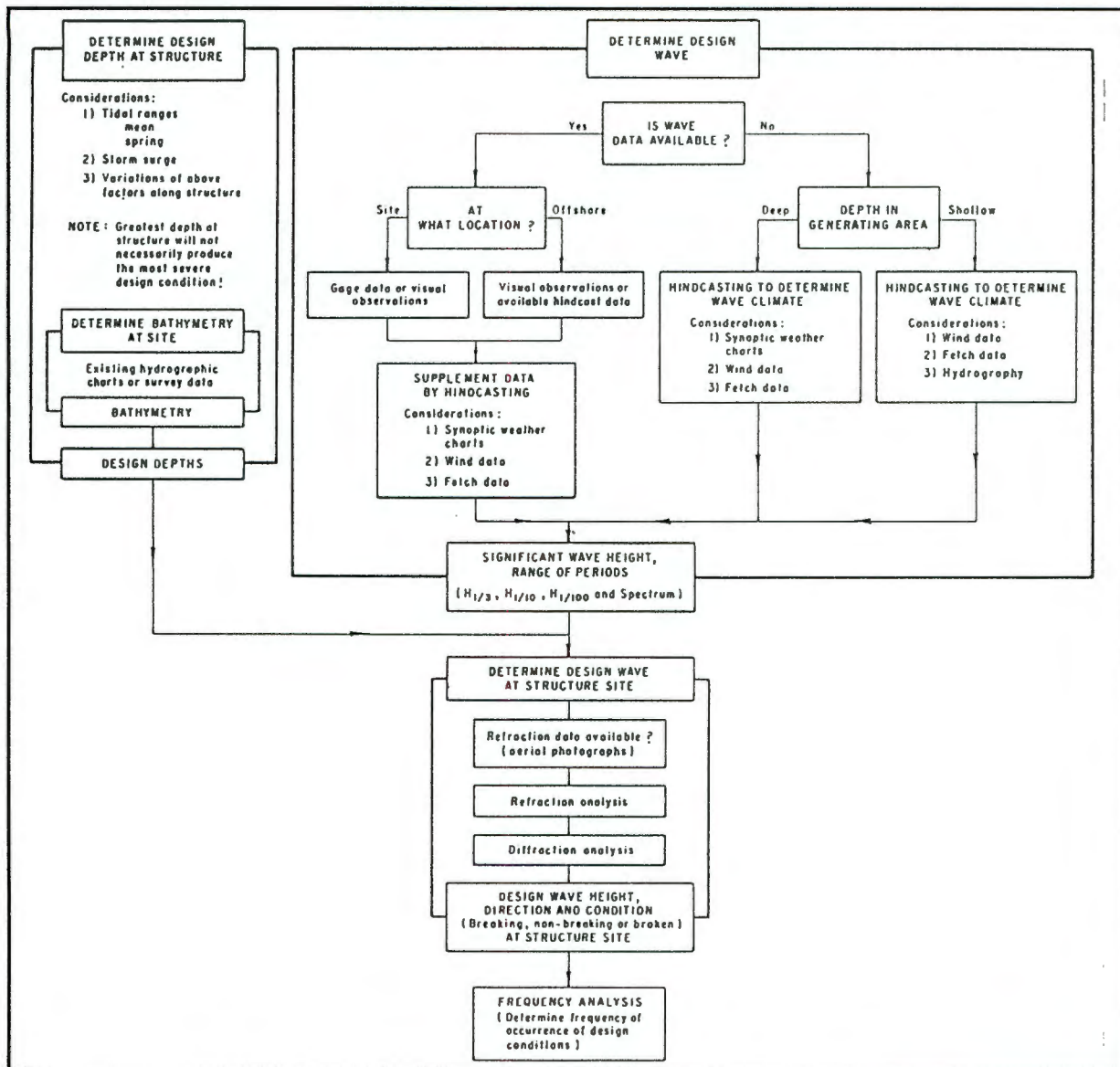
Coastal engineering design can be seen as a procedure by which a structure is conceptualized and developed to fulfill specific functional, structural, environmental and economic requirements. A fundamental component of this design procedure is the quantification of the particular coastal environment within which one is operating. This may be referred to as the establishment of the design conditions. Whilst this process may be complex, it is possible to simplify the procedure in the form of a logic diagram as illustrated in figure 3.1.

It is common in coastal engineering practice, to consider the determination of the design depth of the structure and the design wave independently. Furthermore, whilst a significant amount of effort has been expended on the probabilistic assessment of design waves, relatively little work appears to have been carried out with regard to the assessment of design water depth. In many instances water depth is treated in a deterministic or quasi-statistical manner. In order to provide a more rational quantitative basis to design it is proposed that combined or joint conditions be evaluated. Thus both water depth and wave height are assessed as stochastic variables and their combined probability used to characterize the environmental design conditions.

The importance of using a design condition approach, with regard to water depth and wave height, becomes most relevant when structures are assessed for depth limited conditions. Depth limited conditions refer to the situation where the magnitude of the design wave is determined, and thus limited, by the water depth in front of the structure. This is commonly referred to as breaking wave or broken wave conditions. A broad spectrum of structures are designed for these conditions, for example shore protection works, breakwaters, pipeline shore crossings, sand bypass schemes and shallow water outfalls.

The relative significance of water depth increases in shallow water ($d/gT^2 < 0.01$) due to the direct relationship between breaking wave height and breaker depth. Under deep

FIGURE 3.1



water conditions ($d/gT^2 > 0.1$) wave height and water depth (or sea level super-elevation) are generally considered as independent. CERC (1984) presents two graphs based on the work by Weggel (1972) and Goda (1970) which define a relationship between breaking wave height and breaker depth for known deep sea conditions (see figures 3.2. and 3.3.). Gunbak (1977) c.f. Bruun (1985) summarized a number of studies and presents a relationship between breaker index (H_b/d_b) and the similarity coefficient ξ_b , also known as the Iribarren number. (see figure 3.4.).

where $\gamma_b = H_b/d_b =$ breaker index

$$\xi_b = \frac{\tan \alpha}{\sqrt{H_b/L_b}}$$

and $\alpha =$ bed slope
 $H_b =$ breaking wave height
 $L_b =$ wave length
 $d_b =$ breaker depth

It can be seen that γ_b increases as ξ_b increases. Hence steep bed slope and relatively small wave steepness will result in larger breaker index ratios. As the breaker index increases in magnitude so the relative importance of water depth as a determining variable of design wave height increases. It is this interdependency between two stochastic variables which is not normally addressed in conventional design procedures.

The concept of depth limited design may be explored further by considering the preliminary design calculation for rubblemound breakwaters and particularly the theoretical influence of a variable sea level. These formulae will be used in chapter 8 to test the application of the methodology developed in this thesis. Rubblemound breakwaters are normally assessed in terms of the size of the primary armour units required to maintain overall structural integrity at accepted damage levels.

Bruun (1985) lists general stability formulae used to determine armour unit size (see table 3.1. CERC (1984) proposes the use of the Hudson formula, which can be written as follows :

FIGURE 3.2.

BREAKER HEIGHT INDEX VERSUS DEEPWATER WAVE STEEPNESS

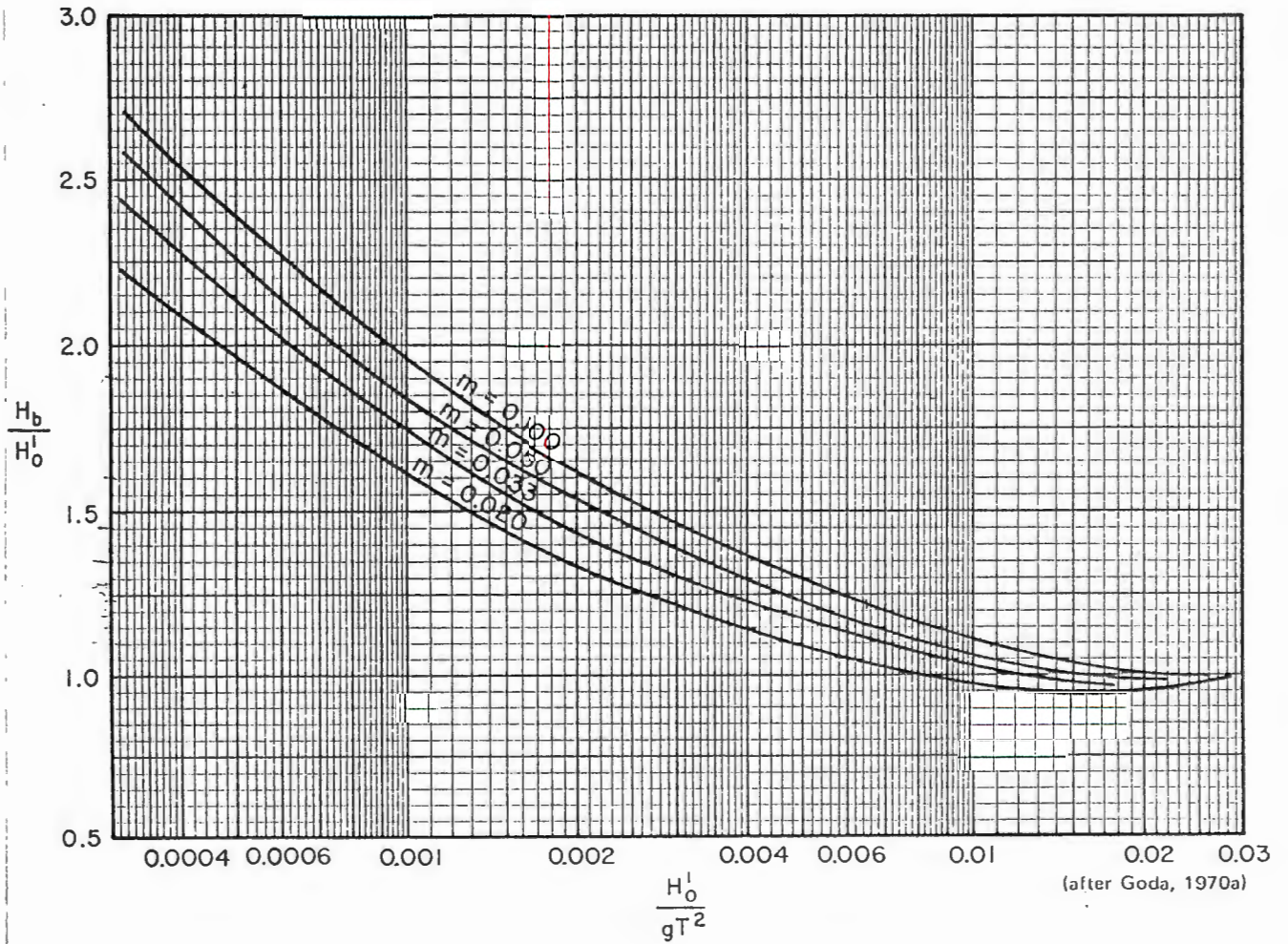


FIGURE 3.3.

α AND β VERSUS H/gT^2

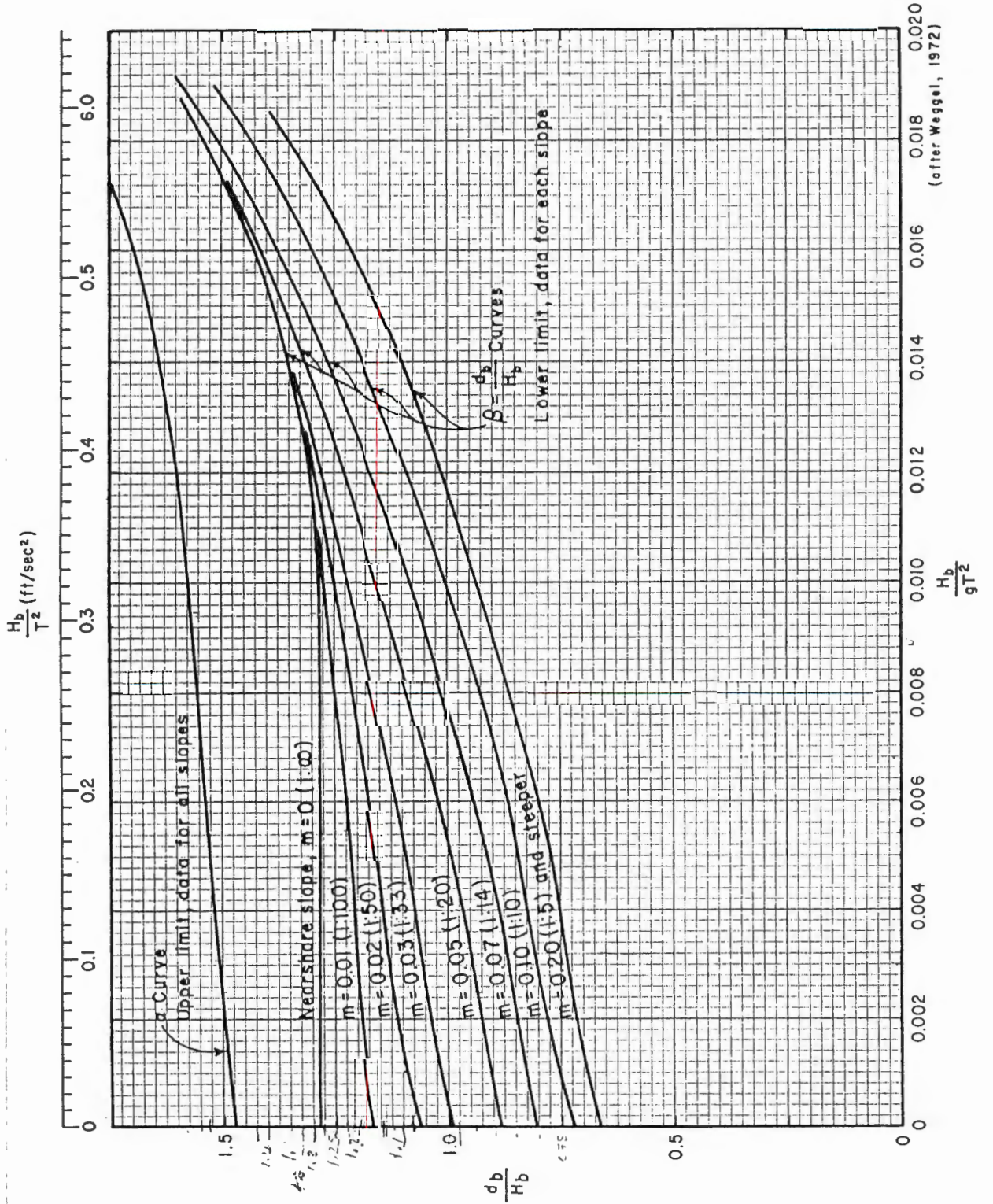


FIGURE 3.4.

BREAKER HEIGHT TO DEPTH RATIO VERSUS $\xi_b = \text{tg } \alpha / \sqrt{H_b / L_b}$

(GÜNBAK 1977)

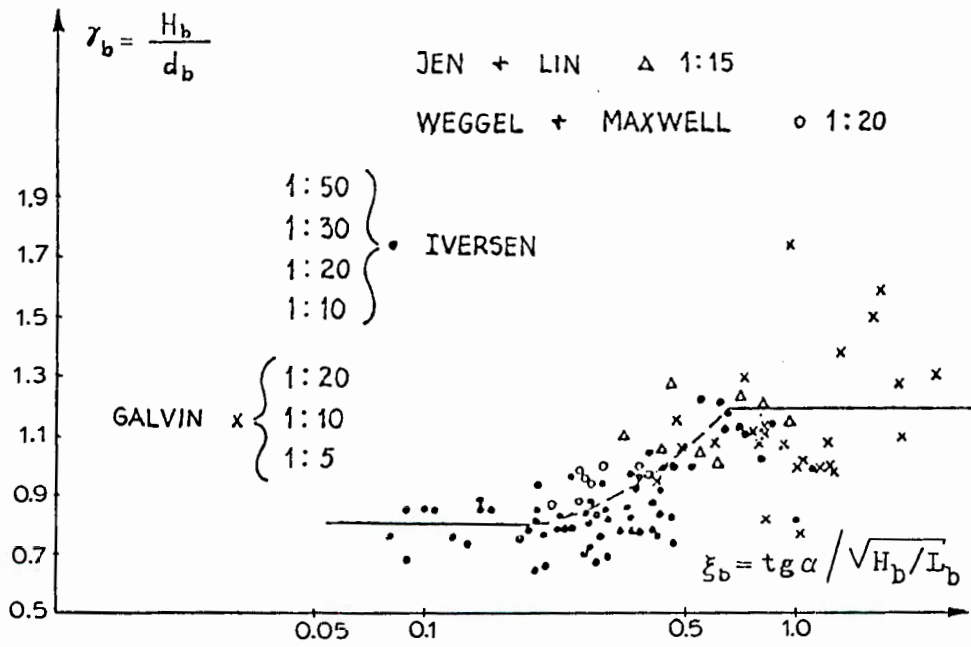


TABLE 3.1.

RUBBLEMOUND STABILITY CRITERIA
(Bruun (1985))

COUNTRY Authors and references	GENERAL FORMULÆ	FIGURE	
		Applied numerical values	f(u) = .
Spain:			
Castro (1)	$W = \frac{0.704}{(\cotg \alpha + 1)^2} \cdot \sqrt{\cotg \alpha - \frac{2}{ps}} \cdot \frac{H^2 \cdot ps}{pw - 1}$	ps = 2.65	$\frac{0.704}{(\cotg \alpha + 1)^2} \cdot \sqrt{\cotg \alpha - 0.754}$
Iribarren* (2) (3) (4)	$W = \frac{K}{(\cos \alpha - \sin \alpha)^2} \cdot \frac{H^2 \cdot ps}{pw - 1}$	K = 0.023 (for d < 0.06 L)	$\frac{0.023}{(\cos \alpha - \sin \alpha)^2}$
Iribarren**(5)	$W = \frac{K}{(\mu \cos \alpha - \sin \alpha)^2} \cdot \frac{H^2 \cdot ps}{pw - 1}$	K = 0.43 μ = 2.38	$\frac{0.43}{(2.38 \cos \alpha - \sin \alpha)^2}$
United States:			
Mathews (unpublished report)	$W = \frac{0.0149}{(\cos \alpha - 0.75 \sin \alpha)^2} \cdot \frac{H^2 \cdot T \cdot ps}{pw - 1}$	T = 2.5 H	$\frac{0.03725}{(\cos \alpha - 0.75 \sin \alpha)^2}$
Epstein and Tyrrel (6)	$W = \frac{K}{(\mu - \lg \alpha)^2} \cdot \frac{H^2 \cdot ps}{pw - 1}$ with K = K(u; μ: d/L)	—	—
Hickson and Rodolf (7)	$W = \frac{0.0162}{\lg^2 \left(45^\circ - \frac{\alpha}{2}\right)} \cdot \frac{H^2 \cdot T \cdot ps}{pw - 1}$	T = 2.5 H	$\frac{0.0405}{\lg^2 \left(45^\circ - \frac{\alpha}{2}\right)}$
Hudson (8) (9)	$W = \frac{1}{K_D \cdot \cotg \alpha} \cdot \frac{H^2 \cdot ps}{pw - 1}$	KD = 3.2 (for 0-1 % damage)	$\frac{1}{3.2 \cotg \alpha}^*$
		KD = 15.9 (for 30-60 % damage)	$\frac{1}{15.9 \cotg \alpha}^{**}$
France:			
Larras (10)	$W = \frac{K \cdot \left[\frac{2\pi H}{L} \cdot \frac{1}{\sin h \cdot \frac{4\pi Z}{L}} \right]^2}{(\cos \alpha - \sin \alpha)^2} \cdot \frac{H^2 \cdot ps}{pw - 1}$	K = 0.0152 With: Z = $\frac{H}{2}$ and $\frac{H}{L} < \sim 0.1$	$\frac{0.0152}{(\cos \alpha - \sin \alpha)^2}$
Beaudevin (11)	$W = K \cdot K_s \left(\frac{1}{\cotg \alpha - 0.8} - 0.15 \right) \cdot \frac{H^2 \cdot ps}{pw - 1}$	K = 0.10 Ks = 2.5	$0.25 \left(\frac{1}{\cotg \alpha - 0.8} - 0.15 \right)$

$$W = \frac{g \rho_s H^3}{K_d \left(\frac{\rho_s}{\rho_w} - 1\right)^3 \cot \alpha}$$

where

$W =$ armour unit size (N)
 $\rho_s =$ density of unit material (kg/m³)
 $\rho_w =$ density of sea water
 $\alpha =$ breakwater slope
 $K_d =$ stability coefficient
 $H =$ design wave height (m)

It can be seen that if the structure is depth limited then :

$$W = \frac{g \rho_s (\gamma_b d_b)^3}{K_d \left(\frac{\rho_s}{\rho_w} - 1\right)^3 \cot \alpha}$$

It is clear that under these conditions the determination of an appropriate water depth and associated wave height combination will have a significant influence on the final armour unit size.

The crest elevation of a rubblemound structure is determined by the amount of overtopping which can be permitted. Overtopping is directly related to the relative water depth, the wave steepness, bed slope, bed roughness and nature of the structure under consideration. Bruun (1985) presents a series of graphs by Inoue (1965), (see figure 3.5.), illustrating the effect of water depth on runup for smooth slopes. Figure 3.6. from CERC (1984), illustrates typical ranges of runup for rubblemound and smooth slopes of varying gradients. Whilst the subject of wave runup is extensive, suffice to say that within the context of this study, a direct relationship between water depth and runup will exist, similar to the breaker index. This implies that water depth plays a significant role in the determination of overtopping and thus crest elevation.

A further point of interest is toe stability. In this instance maximum wave downrush or rundown combined with a particular sea level draw down becomes important.

Gunbak (1979) uses the following formulations :

FIGURE 3.5.

EFFECT OF WATER DEPTH ON WAVE RUNUP (INOUE 1965)

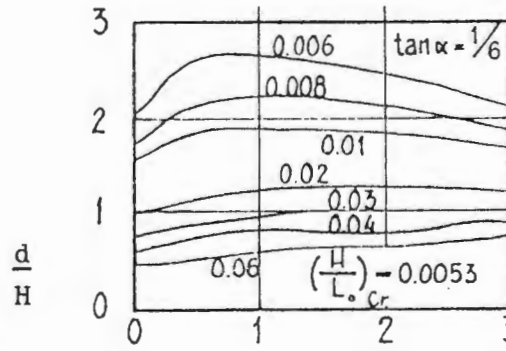
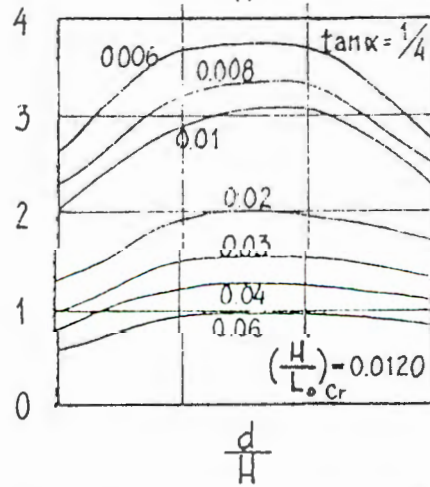
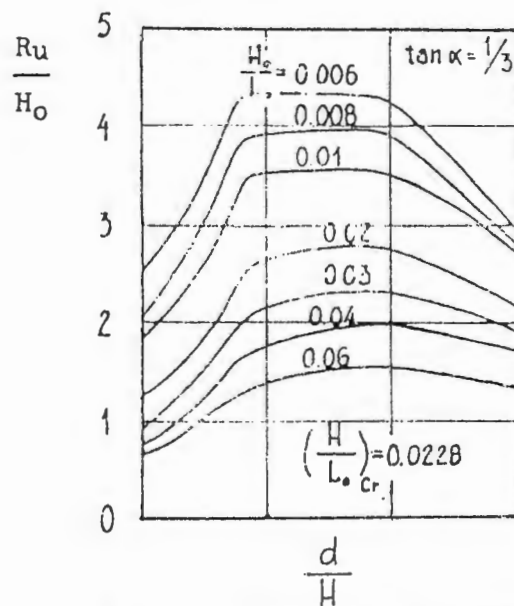
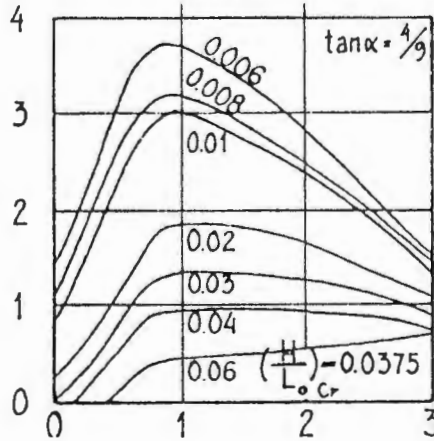
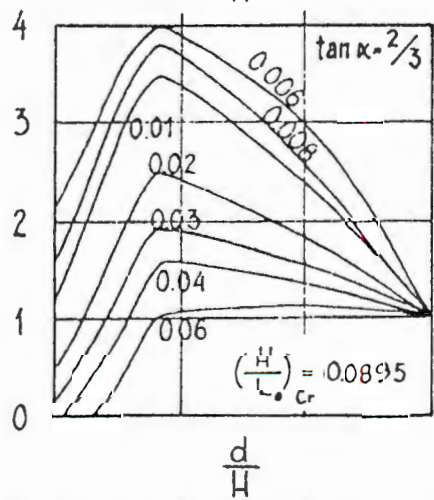
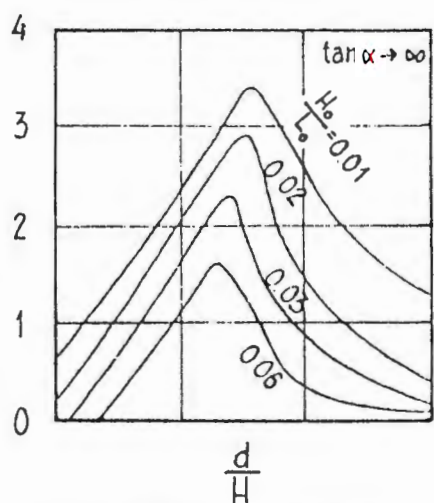
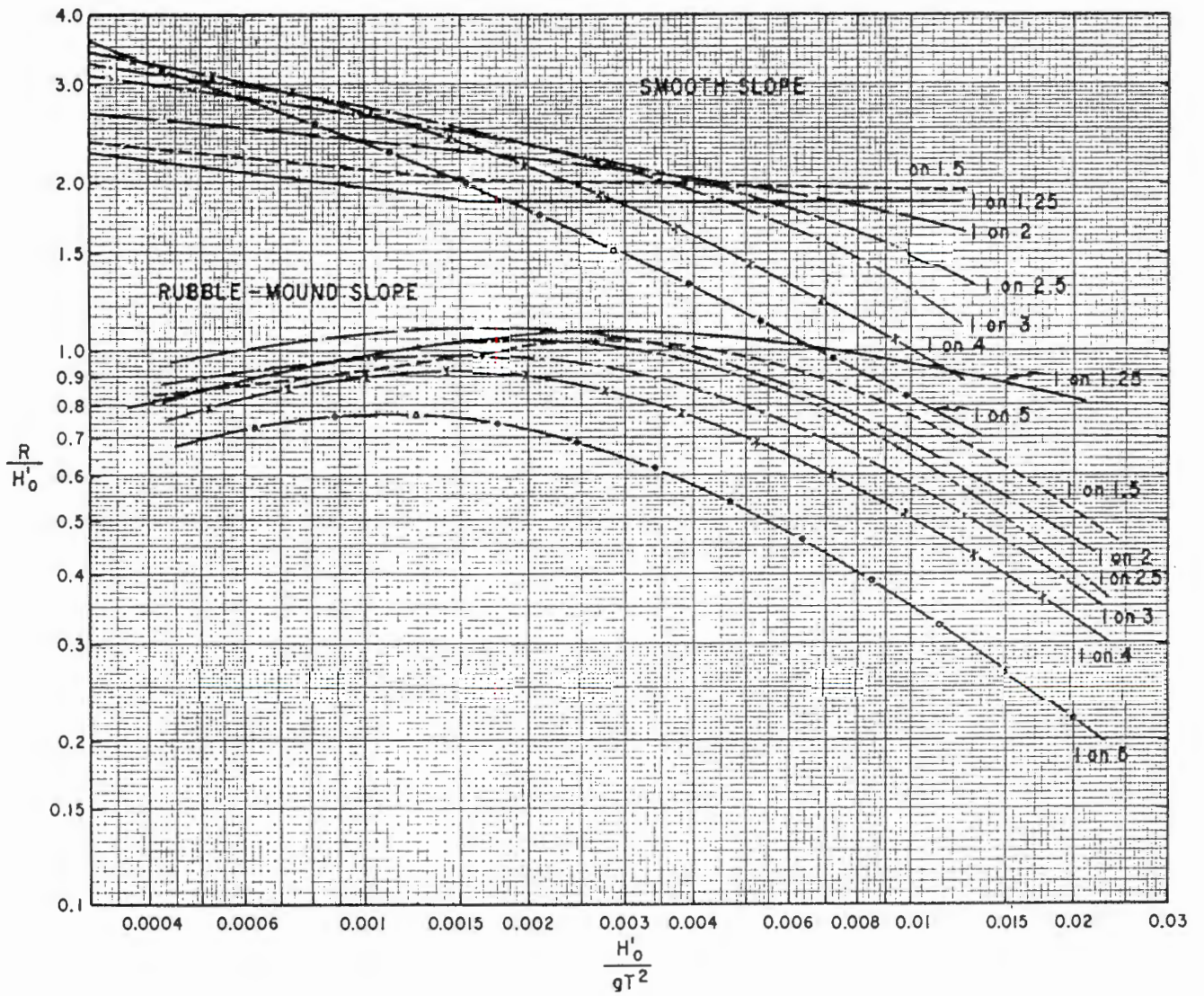


FIGURE 3.6.

COMPARISON OF WAVE RUNUP ON SMOOTH SLOPES WITH RUNUP ON PERMEABLE RUBBLE SLOPES



$$\frac{R_d}{H} = -0.27 \xi \quad \text{for} \quad \xi \leq 3.70$$

$$\frac{R_d}{H} = -1.0 \quad \text{for} \quad \xi > 3.70$$

where

R_d = wave run down (m)

H = expected wave height (m)

$$\xi = \frac{\tan \alpha}{\sqrt{H/L}}$$

In practice it is recommended, that, for steep breakwater slopes, the primary armour units are extended to a level $-2 H_s$ below the minimum sea level stand.

(Bruun (1985)). CERC (1984) proposes similar criteria. Figure 3.7. illustrates some common guidelines in terms maximum drawdown versus wave height considered important when assessing toe stability. The stability of rubble mound foundations are evaluated in terms of the following equations (CERC (1984)) :

$$W = \frac{g \rho_s H^3}{N_s^3 \left(\frac{\rho_s}{\rho_w} - 1 \right)^3} = \frac{g \rho_s (\gamma_b d_b)^3}{N_s^3 \left(\frac{\rho_s}{\rho_w} - 1 \right)^3}$$

(for depth limited conditions)

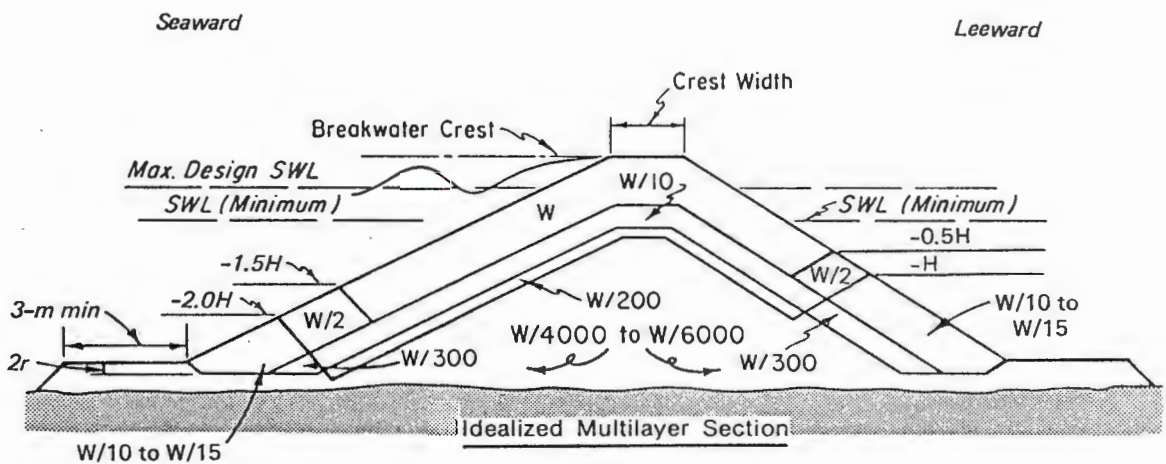
where

N_s = design stability number (figure 3.8.)

From the above equation and graph it can be seen that the armour unit size will increase as the depth ratio d_i/d_s decreases. The water depth plays a crucial role in these calculations. Another phenomena affecting toe stability is wave reflection. Wave reflection is highly correlated with the prevailing water depth at the toe. If the reflective index is larger than 0.25 then it is necessary to pay particular attention to the resulting scour. For structures having multiple slopes or berms, the reflective index can be expected to vary substantially at different water depths.

FIGURE 3.7.

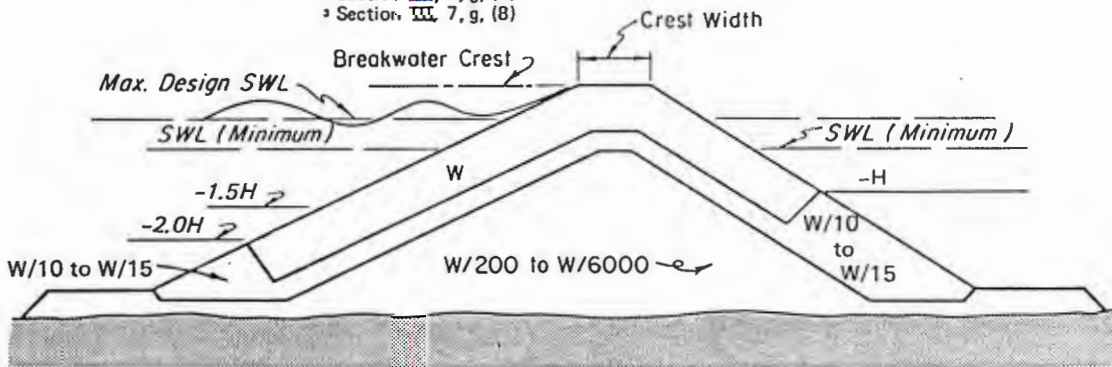
RUBBLEMOUND SECTION FOR SEAWARD WAVE EXPOSURE WITH ZERO TO MODERATE OVERTOPPING CONDITIONS



Rock Size	Layer	Rock Size Gradation (%)
W	Primary Cover Layer ¹	125 to 75
W/2 and W/15	Secondary Cover Layer ²	125 to 75
W/10 and W/300	First Underlayer ³	130 to 70
W/200	Second Underlayer	150 to 50
W/4000-W/6000	Core and Bedding Layer	170 to 30

H = Wave Height
 W = Weight of Individual Armor Unit
 r = Average Layer Thickness

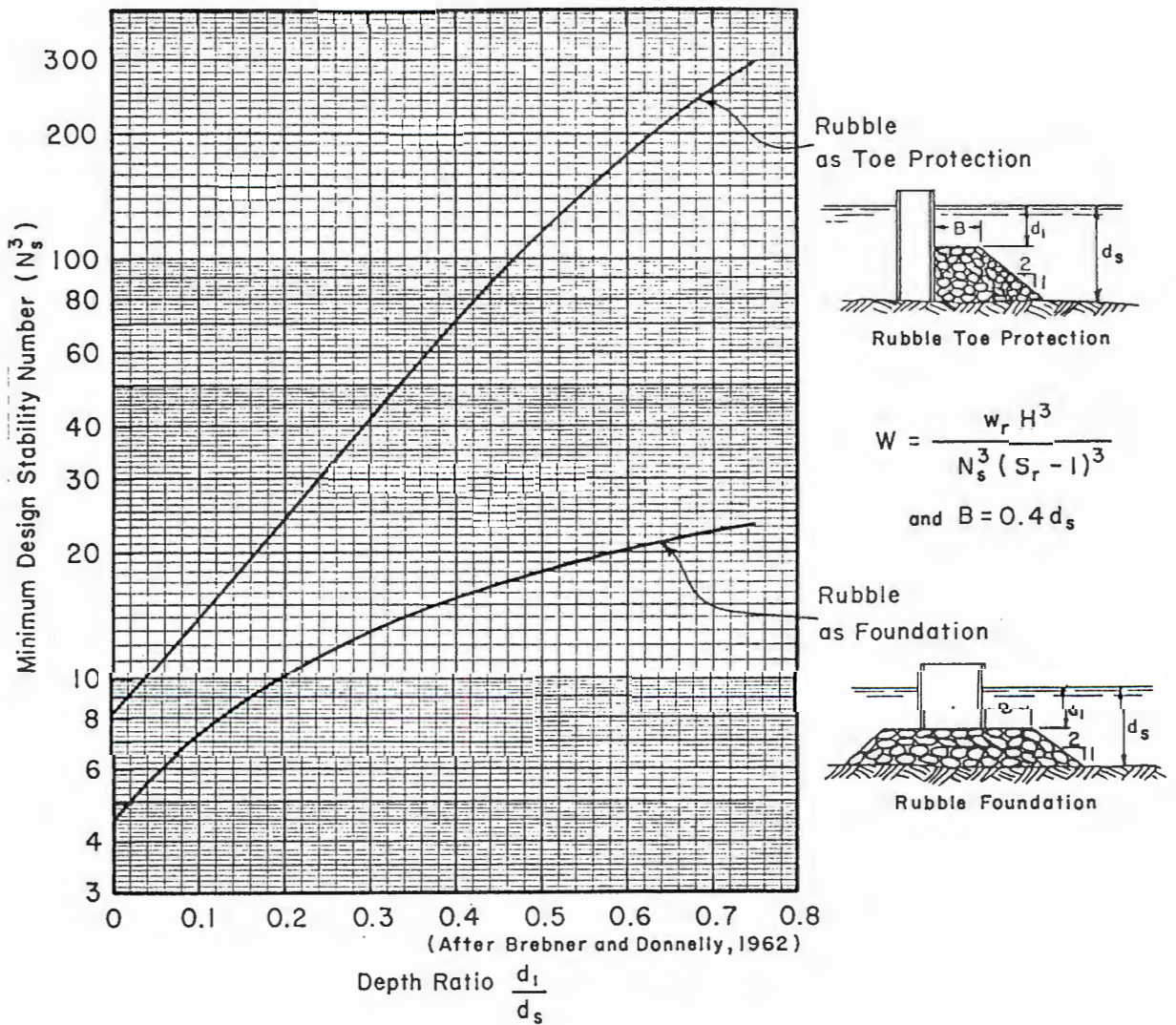
For concrete armor: ¹ Sections III, 7, g, (1), (2) and (6)
² Section III, 7, g, (7)
³ Section III, 7, g, (8)



Recommended Three-layer Section

FIGURE 3.8.

STABILITY NUMBER N_s FOR RUBBLE FOUNDATION
AND TOE PROTECTION



It can be seen from the foregoing discussion on rubblemound breakwaters that design conditions are sensitive to both wave height and water depth. In a similar fashion it can be shown that vertical walls, sea walls, bulkheads, revetments, piles and submarine pipelines in shallow water are design sensitive to both waves and water depth. The fact that both parameters are stochastic variables necessitates the assessment of the combined probability of occurrence of design events in order to quantify acceptable exposure to hazard or risk.

The assessment of sediment transport and beach morphology represents a further aspect of coastal engineering design where water depth and wave height play an important role. A number of ports in South Africa require some form of dredging to accommodate the natural accretion of sand. Tidal pools, small craft harbours and sea water intake design is particularly sensitive to the uncertainties regarding expected sediment transport rates.

Preliminary design assessments make use of first order approaches to quantify the bulk longshore transport for particular shorelines (CERC (1984) and Muir-Wood and Fleming (1981)). Typical bulk transport equations can be set out as follows :

$$\begin{array}{ll} Q = & f[(H_{brms})^2 c \sin \alpha_b] \\ \text{or} & Q = f[(\gamma_b d_b)^2 c \sin \alpha_b] \\ \text{where} & H_b = \text{breaking wave height (m)} \\ & c = \text{constant} \\ & \alpha_b = \text{incident breaking wave angle} \end{array}$$

It is apparent from the above formulations that longshore transport will be dependent on the prevailing sea level stand and wave height combination. A major shift in emphasis from wave structure interaction is the need for a dynamic rather than static assessment. Hence the duration of exceedence of certain threshold levels becomes more important.

The quantification of on-off shore or cross-shore sand transport has been addressed by Swart (1974), Kriebel and Dean (1984) and Larson (1988) amongst others. Whilst much attention has been given to the concept of equilibrium profile, the combination of

wave height and sea level stand would also appear to play an important role. The combined occurrence of storms and spring tides would appear to have far more devastating consequences on beach erosion than these events occurring independently. This can be illustrated by means of an example of an event which occurred between the 15th and the 18th of October 1992 in the Southern Cape. During these few days extensive sections of beach were removed offshore whilst certain pocket beaches were lost altogether (see plates 3.1.,3.2 and 3.3). It should be noted that storms of similar magnitude and direction occur from time to time without the same consequences. It would appear, in this instance, that the combined effect of spring high tide, moderate edge waves and large waves from a particular direction made a significant contribution towards the extensive offshore transport of sand.

The dispersion and dilution of effluent in the marine environment represents a relatively new area of coastal engineering design. Coastal water quality has become a major environmental, social and political issue with regard to the utilization of the marine environment for both recreational and waste water discharge purposes. In future, not only sewers and industrial pipelines, but also stormwater and estuarine outfalls will be subject to closer scrutiny. Apart from the difficulties associated with the use of the sea as a discharge sink for new schemes, existing schemes will be subject to review and operational optimization.

The evaluation of dilution at a particular site is complex. The parameters affecting dilution are the depth of discharge, discharge rate, local conditions, density stratification, diffuser characteristics and the nature of the effluent. The hydraulics of an outfall are normally divided into initial or jet dilution (in the near field) and secondary or subsequent dilution (in the far field)

Initial dilution is primarily a function of the diffuser manifold design and the water depth. Formulations developed by Roberts (1977) for stagnant uniform conditions can be written as follows:

For $Y/dF \geq 25-30$:

$$\frac{S_m}{F} = 0.107 (Y/dF)^{5/3}$$

PLATE 3.1.

STORM DAMAGE : 16TH OCTOBER 1992



PLETTENBERG BAY

PLATE 3.2.

STORM DAMAGE : 16TH OCTOBER 1992



MOSSEL BAY – DIAZ BEACH



PLETTENBERG BAY

PLATE 3.3.

STORM DAMAGE : 16TH OCTOBER 1992



PLETTENBERG BAY

For $Y/dF < 25-30$:

$$\frac{S_m}{F} = 0.107 \left(1.6 + 5 \left(\frac{Y}{dF} \right) + \left(\frac{Y}{dF} \right)^2 \right)^{5/6}$$

where S_m = minimum dilution on the plane centreline
 F = Froude number
 Y = effective depth (normally $Y = 0.7 \times$ water depth)
 d = port diameter

In both cases dilution relates exponentially to water depth. As most large outfalls operate in water depths varying between 20–60 m the expected relative influence of sea level will be small. The most likely problem will relate to shallow water outfalls and existing diffusers exceeding legal discharge limits during low sea level stands.

Secondary or far field dispersion is dependent on the prevailing environmental conditions, the water depth and the nature of the effluent. The rate of dispersion (or mixing time) can be expressed as follows:

$$T \approx d^2/\epsilon_v \quad (\text{Fischer et al(1979)})$$

where T = mixing time
 d = water depth
 ϵ_v = vertical eddy diffusivity coefficient

Clearly the rate of dispersion is sensitive to the actual water depth when evaluating the optimal utilization of an outfall.

SUMMARY

This chapter has reviewed in broad terms the significance of design sea level as used in coastal engineering practice. There are many more problems relating to harbour engineering, operations and construction which also have a significant bearing on the importance of a rational approach to design sea levels. Whilst this chapter places the need for a quantitative approach to design sea level within the context of this study, chapter 4 goes on to discuss those natural ocean processes affecting sea level in southern Africa.

CHAPTER 4

OCEAN PROCESSES AFFECTING DESIGN SEA LEVEL IN SOUTHERN AFRICA

The modelling of design sea level in southern Africa requires a fundamental understanding of the underlying physical processes. From the literature it would appear that sea level in this region is affected by the following natural phenomena:

Astronomical Tide

Meteorologically Induced Phenomena

Shelf Waves

Long Period Edge Waves

Wave Setup

Wind Setup

Tropical Cyclones

Tsunamis

This chapter discusses, with a view to facilitating the data analysis and model construction, the primary characteristics of these phenomena. Attempt will be made to highlight the relative significance of these processes as they affect design sea level.

4.1. *ASTRONOMICAL TIDE*

The fluctuation of sea level, as a result of astronomical tide, represents the most obvious single phenomenon affecting design sea level. Tides are driven by deterministic processes and are therefore predictable for any point along the coastline. The significance of tidal fluctuations lies in its combination with other random or stochastic processes which contribute towards extreme high or low water stands.

Muir-Wood and Fleming (1981) refer to Doodson (1954) as discovering rudimentary tide tables for London Bridge for 1213. The British Admiralty, who were initially responsible for tidal prediction in Southern Africa, began producing tide tables in 1833. The measurement of tides in Southern Africa has been noted as going back to the 1880's (Gill (1883)). All these data were transferred to the British Admiralty and were not available for review in this study. Tide gauge measurements, in the form of marigrams, are available from the South African Navy hydrographer going back to the early 1930's,

but it was only after 1958 that reasonable records became available. The South African hydrographer maintains an index of tidal archives for the 13 ports under its jurisdiction (see figure 4.1.). Details pertaining to the type of gauge, missing data, data format and location are available. Until recently tidal measurements were made in an analogue format and subsequently digitized by hand. Digitized records appear to be available, in some cases, dating back to 1958 however the quality and extent varies from port to port. TRIG SURVEY (1966) describes primary leveling in South Africa from 1925 to 1965. In this study sea level measurements were used to establish datum bench marks in Durban, Cape Town, East London and Port Elizabeth. Tide gauge measurements over a period of one or two years were used, but were considered inadequate at the time. Subsequent to the 1st of January 1979, the hydrographer adopted a standard relationship of 0,9m difference between chart datum and land leveling datum or mean sea level for Southern African ports.

Tides are driven by the gravitational interaction of the moon and the sun on the earth's ocean mass. Tidal theory has been developed by many researchers from Isaac Newton to Doodson. Numerous comprehensive references are available on this subject and are therefore not repeated here.

The response of sea level, at a particular coastal location, is a function of the nature of the sea bed topography, shallow water effects and Coriolis forces prevalent in that region. Due to the relatively uniform nature of the Southern African coastline, tidal response does not vary significantly. The tidal range varies between a neap range of 0,56 m, a mean range of 1,48 m and maximum range of 2,48 m. Table 4.1 gives a listing of the characteristics of various tide stations around Southern Africa. Luderitz, on the Namibian coast, has the smallest range of 1,96 m, whilst Richards Bay has the largest of 2,48 m. The tidal phase is such that tides on the West Coast occur simultaneously whilst it takes approximately 35 minutes to propagate from Cape Town to Durban.

Work carried out to date would seem to indicate that tidal response acts independently of other natural processes affecting sea level in Southern Africa. The approach taken in this study, therefore, is to consider tide as a statistically independent deterministic process. The specific nature of tides in Southern Africa will be assessed in more detail in chapter 6.

FIGURE 4.1.

LOCATION MAP
(Rossouw 1989)

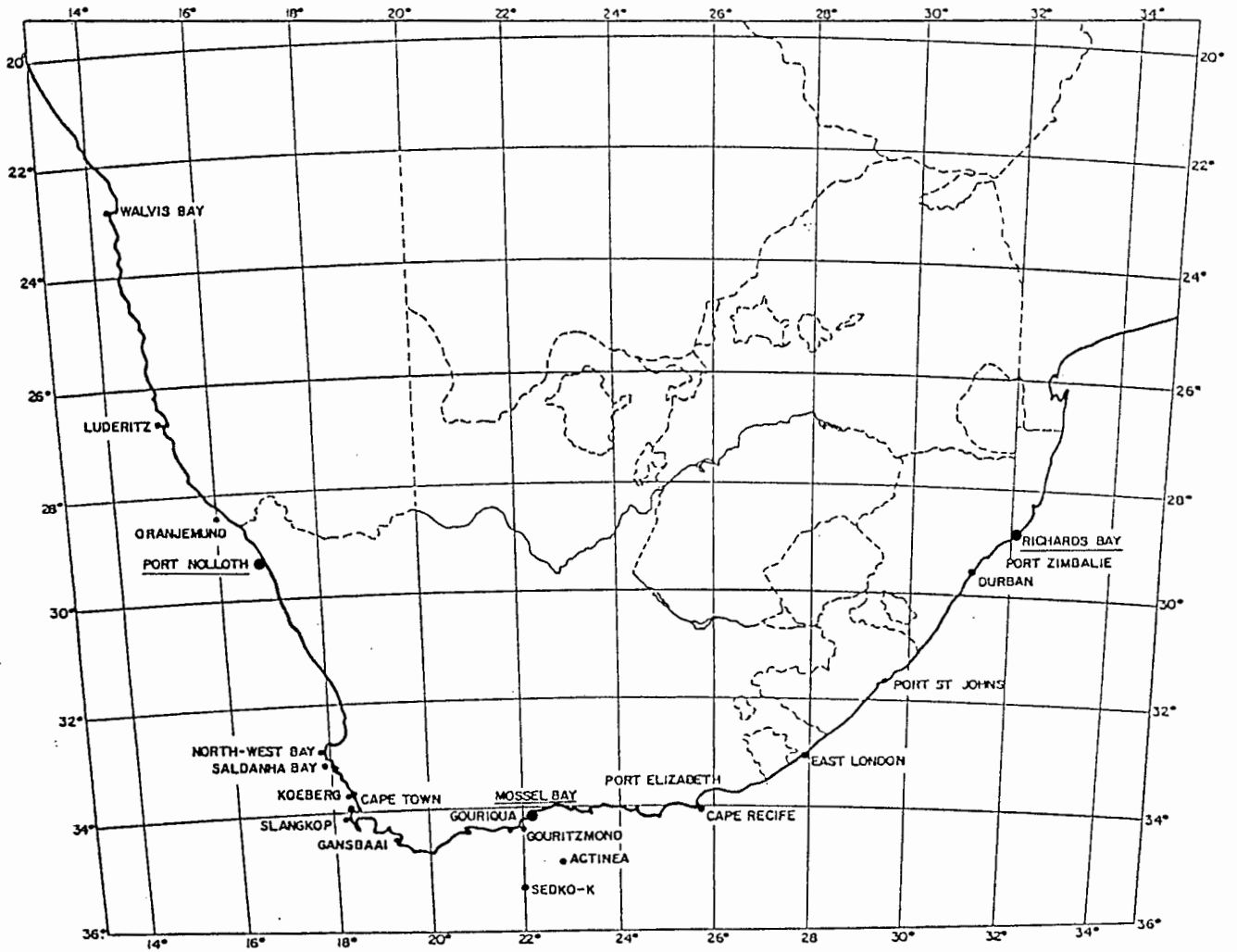


TABLE 4.1.

TIDE CHARACTERISTICS FOR SOUTHERN AFRICAN PORTS

PLACE	LAT	MLWS	MLWN	ML	MHWN	MHWS	HAT
Walvis Bay	-0,05	0,19	0,59	0,90	1,21	1,61	1,92
Lüderitz	-0,20	0,06	0,44	0,74	1,03	1,41	1,76
Port Nolloth	-0,19	0,09	0,55	0,87	1,20	1,66	2,02
Saldanha	-0,06	0,26	0,76	1,01	1,26	1,76	2,09
Cape Town	0,09	0,34	0,78	1,05	1,33	1,76	2,10
Simon's Town	0,07	0,32	0,78	1,06	1,34	1,80	2,14
Hermanus	0,12	0,37	0,81	1,09	1,37	1,81	2,15
Mossel Bay	-0,01	0,25	0,84	1,13	1,41	2,00	2,42
Knysna	0,11	0,36	0,90	1,16	1,43	1,96	2,31
Port Elizabeth	-0,05	0,29	0,84	1,09	1,35	1,90	2,35
East London	0,24	0,37	0,94	1,19	1,44	2,00	2,24
Durban	-0,02	0,24	0,85	1,10	1,35	1,96	2,30
Richards Bay	-0,11	0,19	0,83	1,09	1,35	1,99	2,37

The above levels are referred to CHART DATUM

4.2. METEOROLOGICALLY INDUCED PHENOMENA

Storm surge refers to the departure of sea level from predictable still water level as a direct consequence of a storm. This definition of response, although generally accepted in coastal engineering, is of little value when attempting to assess the characteristics of storm surges in different parts of the world. The commonality relates to their primary driving forces notably, wind stress and pressure. The manner in which these processes generate, enhance and propagate waves is, however, essentially different.

Three major subdivisions may be identified in the literature in respect of storm surges. Tropical cyclones or hurricanes have been relatively well researched and modelled, and consequently their response in coastal regions is well understood and documented. (Bretschneider (1967), Yeh and Yeh (1976), Jelesnianski (1967)). Researchers and engineers in Northern Europe and the United Kingdom have investigated storm surge resulting from extra tropical cyclones and particularly their influence on the coastal areas adjacent to the North sea (Ishiguro (1983), Flather and Proctor (1983), Pugh and Vassie (1978), Tawn and Vassie (1989)). A third avenue of work has been that of the scientists investigating ocean processes. They have essentially singled out and defined subdivisions of storm surges based on the different scales of response of the events (Munk (1961), Shillington (1985) and Le Blond and Mysak (1978)). This has largely confused rather than clarified the coastal engineering interpretation of the problem. It is deemed important therefore that definition be given in this regard for the purposes of coastal engineering practice in Southern Africa.

The approach followed in this study will be to define the process according to the physical characteristics of the response in terms of sea level fluctuations. This approach requires that extensive correlation analysis be carried out to ensure that the different responses do not originate from the same forcing mechanism. If this is found to be the case, then a form of dependency will have to be incorporated in any modelling process.

Sea level research work carried out to date points towards there being five major responses identifiable in existing sea level measurements, notably:

1. *Shelf Waves;*
2. *Long Period Edge Waves;*

3. *Wave Setup;*
4. *Wind Setup;*
5. *Tropical Cyclone surge*

Whilst all the above processes are meteorologically induced (ie: wind, stress and barometric pressure), the manner in which they are initiated and the accompanying level response determine that they be analyzed apart.

4.2.1. *Shelf Waves*

Shelf waves, also referred to as coastal trapped waves, were first investigated by Robinson (1964) for the Australian coast. Since then extensive work has been carried out for the West coast of America (Munk and Smith (1968)) and the southern African coast, (De Cuevas (1985), Schumann (1983), Gill and Schumann (1974)).

Shelf waves are generated by synoptic scale events on the inner continental shelf of southern Africa. According to De Cuevas (1985), the coastal low satisfies the requirements of a forcing mechanism for the generation of the shelf wave. The coastal low, once generated, interacts with the prevailing larger scale atmospheric systems, which then also act as the forcing mechanisms for propagation along the coast. In the case of southern Africa these large scale atmospheric systems are represented by the South Atlantic high pressure system and mid latitude depressions (De Cuevas (1985)). Shelf waves are therefore dependent on the concurrence of a number of meteorological events within a specific time framework. By implication, therefore, the generation and propagation of shelf waves will be dependent on seasonal variations as a result of changes in the synoptic weather patterns.

De Cuevas (1985) states that during the summer period the dominant longshore wind provides the propagating mechanism for the coastal low. The effect is to enhance the barometric factor as the wind and air pressure interact. As the wind field and direction is relatively uniform large changes in sea level are recorded during this period.

During the winter months, the South Atlantic high pressure system moves several degrees to the North resulting in the more frequent passage of cold fronts along the

coast. These frontal systems then provide the forcing mechanism required to propagate the coastal low. Due to the change in wind direction associated with the passage of cold fronts the response during winter would appear to be less than during the summer months (De Cuevas (1985)).

The sea level response, as measured at tide gauges around the coast, indicates that shelf wave amplitudes in the region vary between 100–500mm with wave periods of 2–20 days. The variability in the wave characteristics relates to the fact that shelf waves are not free propagating and are therefore dependent on the prevailing forcing mechanism. Due to the regional nature of this phenomenon there exists a strong correlation between ports around the coast (see figure 4.2.).

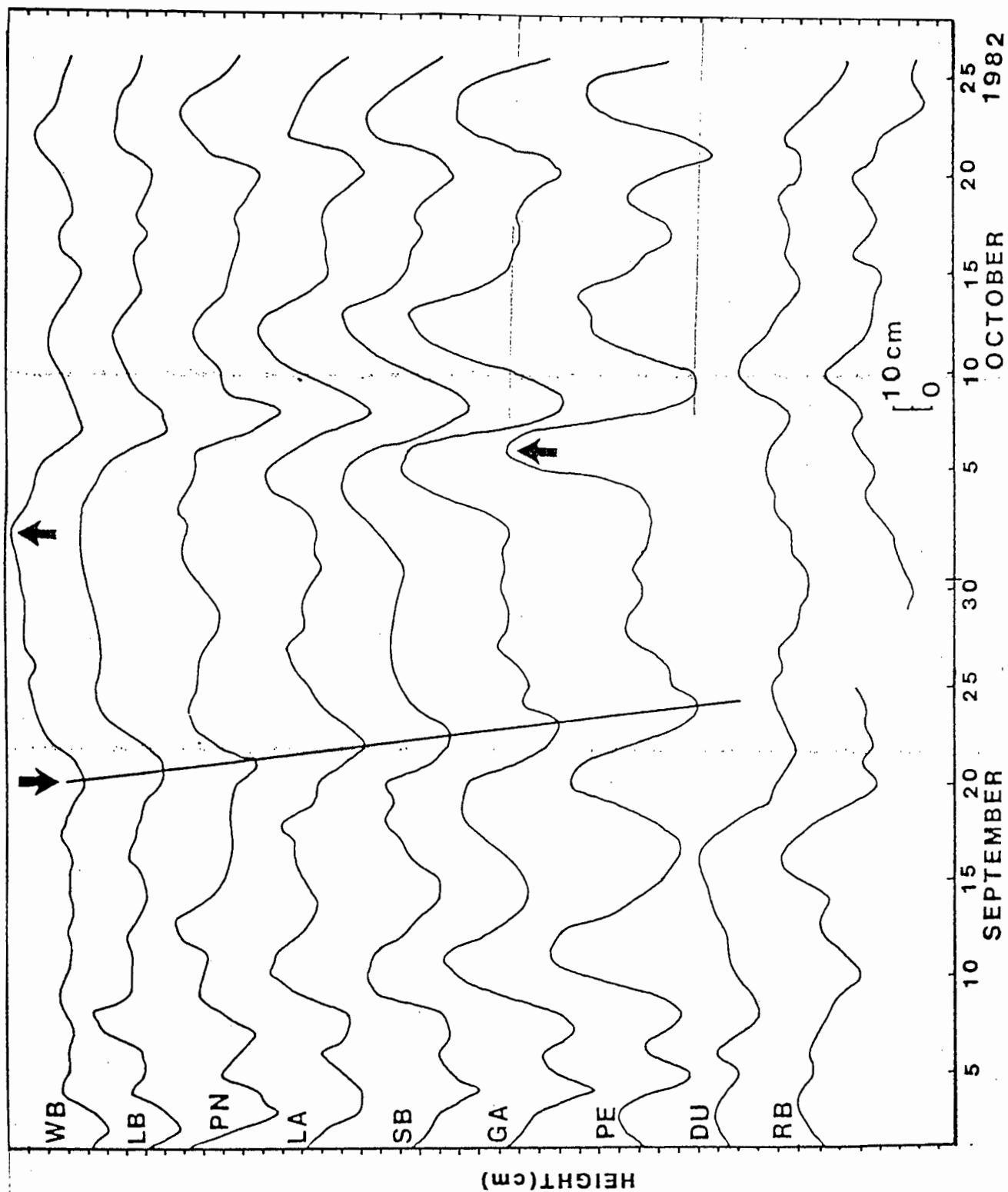
For the purposes of the development of a probabilistic model, it would appear that shelf waves are statistically independent of astronomical tide. It is, however, not clear whether shelf waves are independent with respect to other meteorologically induced phenomena events such as wind waves and long period edge waves. It is expected that weather systems will, when the conditions are conducive, produce some form of combined effect. At this stage in our understanding it is assumed that the development of conditions conducive to shelf waves represent a stochastic process.

4.2.2. *Long Period Edge Waves*

Long period edge waves have been observed along the South African coast in association with rapidly moving micro pressure oscillations by Darbyshire (1963), Darbyshire and Darbyshire (1964) and Shillington (1984). Wave heights at the coast vary between 60–120 cm with periods ranging between 10–60 minutes. Events would appear to have a duration of 6 – 12 hours. The first documented observations of edge waves were made by Munk, Snodgrass and Carrier (1956) off the East coast of the U.S.A. as a result of rapidly moving hurricanes. Le Blond and Mysak (1978) reviewed the theoretical dynamics of long period edge waves (Shillington (1988)). Shillington (1985) investigated long period edge waves in southern Africa. Long period edge waves appear to be initiated by a rapid micro pressure oscillation. Once the sea wave has been established, it must propagate at a similar speed to the fast moving (30m/s) atmospheric oscillation across the continental shelf. This matching between the sea and

FIGURE 4.2.

DAILY MEAN SEA LEVEL 1982 SEPTEMBER 1 - OCTOBER 27



atmospheric waves results in a resonant interaction which enhances the induced wave by up to 10 times (Shillington (1985)). Although Shillington and Van Forrest (1986) have successfully managed to model this process numerically, the dynamics of the actual process appear complex. Thus a degree of variability is apparent in the available data sets. Unlike shelf or coastal trapped waves, no correlation exists between prevailing wind conditions and the generation of edge waves. Shillington (1988) reports these occurrences as taking place approximately 4 – 5 times per year.

Measurements obtained from tide gauges first noted the existence of long period edge waves in Southern Africa. Their marigram signature was almost identical to that of tsunamis (Wigen et al (1981)), but no local tsunamigenic mechanism could be associated with the measured events. Shillington (1984) attributed these events to micro pressure oscillations. Both the west and south east coasts appear equally prone to these events. East coast ports of East London, Durban and Richards Bay show no evidence of edge waves. This is most probably related to the shelf configuration and the influence of the Agulhas current.

It is not clear at this stage whether unusual air pressure forcing waves can be related to larger synoptic weather systems. Shillington (1985) implies that the occurrence of the latter is indeed random but he also mentions that the most probable time of occurrence appears to be around the equinoxes. This aspect will be reviewed as part of this study and discussed in chapter 6. Figure 4.3. illustrates a typical edge wave record.

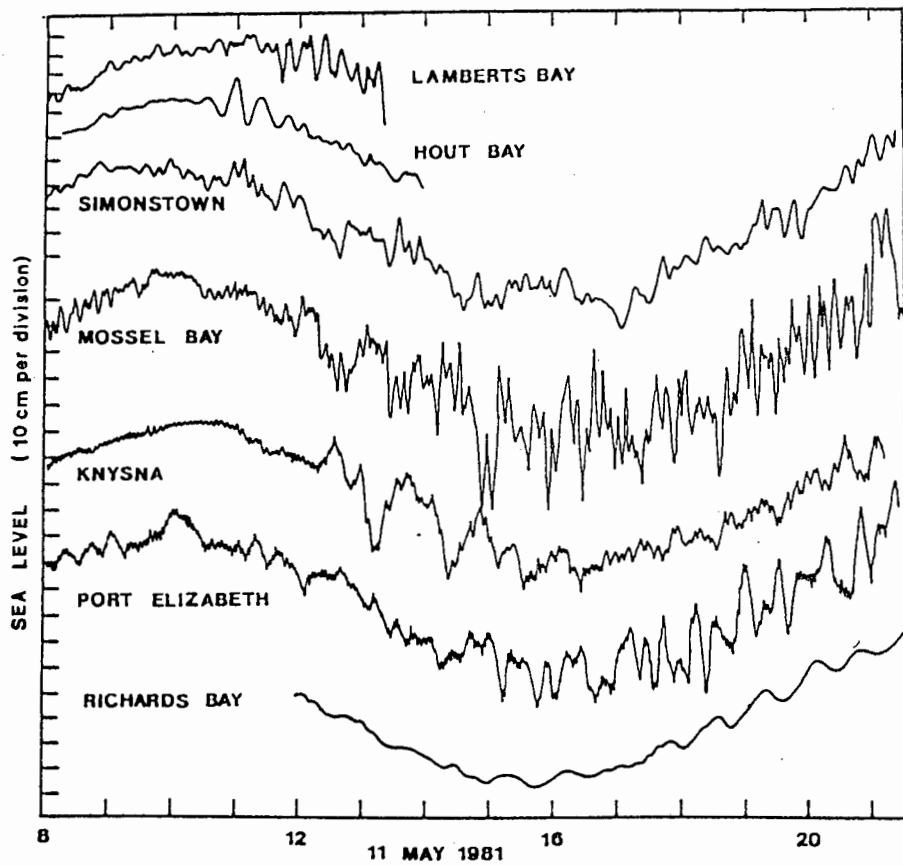
4.2.3. *Wave Setup*

Wave setup was noted as having an influence on nearshore water level by Savage (1957), Fairchild (1958), Dorrestein (1962) and Galvin and Eagleson (1965). CERC (1984) refers to Saville (1962) as undertaking the first quantitative laboratory study on wave setup, however it was most probably Longuet-Higgins and Stewarts (1963)'s work on radiation stress which provided the fundamental theoretical understanding of this phenomenon. Le Mehaute (1969) defines radiation stress S_{zz} as the average value of the sum of $(p_w + \rho V_n^2)$ with respect to time or :

FIGURE 4.3.

EDGE WAVE MEASUREMENTS

(Shillington (1988))



$$S_{zz} = \frac{1}{T} \int_0^T \int_{-d}^{\eta} (p_w + \rho u_n^2) dz dt$$

where p_w = pressure fluctuation due to water waves around the hydrostatic pressure from still water level
 $= p - \rho g z$
 ρu_n^2 = momentum flux

According to linear wave theory radiation stress S_{zz} can be expressed as :

$$S_{zz} = \frac{1}{16} \rho g H^2 \left[1 + 2 \frac{4 \pi d / L}{\sinh(4 \pi d / L)} \right]$$

As a wave propagates from deep to shallow water, so the radiation stress increases according to linear wave theory. This increase in radiation stress leads to an imbalance in the momentum flux which is counteracted by an external force. This external force is provided by differences in hydrostatic pressure. (Le Mehaute (1969)).

Le Mehaute goes on to show that by applying the momentum theorem the following relationship may be obtained:

$$\frac{d\bar{\eta}}{dx} = - \frac{1}{\rho g d} \frac{dS_{zz}}{dx}$$

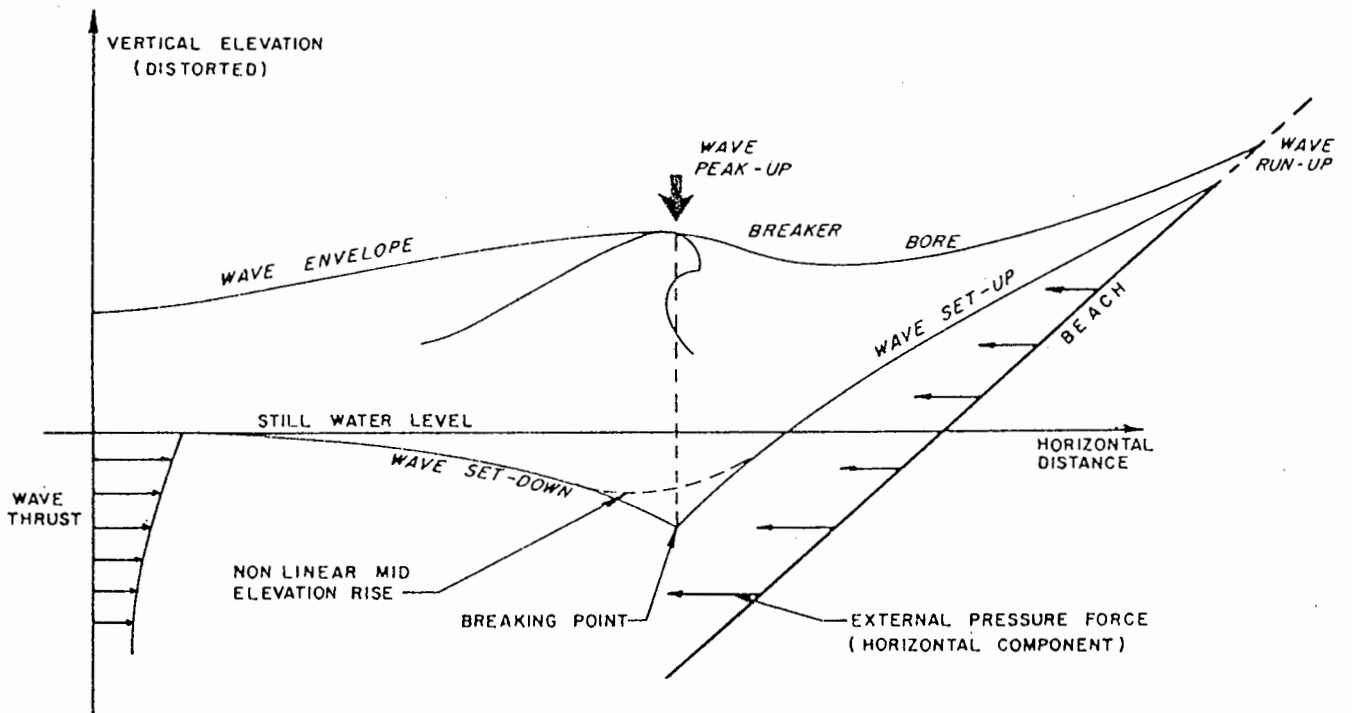
where $\bar{\eta}$ = water surface elevation

Thus as S_{zz} increases so the surface elevation decreases. This depression will have a maximum value just prior to wave breaking when the value of S_{zz} is at a maximum point. Similarly subsequent to wave breaking, the wave energy dissipates and S_{zz} decreases resulting in an increase in the surface elevation in the surf zone. This increase in surface elevation is known as wave setup (see figure 4.4.).

Longuett-Higgins and Stewart (1963) developed the following empirical formulations for wave setup.

FIGURE 4.4.

EFFECTS OF RADIATION STRESSES IN THE SURF ZONE



$$\begin{aligned}\Delta S &= S_b + S_w \\ S_b &= \text{set down at breaking zone} \\ S_w &= \text{setup at the shore}\end{aligned}$$

where

$$S_b = - \frac{g^{1/2} H_o^2 T}{64 \pi d_b^{3/2}} \quad (\text{CERC}(1984))$$

where and $\Delta S \doteq 0.15d_b$

$$\begin{aligned}H_o &= \text{unrefracted deep water significant wave height} \\ T &= \text{wave period} \\ d_b &= \text{depth of water at breaker point} \\ g &= \text{acceleration of gravity}\end{aligned}$$

From the above formulation the wave setup S_w can be calculated. It should be emphasized that it is the continuous occurrence of high wave energy breaking that results in the super elevation of the water level. It may be expected that this process is quasi-steady and that the water level would fluctuate over several minutes. Hansen (1978), (c.f. Bruun (1985)) proposes maximum values of wave setup according to the following empirical formula:

$$S_w = 0.3 H_b$$

where $S_w =$ maximum wave setup

$$H_b = \text{the significant breaking wave height}$$

Holman (1990), summarizes present knowledge on wave setup indicating that, for natural beaches, setup will be between 17-50% of the incident wave height. Longuett-Higgins et Stewart (1963) and Hansen (1978)'s work was based on the response of monochromatic waves (swell) at the shore. Work by Goda (1975) has

shown that wave set up due to random waves (sea) is essentially different, being a function of wave steepness and wave grouping. CERC (1984) provides separate curves to illustrate this relationship (see figures 4.5(a) and 4.5(b)).

Wind generated gravity waves represent the primary driving force affecting wave setup in Southern Africa. The meteorology of the South Atlantic and the South Indian oceans is not well documented at this stage in as much as it relates to the generation of waves. A number of generalized descriptions have been made by Swart and Serdyn (1981), Rossouw (1989) and SAN (1975) in this regard. Rossouw (1989) attributes the major source of large waves to the passage of low pressure systems, from west to east, associated with the Ferrel westerly wind system (see figure 4.6.). The seasonal variations in conditions can be attributed to the change in position during the summer and winter periods of the South Atlantic and South Indian high pressure anti-cyclones. Hunter (1987) examined a number of storm case histories for the Cape south coast region. It is clear from this work that the relationship between wave climate and synoptic weather systems is not straight forward. Contrary to the earlier literature Hunter (1987) observed that a significant number of large, short period, wind wave events are generated by east moving coastal lows. Based on the available information therefore, it is not clear whether any obvious relationships exist between the causative process affecting wind waves, shelf waves or edge waves. These relationships are discussed in more detail in chapter 6.

4.2.4. *Wind Setup*

Wind setup is the resultant increase in sea level as a direct result of wind stress over a body of water. Wind exerts a horizontal force on the water surface and a surface current in the direction of the wind. CERC (1984) ascribes the sea level fluctuation to the induced horizontal currents in shallow water. If the wind is onshore this results in wind setup whilst conversely if it is offshore it induces a falling level or wind setdown. The former phenomenon is often referred to in engineering circles as storm surge. Within the context of this study wind setup is restricted to the local effects induced by winds measured at a particular location. The distinction between shelf wave effects and wind setup is made on the basis of the different scale of the processes. Whilst shelf waves are the result of regional scale events, their response being measured throughout the subcontinent, wind setup is seldom noted at more than one tide gauge.

FIGURE 4.5.(a).

MEASURED RANDOM WAVE (SEA) SETUP ON A 1 ON 30 FLAT SLOPE

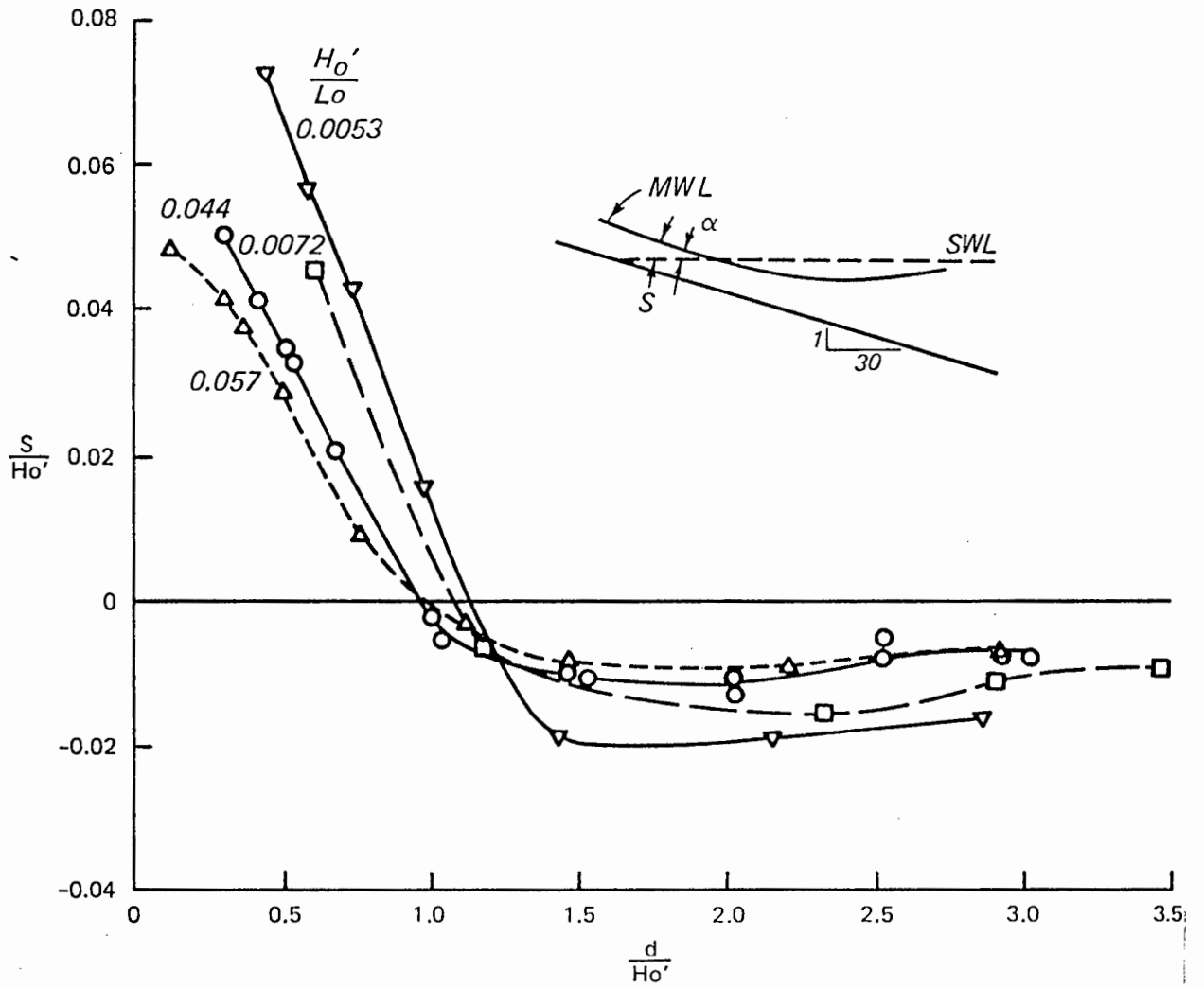


FIGURE 4.5.(b).

PREDICTED RANDOM WAVE (SEA) SETUP ON PLANE SLOPES

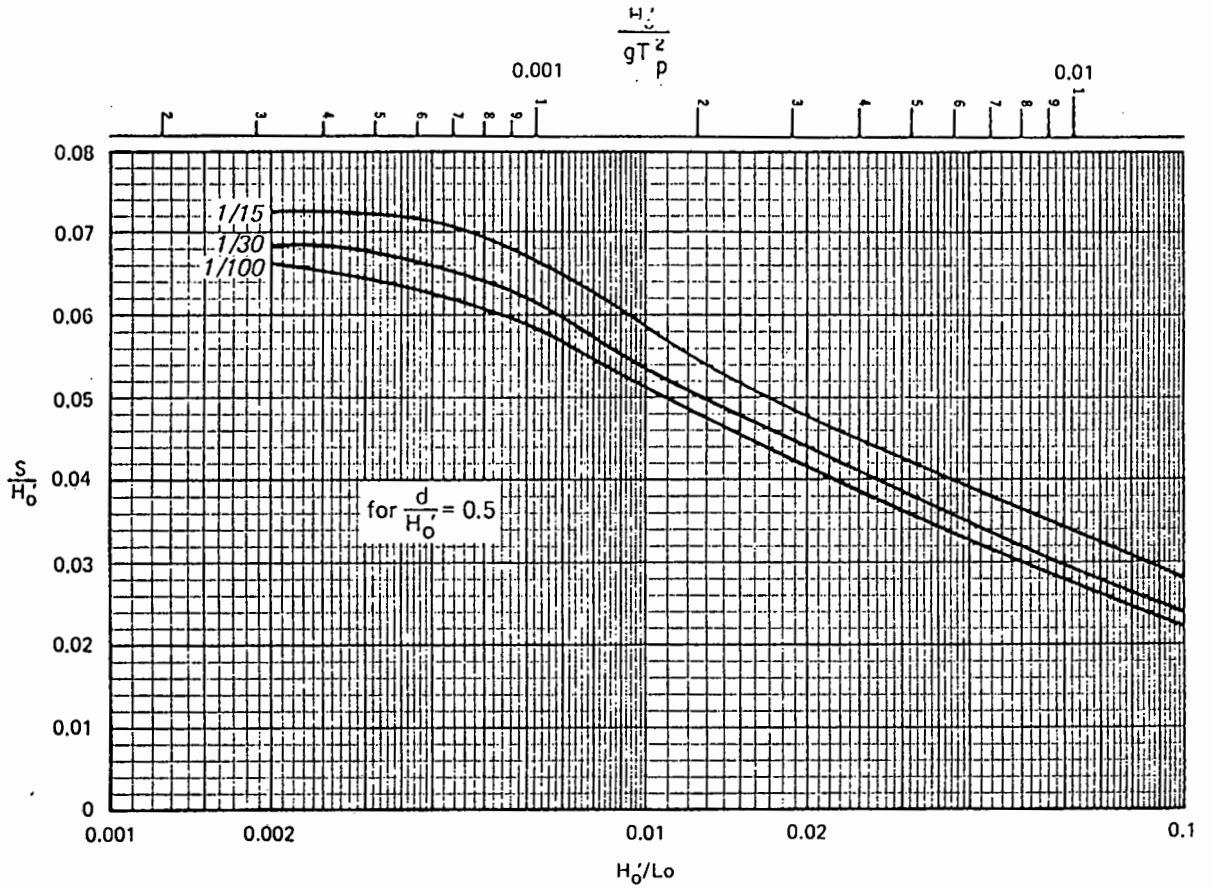
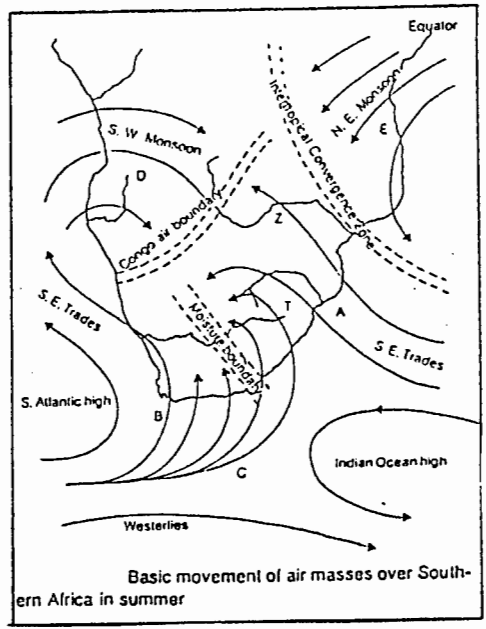
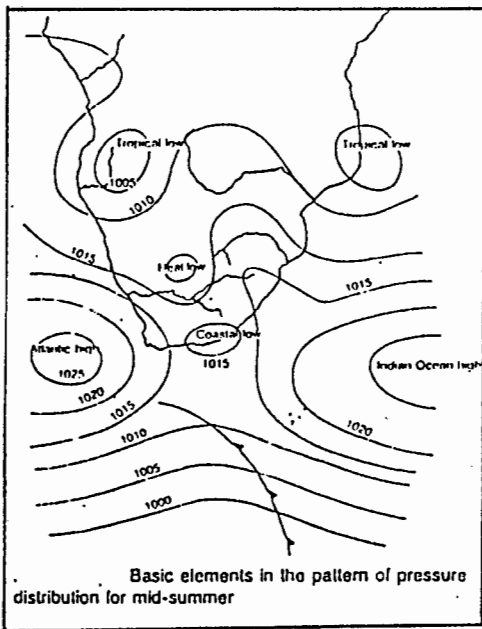
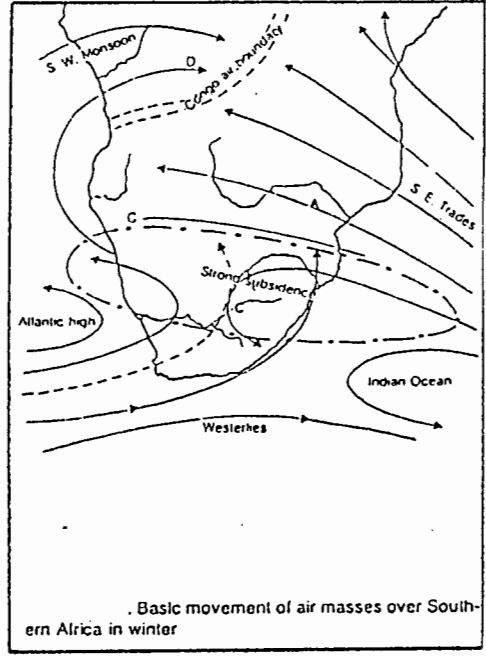
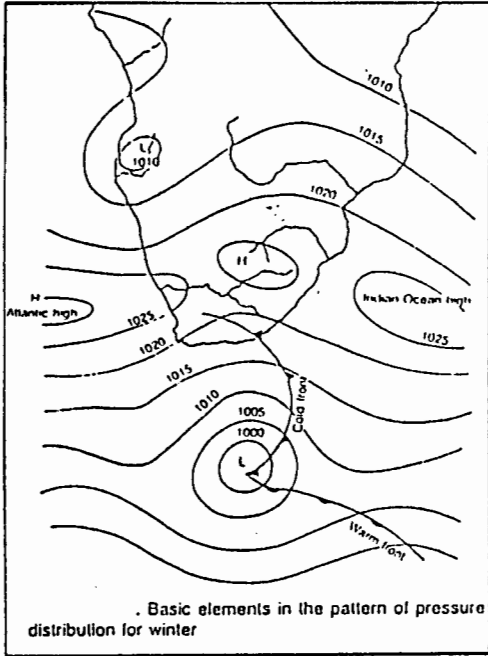


FIGURE 4.6.

PRESSURE AND WIND SYSTEMS OVER SOUTHERN AFRICAN OCEANS
(Hurry and Van Heerden (1982))



Muir-Wood and Fleming (1981) present a relationship between sea surface slope, as a result of surface wind stress, versus windspeed and water depth

$$S = \frac{Cn\rho_a U^2}{\rho d}$$

where	S	=	sea level slope
	C	=	friction factor
	ρ_a/ρ	=	1.25×10^{-3}
	n	=	$1.15 < n \leq 1.30$
	U	=	windspeed at 10 metres above m.s.l.
	d	=	water depth

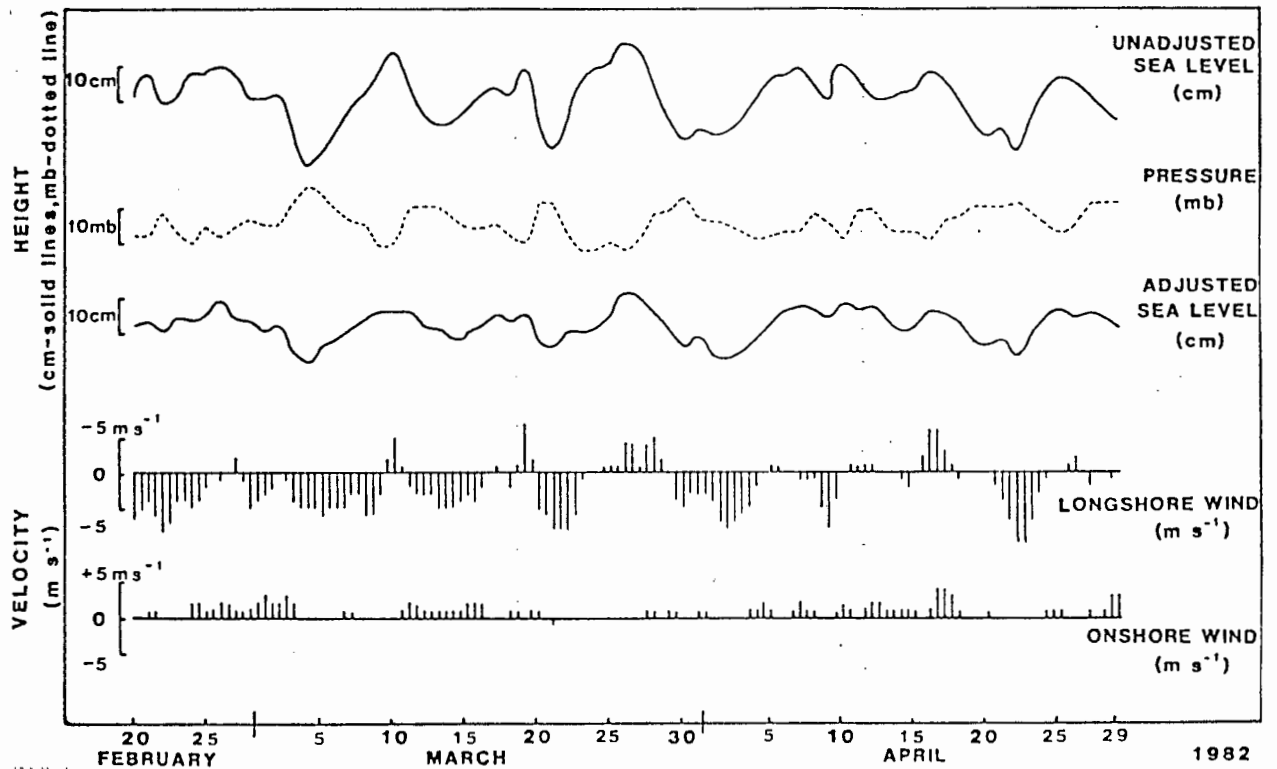
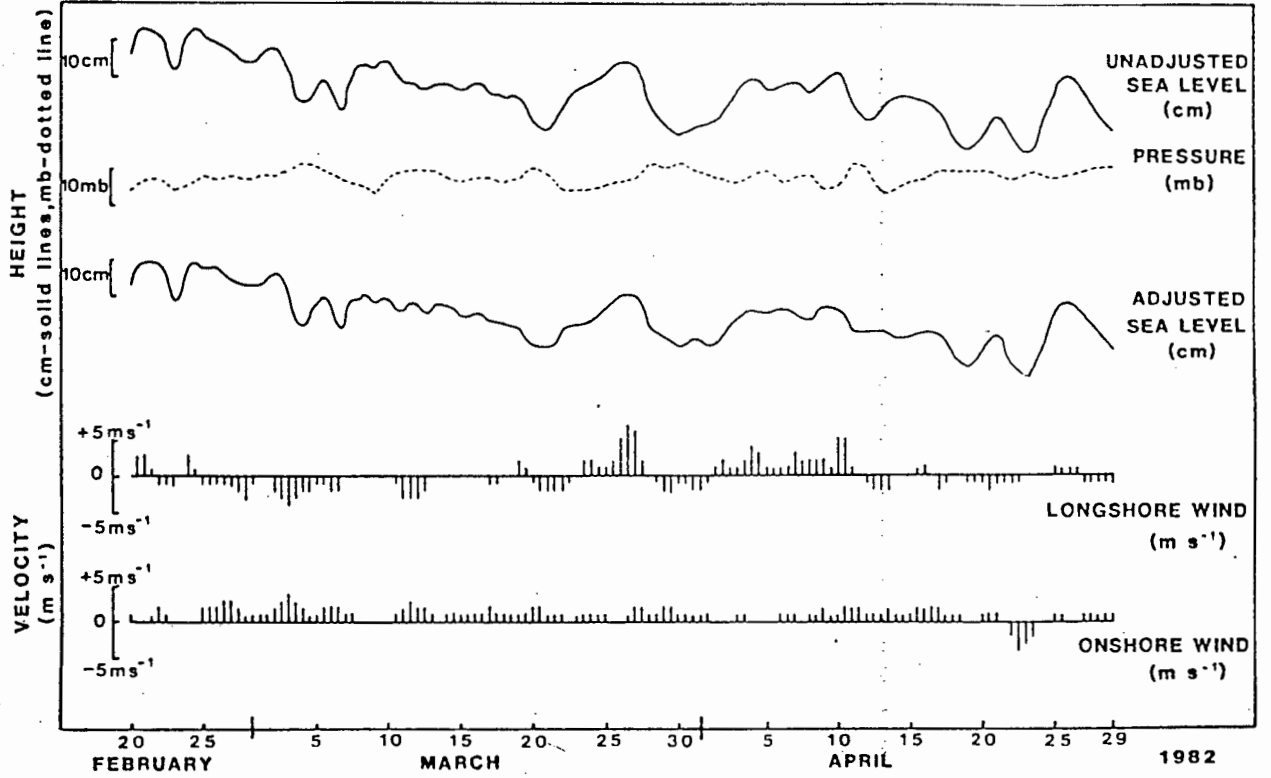
It can be seen from this relationship that wind slope is inversely proportional to water depth for a given windspeed. This implies that shallow embayments or inlets will be more prone to wind setup than rapidly shelving coastlines. The southern African coastline is primarily steep sloped resulting in relatively deep water close inshore. The impact of wind setup is therefore not particularly noticeable in this region. Locations such as Saldanha Bay and False Bay are the most likely areas in this region where wind setup could play a significant role in the determination of design sea level. De Cuevas (1985) analyzed wind, barometric and sea level records for weather stations at Lamberts Bay and Granger Bay on the west and south coasts respectively. These figures are reproduced in figure 4.7. It is clear from these data that no significant relationship between onshore wind and sea level rise exists for these ports. On this basis limited attention will be given to wind setup in this study.

4.2.5. *Tropical Cyclones*

Tropical cyclones, also known as hurricanes, normally occur, in the region to the north east of Madagascar. They generally move in a south westerly direction before finally turning back to the south east around 20–30° S SAN (1975). Occasionally these cyclones continue on a south westerly track and impact on the northern coast of Zululand and Mozambique. Tinley (1985) reports a large event in 1970 where substantial coastal erosion was recorded. SAN (1975) notes that the effects of tropical

FIGURE 4.7.

DAILY FILTERED SEA LEVEL, ATMOSPHERIC PRESSURE AND WIND COMPONENTS AT LAMBERTS BAY AND GANSBAAI



cyclones are from time to time recorded at Durban, stating they seldom impact on the coast. Tropical cyclones occur primarily from December to May with a frequency of 4 to 5 per annum in the western Indian Ocean (SAN (1975)).

Tropical cyclones are characterized by a well defined circular wind structure around a low pressure cell. Sustained wind speeds of up to 70 knots typify most events. Whilst the circular wind speeds are high, the net forward movement of the storm is relatively slow. The combined effect of intense wind stress and moving pressure induces significant water level fluctuations in the coastal areas. Numerous numerical models have been developed in this regard over the past 30 years (CERC (1984)).

Tropical cyclones are a rare occurrence in southern Africa. Very little work has been carried out to date to quantify the effects on sea level. They would appear to be relevant only in terms of coastal engineering structures on the north eastern coast of Natal.

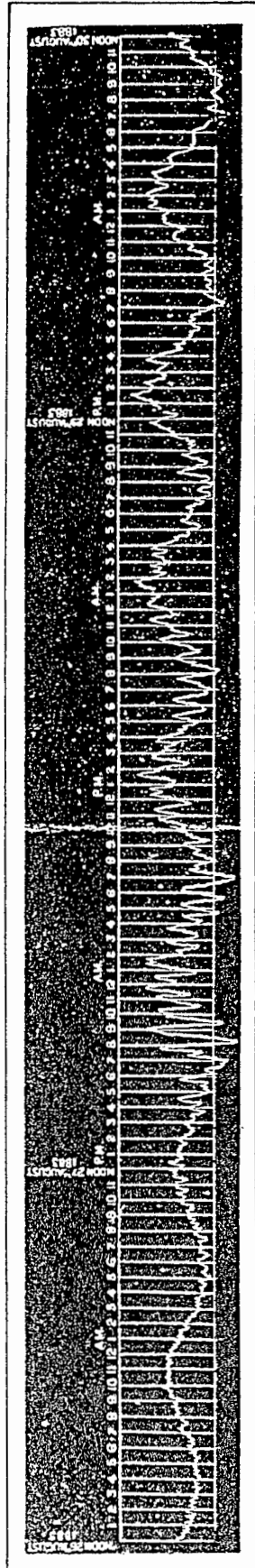
An assessment of sea level records for Richards Bay from 1977 - 1990 gave no clear indication of past events in spite of tropical cyclones having been recorded during this period. It would appear that resulting storm surges are localized and largely dependent on coastal configuration. The lack of substantial data and relatively localized nature of these events resulted in limited work being undertaken on tropical cyclones in terms of this study.

4.3. *TSUNAMIS*

The term tsunami refers to a gravity wave system, formed in the sea, following any large scale, short duration disturbance of the free surface. The major generating mechanisms relate to specific types of submarine earthquakes, landslides, turbidity currents and explosive volcanoes. Tsunamis have been responsible for significant destruction and loss of life throughout history for communities and facilities situated in coastal regions. South Africa has not been subject to large scale or severe tsunami events within the context of recorded history. The most notable event to date relates to the eruption of a volcano at Krakatoa in 1883 (see figure 4.8.). The tsunami measured approximately 0,75m at Port Elizabeth. Despite the apparent lack of records a number of tsunamigenic sources which could pose a potential threat to this coastline, can be identified.

FIGURE 4.8.

PORT ELIZABETH TIDE RECORD (1883)



Nature, 25 Oct, 1883

The Tidal Wave, South Africa

The vast majority of tsunamis are induced by rapid underwater tectonic disturbances resulting from submarine earthquakes. These events originate from vertical or dip slip motion of the sea bed. The parameters which are considered to affect the scale of the tsunami are focal depth, earthquake magnitude, maximum displacement, resultant ground displacement, orientation and shape and after shock area. There appears to be a threshold magnitude below which it is unlikely that a tsunami will be generated from a specific tectonic displacement. Uncertainty relating to the capability of various sources results largely from time scale differences between the existing historical records and the recurrence interval of these mechanisms which constitutes a significantly longer time period. Brandsma et al (1976) used two major criteria for identifying potential sources:

1. location in a major shallow seismic belt;
2. situated in one of the subduction zones where thrust faulting accompanies the under thrusting of an oceanic plate beneath a continental plate.

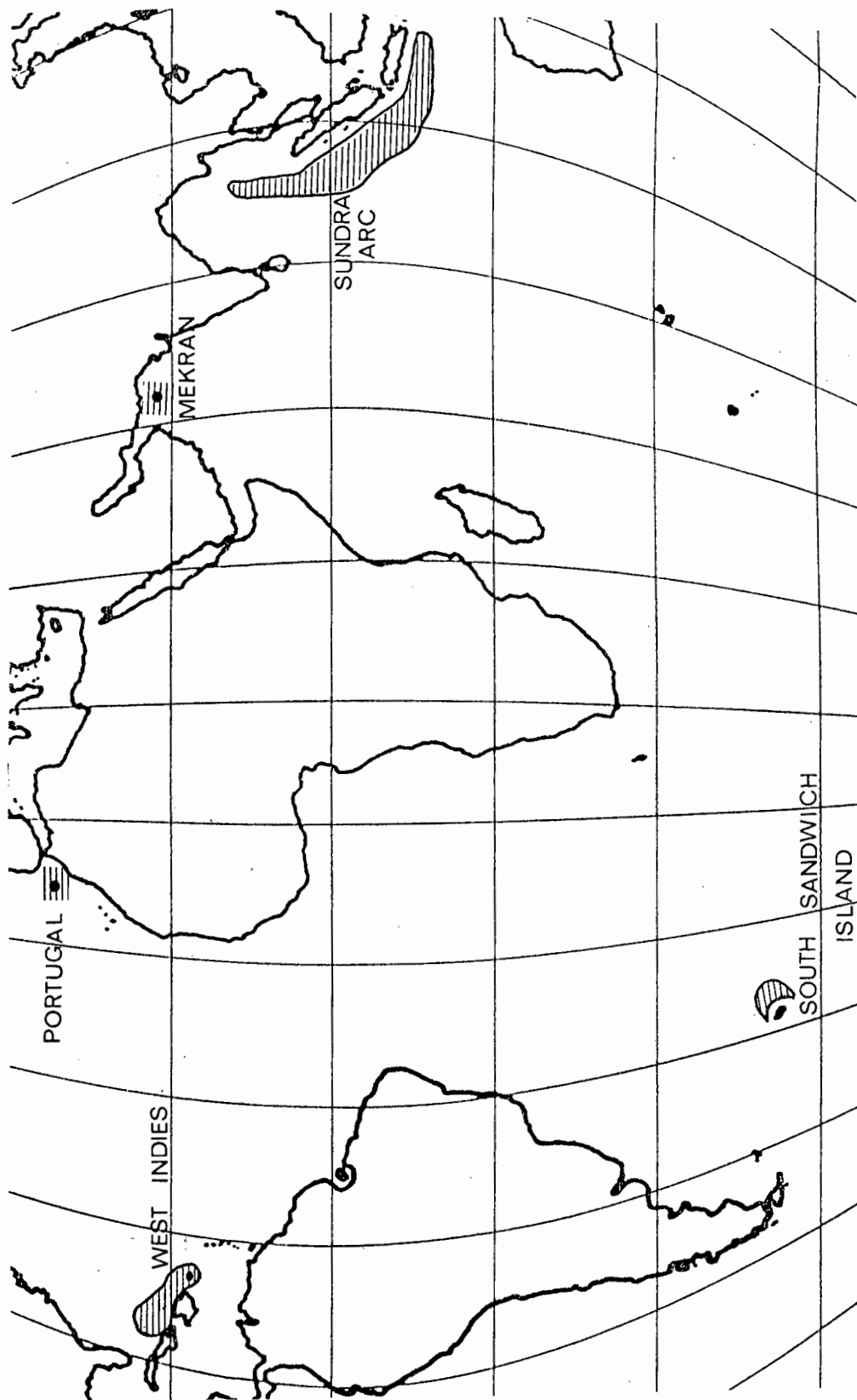
Figure 4.9. indicates tsunamigenic areas considered relevant to southern Africa.

Apart from earthquakes, non seismic events such as explosive volcanoes, submarine landslides and turbidity currents should also be considered. Volcanic explosions below the sea bed have been known to induce large scale tsunamis. These events are related primarily to central type volcanoes, such as Krakatoa (1883), where large quantities of material are blown away, inducing rapid movement of the sea surface. Areas in the East Indies and Azores are considered most pertinent to southern Africa. Submarine landslides and turbidity currents have induced tsunamis measured in the Mediterranean and East Atlantic ocean. These events can be related to the seismic activity in the region, slope steepness, stability of slope geology, major ocean currents and sediment distribution. Sources most likely to influence water levels will be situated close to the site, most probably just off the continental shelf. There is, as yet, little information available relating to these events in southern Africa.

Tsunamis are long period waves (5min-2hrs) which propagate at the shallow water wave celerity equal to the square root of gravitational acceleration multiplied by the water depth. Outside the generating area the short waves are dissipated by friction, breaking or non linear wave interaction resulting in a long wave train. Trans ocean

FIGURE 4.9.

TSUNAMI SOURCE MECHANISMS FOR SOUTH AFRICA



propagation may be simulated using two dimensional linear long wave computational models without compromising the physical characteristics of the wave system. Input to these models should be derived from a tsunami generating model on the outer boundaries of the source area. The results represent the boundary conditions for a shelf or nearshore transformation model. A tsunami approaching and travelling on the continental shelf is subject to a complex transformation which is not readily modelled using linear techniques. For the longer wave periods (+ 1hr) a degree of reflection is possible from the shelf transition which effectively reduces the energy transmission. Different coastlines will therefore result in different wave energy characteristics depending on the nature of the shelf topography.

Tsunamis represent rare events in Southern Africa. Whilst they may be of interest to sensitive coastal installation, such as nuclear power stations, general engineering design would not require the evaluation of these phenomena. As in the case of wind setup and tropical cyclones, tsunamis will not be considered as significant in terms of this work.

SUMMARY

This chapter reviewed the ocean processes affecting sea level in Southern Africa. Two major categories of phenomena were identified as influencing sea level in this region, namely astronomical tide and meteorologically induced shelf waves, long period edge waves and wind waves. The actual measured characteristics of these phenomena will be investigated in chapter 6. The fundamentals of a probabilistic approach to design sea level are set out in chapter 5.

CHAPTER 5

A PROBABILISTIC APPROACH TO DESIGN SEA LEVELS

The objective of this chapter is to set out a framework within which the methods and theory applied in this thesis can be described. The primary purpose of this study is to develop a rational approach to the assessment of design sea level, with particular attention to the dominant causative processes affecting Southern Africa. The basis of this approach will be described in this chapter. Limit state design is explained with a view to illustrating the relevance of probabilistic applications in design practice. The Monte Carlo simulation technique, its basis and application is described as this technique is extensively used later in the study. Extreme value statistics plays an important part in the assessment of rare design events in chapter 6. The final section of this chapter considers the significance of design return period as used in engineering practice. Apart from defining the concept, the inherent limitations of this approach are illustrated. Return period will be used throughout this study as a measure of hazard or risk.

5.1. LIMIT STATE DESIGN

The limit state philosophy is a generally accepted approach to design in civil engineering practice. The objective is to ensure that there is a reasonable probability that the structure being designed will not become unfit for the use for which it is required (SABS (1980)). The development of codes of practice have generally centered around the specification of global factors of safety. These safety factors, more often than not, reflect the prevailing level of uncertainty or ignorance regarding the levels of loading, the geometric and material characteristics, the detailed response, and consequences of failure (Ellinas et al (1984)). This lack of sensitivity in design has led to the introduction of partial factors of safety. These are applied to loadings, geometric and material characteristics, and are generally based on the use of statistical data. As more research work is undertaken into these aspects of design, so codes of practice will evolve towards the quantification of overall exposure to risk for a particular structure. This procedure will involve the assessment of the stochastic nature of the load or demand on, and the strength or resistance of a structure. This can be formulated as follows :

$$\begin{aligned} F &= R_f - D_f \\ P_f &= P(F < 0) \end{aligned}$$

where

R_f	=	Resistance or strength of the structure
D_f	=	Demand or load on the structure
P_f	=	Probability of failure of the structure
F	=	Failure

(Geustyn (1987))

Probabilistic design recognizes the inherent variability and stochastic nature of the components affecting both load and resistance. A framework has been developed in order to incorporate different levels of assessment of the reliability (the inverse of the probability of failure) of a structure based on the level of information available. Mol et al (1984) and Tholf-Christensen and Baker (1982) refer to the following levels of design:

Level I QUASI-PROBABILISTIC APPROACH

Design methods in which appropriate degrees of structural reliability are provided on a structural element basis by the use of partial safety factors related to pre-defined characteristics or nominal values of the major structural and loading variables. Most limit state design codes currently in use apply this approach. SABS (1980) represents an appropriate example.

Level II PROBABILISTIC APPROACH

Design method involving the simplification (normally linearization) of the failure function in order to calculate the probability of failure of a structure. The level II approach considers the load and resistance as stochastic variables. Mol et al (1984) describes several methods using the level II approach. These are the mean value method which uses the linearization of the mean values in the failure function, the advanced first order moment approach where the failure function is linearized in its point of maximum probability density and the advanced full distribution approach (AFDA) where the actual distribution is approximated by a normal distribution with the same density in the design point.

Level III FULL PROBABILISTIC APPROACH

Design method requiring the exact calculation of the probability of failure for a structure based on the full probability description of the load and resistance function. Mol et al (1983) represents the convolution function as :

$$P(F < 0) = \int_0^{\infty} f_r(\phi) F_\ell(\phi) d\phi$$

where $f_r(\phi)$ = probability density function of resistance
 $F_\ell(\phi)$ = cumulative probability distribution function of the load ℓ

Geustyn (1987) presents an example of the application of this approach to the assessment of the probability of failure of a submarine pipeline in intermediate water depths.

5.2. APPLICATION TO DESIGN SEA LEVEL

The determination of design sea level represents, in effect, the quantification of the load or demand (D_f) imposed on the structure, process or system. The primary objective of this thesis is to develop a probabilistic methodology to evaluate the load component to enable a level II or III approach to be implemented in coastal engineering design practice. Conventional design practice makes use of certain rules of thumb to determine what combination of phenomena should be used for assessing design levels. These general rules are based on past experience which, in many instances, has been derived from work undertaken in entirely different environments. The nuclear industry has produced a number of guidelines in this regard where actual recurrence periods for different combinations of hazards are recommended. (IAEA-50-SG-S10B(1983), ANSI/ANS-2.8(1981)). Although these guidelines are useful they provide no indication of the actual exposure to risk. It is within this framework that a new approach is proposed whereby the risk will be assessed using all the available information for a particular location.

The first step in the process is to define the various phenomena affecting sea level in a specific region. This has been set out chapter 4 and can be summarized as follows:

$$Z(t) = f(S_t, S_s, S_w, S_e, S_{ts}, S_{wd})$$

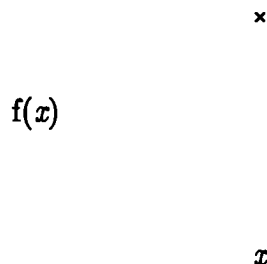
where	Z	=	Prevailing sea level
	S _t	=	Astronomical tide level
	S _w	=	Wave setup level
	S _s	=	Shelf wave level
	S _e	=	Edge wave level
	S _{ts}	=	Tsunami level
	S _{wd}	=	Local wind setup level

In each instance a probability density function (pdf) can associated with a particular natural phenomena for a specific location along the coast. These pdf's are then convolved, taking into account possible inter dependencies, in order to determine the combined probability of occurrence or exceedance of a specified sea level. The methods used to combine these pdf's include numerical, Monte Carlo simulation, analytical and Taylor series techniques. Together with appropriate extreme value statistics these methods are used to develop a generalized probabilistic model for assessing the nature of the combined distribution.

The generalized formulation of the probability of non exceedance of a level x can be given as follows :

$$F_x(X \leq x) = \int_{-\infty}^x f_x(x) dx \quad (1)$$

This can be illustrated graphically as



The distribution function is based on the assumption that:

1. $F_x(-\infty) = 0; F_x(+\infty) = 1.0$
2. $F_x(x) \geq 0$, is non decreasing with x
3. $F_x(x)$ is continuous with x

(Ang & Tang 1975)

Accordingly the probability of exceedance can be written as the inverse of non exceedance:

$$F_x(X > x) = 1 - F_x(X \leq x) \quad (2)$$

The combination of pdfs must satisfy the overall conditions relating to the distribution, therefore for two random variables X and Y where:

$$Z = g(X, Y)$$

if $Z = X + Y$

then $F_z(z) = \int_{-\infty}^{\infty} \int_{-\infty}^{g^{-1}} f_{x,y}(x, y) dx dy \quad (3)$

where: $g^{-1} = g^{-1}(z, y)$

thus: $F_z(z) = \int_{-\infty}^{\infty} \int_{-\infty}^z f_{x,y}(g^{-1}, y) \left| \frac{\partial g^{-1}}{\partial z} \right| dz dy$

The probability density function for z is:

$$f_z(z) = \int_{-\infty}^{\infty} f_{x,y}(g^{-1}, y) \left| \frac{\partial g^{-1}}{\partial z} \right| dy \quad (4)$$

If x and y are statistically independent then:

$$f_z(z) = \int_{-\infty}^{\infty} f_x(z-y) f_y(y) dy \quad (5)$$

or in terms of x :

$$f_z(z) = \int_{-\infty}^{\infty} f_x(x) f_y(z-x) dx$$

$$F_z(z) = \int_{-\infty}^{\infty} \int_{-\infty}^z f_x(x) f_y(z-x) dx dz$$

If sea level is considered a function of four different processes such that:

$$Z = S_t + S_w + S_s + S_e$$

Then the generalized form for the new CDF can be written as:

$$F(z) = \int \int \int_{-\infty}^{\infty} \int_{-\infty}^{g^{-1}} f_{T,W,S,E}(t,w,s,e) dt dw ds de$$

The approach followed, when convolving the different pdfs, is dependent on the complexity of the distribution function and the degree of inter dependency between the variables. In this study the Monte Carlo simulation technique presents the most useful method for combining pdfs. The following section sets out the basic principles pertaining to the application of this approach.

5.3. MONTE CARLO SIMULATION

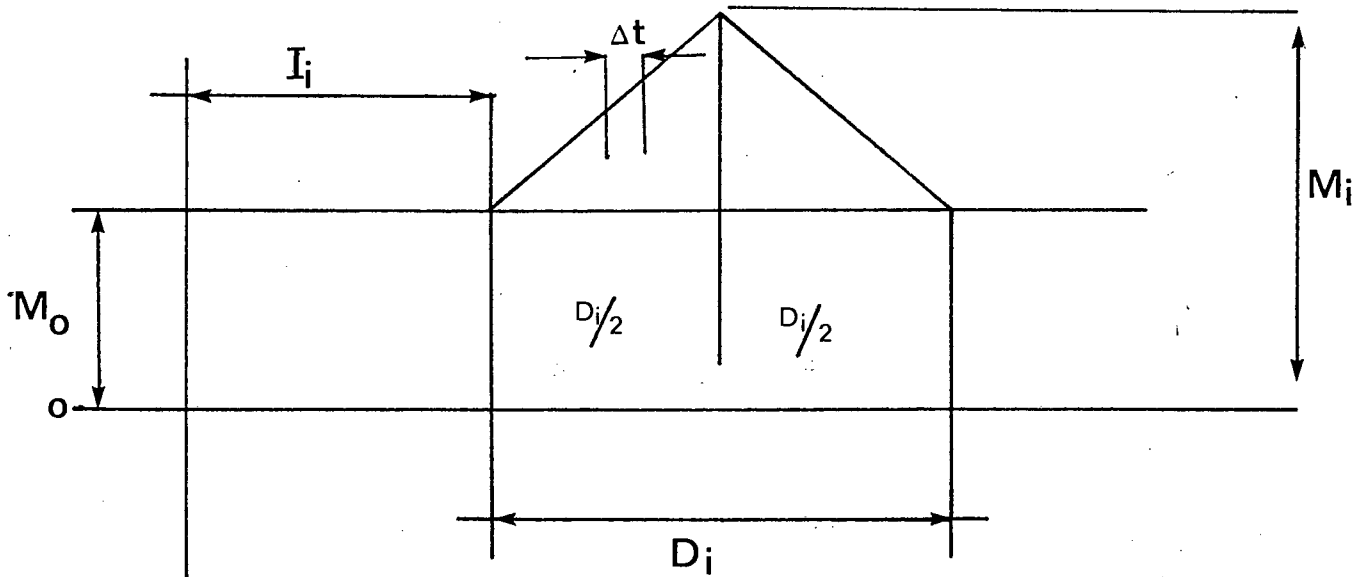
The term "Monte Carlo" was introduced by Von Neumann and Ulam during World War II as a code word for secret work on the atomic bomb project at Los Alamos. (Rubinstein (1981)). The method was used as early as 1908 by Student to estimate the correlation in the t-distribution. Monte Carlo simulation represents a special type of numerical method. (Naylor et al (1966) c.f. Rubenstein (1981)) describes simulation as a numerical technique for conducting experiments on a digital computer, which involves certain types of mathematical and logical models that describe the behaviour of business or economic systems over extended periods of real time. Monte Carlo simulation is a form of stochastic simulation. Stochastic simulation relates to

experimenting with a model, of a system, over time by sampling stochastic variables from probability distributions. Rubenstein (1981) lists the differences between Monte Carlo methods and normal simulation as follows:

1. In the Monte Carlo method time does not play as substantial a role as it does in stochastic simulation.
2. The observations in the Monte Carlo method, as a rule, are independent. In simulation the observations are serially correlated.
3. In the Monte Carlo method it is possible to express the response as a simple function of the stochastic input variables. In simulation the response is usually complex and can be expressed explicitly only by the computer program itself.

The Monte Carlo method provides a technique of generating random numbers according to a given probability function. The process may be repeated any number of times resulting in a synthetic data set which reflects the inherent statistical properties of the original probability distribution function.

The modelling approach taken in this study is based on a special form of regenerative process. Three components of a process are considered, namely : Magnitude - ($M(t)$); Duration - ($D(t)$) and Interval - ($I(t)$). Each component represents a stochastic process $\{M(t):t \geq 0\}$, $\{D(t):t \geq 0\}$, or $\{I(t):t \geq 0\}$ based on different probability distributions. Provision is made for a conditional probability between magnitude and duration. Single events may be graphically represented as follows :



A triangular distribution is assumed as representative of the event time history. The initiation of a particular event $E(t)$ is marked by the termination of the preceding time interval I_i . The event magnitude time history from this point is serially correlated at time intervals Δt according to the following formulation :

$$\begin{aligned} M(t) &= M_0 + 2 \left(\frac{M_i - M_0}{D_i} \right) (t) && \text{for } 0 \leq t < D_i/2 \\ M(t) &= 2M_i - M_0 - 2 \left(\frac{M_i - M_0}{D_i} \right) (t) && \text{for } D_i/2 < t \leq D_i \end{aligned}$$

Each singular event may be considered as independent and identically distributed in terms of its parameters M_i , D_i and I_i . The stochastic process $\{M(t), t \geq 0\}$ is generated over a period of one year after which it is combined at Δt intervals with other deterministic and stochastic processes. The model is run over a period of 100 years which represents one complete cycle. Experiments consist of 5-10 cycles, the annual maximum values are sorted and then averaged.

In most instances Magnitude (M_i) and Duration (D_i) are jointly distributed random variables, where $i \in \text{Integer}$. According to Ang and Tang (1984) the joint probability distribution can be written as follows:

Let X_1, X_2, \dots, X_n be a set of n random variables:

$$f_{x_1 \dots x_n}(x_1 \dots x_n) = f_{x_1}(x_1) f_{x_2}(x_2/x_1) \dots f_{x_n}(x_n/x_1 \dots x_{n-1})$$

where $f_{x_1}(x_1)$ is the marginal Pdf of X_1 ;
 and $f_{x_k}(x_k/x_1 \dots x_{k-1})$ is the conditional Pdf of X_k
 given $X_1 = x_1, \dots, X_{k-1} = x_{k-1}$

This corresponds to the cumulative distribution function (CDF)

$$F_{x_1 \dots x_n}(x_1 \dots x_n) = F_{x_1}(x_1) F_{x_2}(x_2/x_1) \dots F_{x_n}(x_n/x_1 \dots x_{n-1})$$

where $F_{x_1}(x_1)$ and $F_{x_k}(x_k/x_1 \dots x_{k-1})$ are marginal and conditional CDF's of X_1 and X_k respectively.

Thus for a set of uniform random numbers (n_1, n_2, \dots, n_n) the value X_1 may be determined by :

$$X_1 = F_{x_1}^{-1}(n_1)$$

and X_2 using the conditional CDF $F_{x_2}(x_2/x_1)$ such that

$$X_2 = F_{x_2}^{-1}(n_2/x_1)$$

This approach is used to generate values of event duration when they are dependent on the magnitude of the event. Thus if :

$$\text{then magnitude} = F_{\text{mag}}^{-1}(n_1)$$

$$\text{duration} = F_{\text{duration}}^{-1}(n_2/\text{magnitude})$$

A further condition of some interest to the modelling process is correlated sampling. This may arise when two separate processes need to be combined in order to obtain an extreme sea level. In most instances processes such as wind waves, shelf waves, edge waves and tides are considered as statistically independent. However, there may exist circumstances where this condition is not true, thus according to Ang and Tang (1984) for two processes A and B there exists a condition Z such that:

$$Z_A = g(A, X)$$

$$\text{and } Z_B = g(B, X)$$

$$\text{where } X = \text{set of random variables}$$

If these processes are summed then the mean values relate to

$$Z = Z_A + Z_B$$

and variance to

$$\text{VAR}(Z) = \text{VAR}(Z_A) + \text{VAR}(Z_B) - 2 \text{COV}(Z_A, Z_B)$$

If Z_A and Z_B are positively correlated then $\text{COV}(Z_A, Z_B) > 0$

the random numbers are generated by:

$$Z_{A_j} = g [A, F_{x_1}^{-1}(n_1), F_{x_2}^{-1}(n_2), \dots, F_{x_n}^{-1}(n_n)]$$

$$Z_{B_j} = g [B, F_{x_1}^{-1}(n_1), F_{x_2}^{-1}(n_2), \dots, F_{x_n}^{-1}(n_n)]$$

where n_1, n_2, \dots, n_n are independent uniformly distributed random numbers. The combination of correlated random processes in a stochastic simulation model requires a detailed definition of the exact nature of the inter relationships. These associations are expected to relate to inter event durations and magnitude.

5.4. EXTREME VALUE STATISTICS

Of primary interest in the determination of design sea level is the expected maximum and minimum or limiting state conditions. It is therefore the extreme value or tail values which are normally of most interest. Having established that one is operating within the ambit of extreme value theory it is possible to apply this approach to gaining more information. A brief review of the most relevant points are given here as they relate to their later application in chapter 6. Castillo and Sarabia (1992) propose the following general expression of order statistics.

Let $(X_1, X_2, X_3, \dots, X_n)$ be a random sample from a given population. If these values are ranked such that $[X_1 \leq X_2 \leq \dots, X_n]$ then the r th member, X_r of this new sequence is called the r th statistic, where X_1 and X_n represent the minimum and maximum extreme values. It is assumed that X_1, X_2, \dots, X_n are independent identically distributed random variables from a continuous parent population with a cumulative distribution function (CDF) $F(x)$ and a probability distribution function (PDF) $f(x)$.

To determine the probability distribution of order statistics Castillo and Sarabia (1992) propose the following generalized form of joint PDF for $X_{r_1}, X_{r_2}, \dots, X_{r_k}$; $r_1 \leq r_2 \leq \dots, r_k$ be k order statistic from random sample of size n . The PDF of this set of order statistics is :

$$f_{r_1, r_2, \dots, r_k}(x_1, x_2, \dots, x_k) = n! \prod_{i=1}^k f(x_i) \prod_{j=1}^{k+1} \frac{[F(x_j) - F(x_{j-1})]^{r_j - r_{j-1} - 1}}{(r_j - r_{j-1} - 1)!}$$

where

$$r_0 = 0; r_{k+1} = n+1; x_0 = -\infty, \text{ and } x_{k+1} = \infty$$

From this generalized form it is possible to determine the distribution of maxima, minima and rth order statistics. Ang and Tang (1984) shows that for maximum and minimum values where :

$r = n$ (maxima) the PDF and CDF of the sample maximum is :

$$f_n(x) = nF^{n-1}(x)f(x) \quad F_n(x) = F^n(x)$$

Similarly for the minimum PDF and CDF (where $r = 1$)

$$f_1(x) = n[1-F(x)]^{n-1}f(x) \quad F_1(x) = 1-[1-F(x)]^n$$

It has been shown that for large n the asymptotic distribution for the extremes tend to converge on particular limiting forms as $n \rightarrow \infty$. Fisher and Tippett (1928) and Leadbetter et al (1983) and Gumbel (1958) describe three types of non degenerative distributions, $H(x)$ satisfying the requirement :

$$\lim_{n \rightarrow \infty} H_n(a_n + b_n x) = \lim_{n \rightarrow \infty} F^n(a_n + b_n x) = H(x)$$

where $H_n(x) = \text{Prob}[Z_n \leq x] = F^n(x)$

for CDF $H_n(x)$ of the maximum values Z of sample size .

Limit distributions for Maxima can be summarized as follows :

Frechet	$H_c(x) = \exp(-x^{-c})$	if	$x \geq 0;$	$c > 0$
	$H_c(x) = 0$	if	$x < 0;$	$c > 0$
Weibull	$H_c(x) = \exp[-(-x)^{-c}]$	if	$x \leq 0;$	$c < 0$
	$H_c(x) = 1$	if	$x > 0;$	$c < 0$
Gumbel	$H_0(x) = \exp[-\exp(-x)]$		$-\infty < x < \infty$	

A similar exercise may be carried out for the minimum values (Castillo and Sarabia (1992)).

The implication is, therefore that for extremes, where n is large, the parent distribution $F(x)$ will tend towards one of the above distributions. The tendency of a parent distribution to converge towards one of these asymptotic forms is referred to as the domain of attraction for maxima and minima. Castillo and Sarabia (1992) present the following guide to identifying the domains of attraction of parent distributions :

- If $c > 0$, then $F(x)$ belongs to a Frechet type domain of attraction
- If $c = 0$, then $F(x)$ belongs to a Gumbel type domain of attraction
- If $c < 0$, then $F(x)$ belongs to a Weibull type domain of attraction

Castillo (1988) c.f.Castillo and Sarabia (1992) gives the following guidelines :

1. A parent distribution with non finite end point in the tail cannot lie in a Weibull domain of attraction.
2. A parent distribution with non finite end point in the tail of interest cannot lie in a Frechet type domain of attraction.

The following common distributions may be listed :

TABLE 5.1.

Parent Distribution	Extreme Maxima	Extreme Minima
Normal	Gumbel	Gumbel
Exponential	Gumbel	Weibull
Gumbel (Maxima)	Gumbel	Gumbel
Rayleigh	Gumbel	Weibull
Weibull (Minima)	Gumbel	Weibull
Uniform	Weibull	Weibull

(Castillo and Sarabia (1992))

The selection of a particular limit distribution is not always possible from the parent distribution, as the latter is not always known. Thus it is necessary to use different assessment methods such as the probability method, the weighted least squares method or the curvature method.

A problem common to most environmental processes, such as waves, wind and sea level variations, it is the lack of adequate long term data sets. Consequently methods developed to assess annual maximum values are seldom applied with any measure of confidence. In an attempt to gain more information from these limited data sets two methods, the Largest Order Statistics method and the threshold method, have been developed. The largest order statistics method is described by Smith (1986) and Tawn (1988). The method is an extension of the equation :

$$P [(M_m - b_m)/a_m \leq x] \rightarrow F(x) \text{ as } M_m \rightarrow \infty$$

to the asymptotic joint distribution of the r largest values from a sample of size M_m (r is fixed, whilst $M_m \rightarrow \infty$). The threshold method, also known as the partial duration series analysis method, moves away from the order statistics approach towards the definition of independent identically distributed random events. If a physical understanding of the process under consideration is known then it becomes possible to define the lower limits of the individual events. Manoha et al (1986), Smith (1988) and Salih et al (1988) use this approach.

Practical application of the threshold approach would appear to be fairly robust in terms of short duration or poor data sets. By placing physical limitations on the definition of events it is possible to produce a new set of independent identically distributed values. The approach proposed here makes use of the definition of a three parameter threshold limit for events. These thresholds are magnitude, duration and recurrence interval. It can be seen that the definition of a particular threshold level, such as magnitude, determines the nature and extent of the duration and recurrence interval data sets.

The procedure consists of selecting a series of limiting values with a view to obtaining the largest possible number of events. Each resulting data set is subject to a dependency or randomness test. The randomness test requires that :

- a. Successive events in a sequence are statistically independent.
- b. Generated values are uniformly distributed.
- c. Values do not repeat themselves within a given data set.

The data set meeting these basic criteria with the largest number of events is normally selected. The treatment of missing data and expected event recurrence interval requires special attention if the threshold analysis is used as this will affect distribution fitting and extrapolation of data. It is apparent that each individual set of thresholds will have unique characteristics which need to be considered when assessing particular design return periods.

In this study it is important that events which may affect sea level fluctuations are defined and combined with all other prevailing conditions. For example, astronomical tide should be combined with wind wave storm events and shelf wave events at certain levels. Relatively small wave storm events combined with spring tide conditions could represent a limiting design condition. Therefore apart from the physical limitations regarding the definition of an event it is necessary to define the minimum level at which a process will affect the combined loading condition.

5.5. PERSPECTIVE ON DESIGN RETURN PERIOD

It is common in engineering practice to relate design conditions to a specific return period. Examples of these may be considered as the 1/50 flood line level, 1/100 year design wave and the 1/5 year wind event. Whilst this form of hazard definition is generally used, it remains important to understand the exact meaning of return period in terms of exposure to risk.

Tawn (1988) defines return period, for a sequence of independent identically distributed random variables, as the expected waiting time between two independent events of the same magnitude.

$$\text{Return Period} = T(x) = 1/P(X > x) \quad (1)$$

where $P(X > x)$ = probability of exceedence of process X of level x .

The design return period, therefore, relates inversely to the probability of an event x equalling or exceeding a particular event magnitude or level. Hence the reference in design codes to a 1/100 year design condition as relating to a particular extreme loading condition. This annual connotation implies that X represents an annual maxima. As pointed out in the previous section there exist many instances where events are defined as those occurrences exceeding a specific threshold level. Consequently the concept of annual maxima must be altered to accommodate this condition. Hence equation (1) was altered to :

$$T(x) = r/P(X > x) = \frac{r}{1 - P(X \leq x)}$$

SANECOR (1985)

where r = expected interval between each independent event. (This is often taken as the recording interval between measured data points).

An alternative to this format is given by Tawn & Vassie (1989).

If $G(x) = (P^{N\theta_n}(x))$ from extreme value theory (Ang & Tang (1984)).

$$T(x) = \frac{1}{1 - G(x)} = \frac{1}{(1 - P^{N\theta_n}(x))} \doteq \frac{1}{N\theta_n (1 - P(x))} \quad \text{as } N \rightarrow \infty$$

where $N\theta_n$ = expected number of independent events per annum

$P(x)$ = distribution for all data

$G(x)$ = distribution for annual maxima

A further assumption of importance with regard to the determination of return period relates to the distribution function. In order to calculate a specific return period it is necessary to fit a suitable distribution function. The limitations relating the observed data set often introduce additional uncertainty as to the suitability of particular distribution functions. It is therefore necessary to apply confidence limits to the prediction based on the type of distribution and number of available data points.

The relevance of return period in design depends entirely on the structure under consideration, in particular, its sensitivity to the recurrence of the design conditions. If the encounter probability is defined as the probability that the design conditions will be equalled during the life of the structure such that :

$$\begin{array}{l} \text{Encounter probability} \\ \text{where} \\ \text{then if} \end{array} \quad \begin{array}{l} = \\ L \\ T(x)/L > 1 \end{array} \quad \begin{array}{l} = \\ = \\ \end{array} \quad \begin{array}{l} 1 - \left[1 - \frac{1}{T(x)} \right]^L \\ \text{design life of structure} \end{array}$$

the encounter probability (E) can be written as

$$E \quad \doteq \quad 1 - \exp(-L/T(x))$$

It can be seen from the above formulations that if the expected life of the structure is taken as the return period then there exists a 63% probability of encountering these conditions in the life of the structure. It is important that the designer appreciate the significance of this fact when assessing overall exposure to risk, where risk is defined as :

$$\text{Risk} = \text{Encounter probability} \times \text{consequences of failure.}$$

For structures which are designed for single sea level or wave conditions, such as an offshore platform or structures designed for a particular number of occurrences, it becomes important to assess the encounter probability rather than the return period. This study will make use of the return period concept within this limiting framework with a view to complying with existing engineering practice.

SUMMARY

This chapter has set out in broad terms the theoretical framework which will be applied for the data analysis and the development of a convolution model. Some attention has been given to the interpretation of results in terms of the definition of return period. Chapter 6 considers the detailed analysis of the data which were available for different regions in Southern Africa.

CHAPTER 6

DATA ANALYSIS

Environmental data pertaining to sea levels around the coast of southern Africa, were obtained from a number of institutions. The available data sets are given in table 6.1. A significant amount of these data have been used by previous researchers to form a basic understanding of the processes affecting sea level. The approach taken in this study has been to subject each data set to a standard analysis procedure. Figure 6.1. illustrates the procedure in the form of a flow diagram. The available data are discussed under the headings used in chapter 4 to describe ocean processes. Various cross correlation techniques are used to identify and evaluate potential dependency relationships between different processes, especially those pertaining to meteorological events.

The data analysis has been restricted to three ports in southern Africa. These ports were selected due to their locations on the east, south, and west coasts in order to assess the regional characteristics of the sub continent. All available data has been analyzed for astronomical tides, shelf waves, wave climate and edge waves. Further attention is given to possible interrelationships between processes such as shelf waves, wind waves, wind speed, and edge waves.

6.1. ASTRONOMICAL TIDE

Astronomical tide is a deterministic component of sea level which may be calculated for any location along the coastline. Tidal levels are calculated using at least 366 days of good hourly sea level readings for the site under consideration. The data are filtered to remove all non tidal (residual) components prior to determining the harmonic constituents associated with that record. For the purpose of this study, it was necessary to generate at least 18,6 Julian years of hourly tidal levels for each site. The **TOGA** Sea Level Centre/National Oceanographic Data Centre Software Package (Caldwell and Kilonsky) (1977) model, which is described in more detail by Foreman (1977), was used. The package calculates 68 tidal constituents which are then used to predict hourly levels.

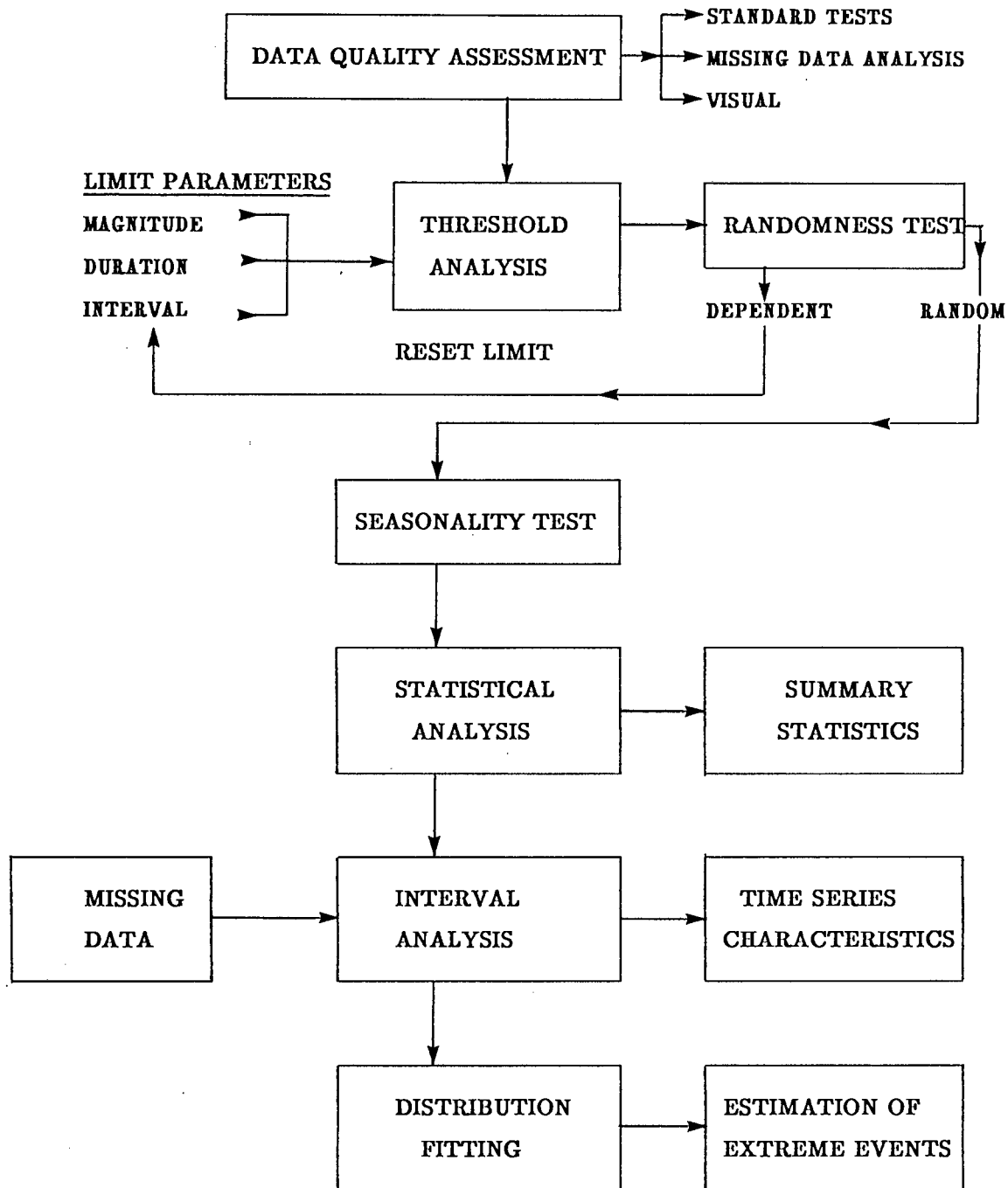
A verification program available from the SAN Hydrographer was used to assess the **TOGA** package. The SAN model is updated on an annual basis and therefore includes longer term constituents.

TABLE 6.1**SUMMARY OF AVAILABLE DATA SETS**

TYPE	LOCATION	SOURCE	PERIOD (years)	LENGTH (hours)	BAD DATA (hours)
Hourly Sea Level	Mossel Bay	UCT	1980-88	78912	7920
Hourly Sea Level	Richards Bay	SAN	1977-83 1989-90	78864	20936
Hourly Sea Level	Port Nolloth	SAN	1958-90	289422	21501
Hourly Sea Level	Simons Bay	UCT	1970-88	166554	10788
Six Hourly Wave	Mossel Bay	CSIR	1978-91	114855	40701
Six Hourly Wave	Richards Bay	CSIR	1980-92	108092	52591
Six Hourly Wave	Port Nolloth	CSIR	1987-92	45195	8072
Six Hourly Wave	Gouriqua	AEC	1986-90	32340	9867
Hourly Wind Speed	Gouriqua	AEC	1987	8760	1230

FIGURE 6.1.

FLOW DIAGRAM
DATA ANALYSIS



Complete hourly sea level data sets for Mossel Bay are available for 1980, 1982 and 1985. The TOGA model requires only one complete year of hourly sea level data in order to calculate the specific tidal constituents. Hourly predictions were generated for 1992 using 1980 and 1982 tidal constituents independently. The results obtained from both runs were not identical in every respect.

In order to compare the prediction given by the TOGA model, using 68 tidal constituents, and the SAN model, using 54 tidal constituents, predicted data for 1992 were compared. The following results were obtained (see table 6.1.1.).

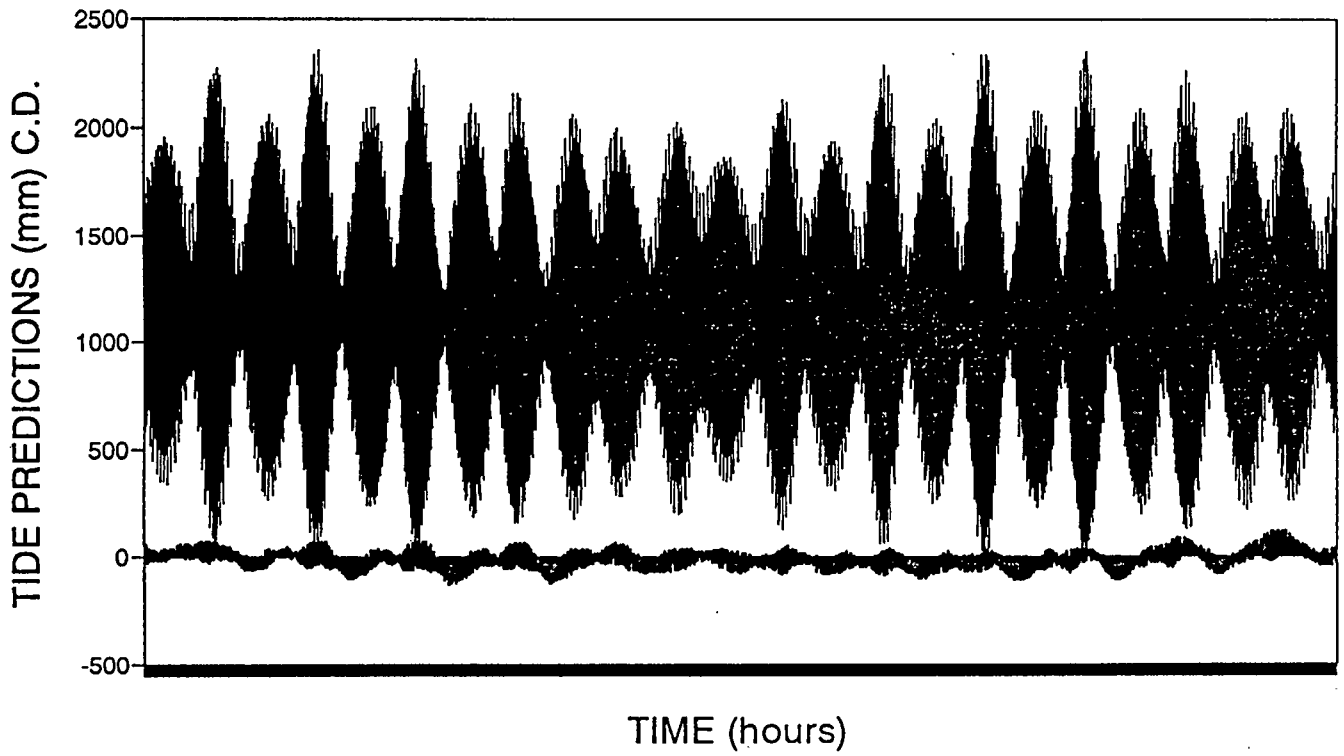
TABLE. 6.1.1.

COMPARISON BETWEEN TOGA AND SAN MODEL : for 8192 hourly values

	TOGA PREDICTIONS		SAN PREDICTIONS	RESIDUALS	
	1982	1980		SAN-TOGA 80	SAN-TOGA 82
Mean	1097.60	1118.80	1127.30	8.45	29.668
Std	482.69	483.25	481.74	48.41	31.39
Max	2299.00	2361.00	2359.00	161.00	131.00
Min	-84.00	-116.00	0.00	-109.00	-69.00

The residuals represent the difference between the SAN and TOGA models. From the above results it would appear that the SAN model has an expected positive shift of 29.6mm with regard to the TOGA model. It is obviously most important that the correct tidal values are predicted as these values are used to calculate the tidal residual component from the observed records. A shift of this magnitude will undoubtedly affect the resulting data set. The following graphs illustrate the nature of the problem. Figure 6.1.2. shows a time series plot of the superimposed hourly prediction and the associated magnitude of the residual value. A visual inspection confirms that the relative magnitude of the residual is small. Figure 6.1.3. displays the relationship between TOGA and SAN values giving some indication of the variance whilst figure 6.1.4. illustrates the hourly time series of residual values. The nature of

FIGURE 6.1.2
TOGA vs SAN MODELS



▨ SAN-TOGA PREDICTION — SAN PREDICTIONS

FIGURE 6.1.3
TOGA vs SAN PREDICTIONS

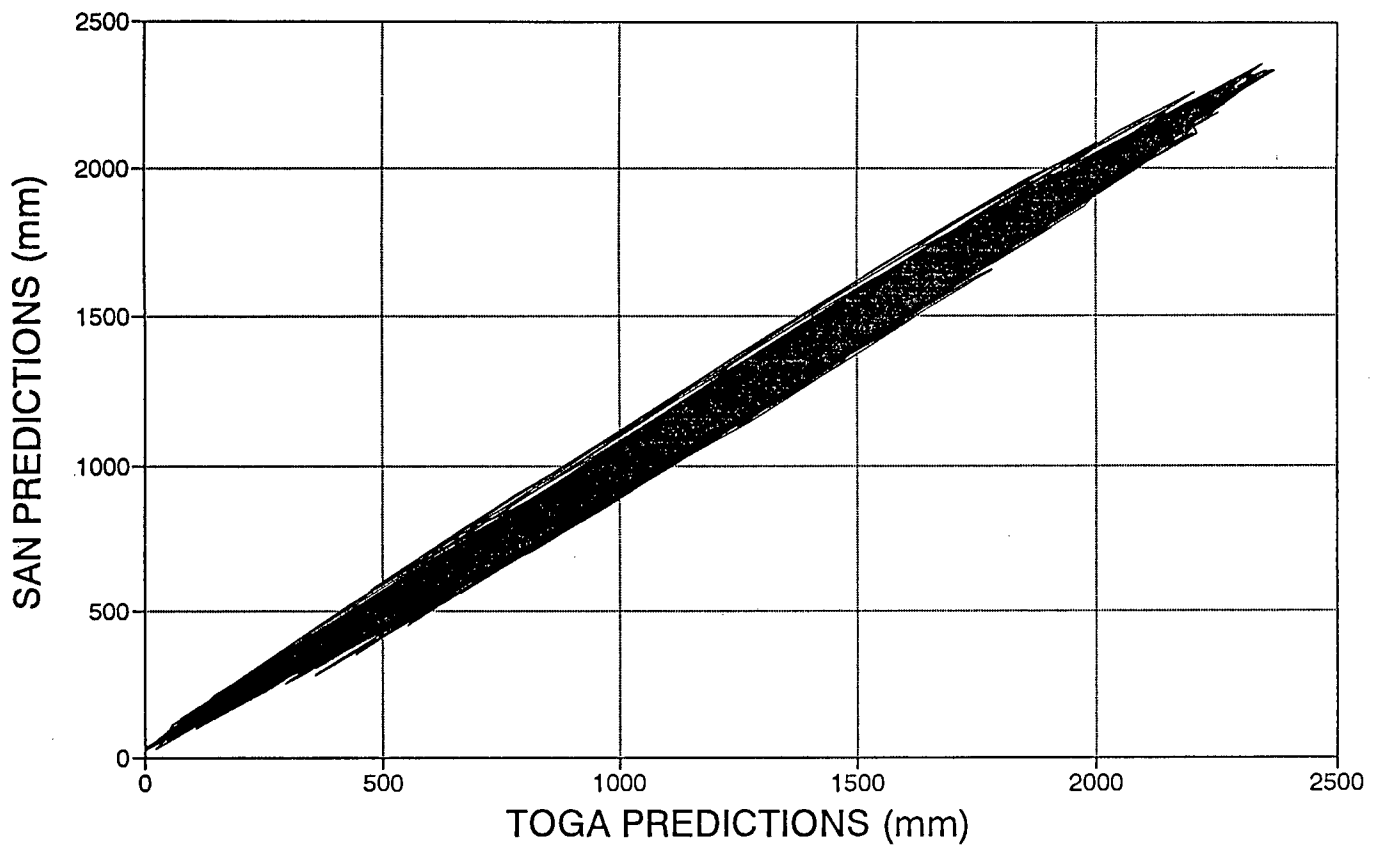
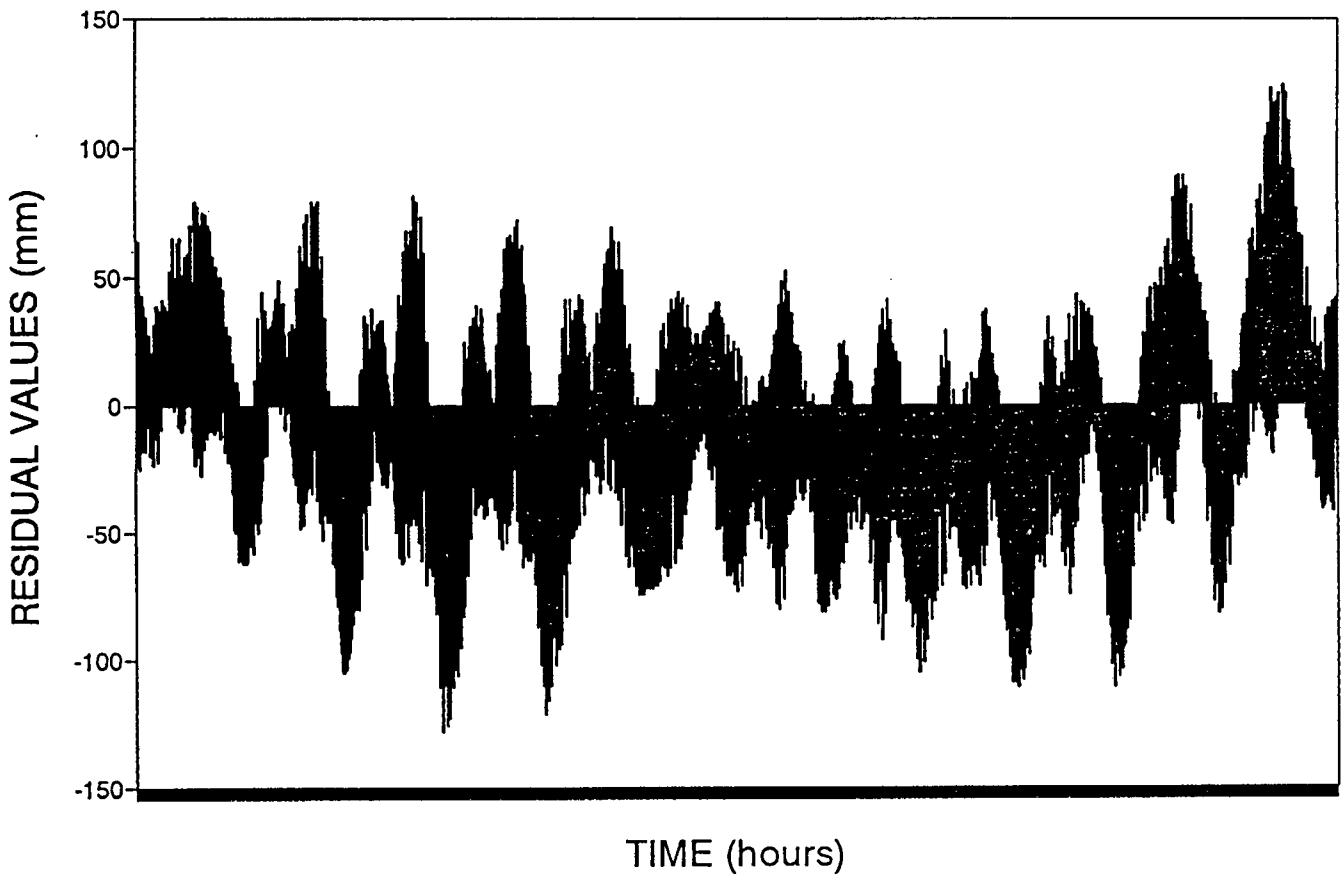


FIGURE 6.1.4
RESIDUAL VALUES



the residual appears to have significant frequency components which would indicate differences in the tidal constituents used in addition to the apparent datum shift.

The discrepancies noted between the TOGA prediction and the SAN model would appear to relate to a combination of different tidal constituents and the mean datum level used. De Cuevas (1985) mentions that the expected seasonal range, for all ports, in mean sea level varies between 20–90mm per year. In order to investigate the effect of a varying mean datum level the tidal prediction for 1992, based on 1988 tidal constituents were used in conjunction with a shifted mean of 112.9cm. The resulting residual values (between the TOGA and SAN model) improve. However, maximum predicted results increase by 53 mm which would not appear realistic. This approach was not pursued further.

In terms of this study it is important that there be consensus on the method of tidal prediction used and the results obtained. In order to ensure that the TOGA and SAN models gave equivalent values, within defined limits, a standard year of SAN predicted values for 1992 were analyzed using the TOGA analysis routine in order to calculate the relevant tidal constituents. From these values TOGA predictions were made for 1992 and compared to the original SAN values. The results can be summarized as follows :

TABLE 6.1.2.

NEW TOGA vs SAN RESULTS (8192 values)

	TOGA (using SAN)	SAN	RESIDUALS
Mean	1123.00	1127.00	4.43
Std	481.58	481.74	4.30
Max	2350.00	2359.00	26.00
Min	-14.00	0.00	-13.00

The difference can once again be attributed to the difference between the methods used to calculate the tides. In order to compare the maximum value generated by the model for years where these values were previously calculated, the following predicted values were computed :

TABLE 6.1.3.

TOGA vs SAN PREDICTIONS

YEAR	TOGA	SAN	RESIDUALS
	(using SAN 92 constituents)	MAX	
	MAX	MAX	
1988	2406	2420	-14mm
1989	2352	2390	-38mm
1990	2297	2320	-33mm
1992	2350	2359	-9mm
1993	2378	2380	-2mm

It can be seen that these results are within acceptable limits. This method was deemed adequate for the purpose of generating the 18.6 year record required for modelling tidal levels.

A further test was undertaken on the TOGA prediction model to investigate the nature of the 18.6 year nodal cycle. Hourly values for 1980, 1997 and 1998 were compared. It was found that over an 18,64 year period the cycle repeats itself, however although the general characteristics are the same (in terms of magnitude and phase) the hourly values are not exactly the same. This can no doubt be attributed to longer term constituents in the model. If the hourly values for a 37.2 year period are assessed, it can be clearly seen that an 18.6 year deterministic cycle exists. This observation in the predicted tidal values can be underpinned by the theoretical analysis of tides as described by Lamb (1945) and Muir-Wood and Fleming (1981). Of prime importance to a study of this nature is the fact that the process of interaction between the earth, the sun and the moon is deterministic and repeats itself over a period of 18.6 years. By definition, therefore, a well defined dependency structure exists within these data which requires that any point within the 18.6 year cycle is uniquely related to every other point within that period. The dependency structure of astronomical tide dictates to a large extent the manner within which tides and the stochastic processes, such as shelf

waves and wind waves are combined. The distribution for tidal events is represented by an 18.6 year hourly time series of values of interrelated events over a given period. Any convolution process must, therefore, take place, at the very least, over this period in order to retain unbiased combinational characteristics.

An aspect of interest regarding the analysis of the tides for Mossel Bay relates to the maximum and minimum annual predicted tidal values for this port. The distribution for maximum tidal values is given in Figure 6.1.5. These values may be related to specific return periods which attain a maximum at 1 : 18.6 years. These results in isolation do not have particular quantitative significance as tides follow a deterministic cycle which defines the exact recurrence interval between events. Table 6.1.4. lists the most notable return period values.

TABLE 6.1.4.

EXTREME PREDICTED TIDAL VALUES FOR MOSSEL BAY
(related chart datum)

RETURN PERIOD (yrs)	MAXIMUM (mm)	MINIMUM (mm)
5	2399	11
10	2410	- 30
15	2415	- 53
18.6	2420	- 81

Time series plots of the 100 years annual maximum and minimum values illustrate the 18,6 year cycle and hence the deterministic nature of tides (figures 6.1.6 and 6.1.7.).

The same tidal analysis and prediction procedure was undertaken for Richards Bay and Port Nolloth. In both cases the tidal constituents were used to generate predicted tidal values for all those years for which sea level records were available. A summary of the main tidal characteristics for Mossel Bay, Richards Bay, Port Nolloth and Simons Bay is given in table 6.1.5.

FIGURE 6.1.5
DISTRIBUTION FOR ANNUAL MAXIMA

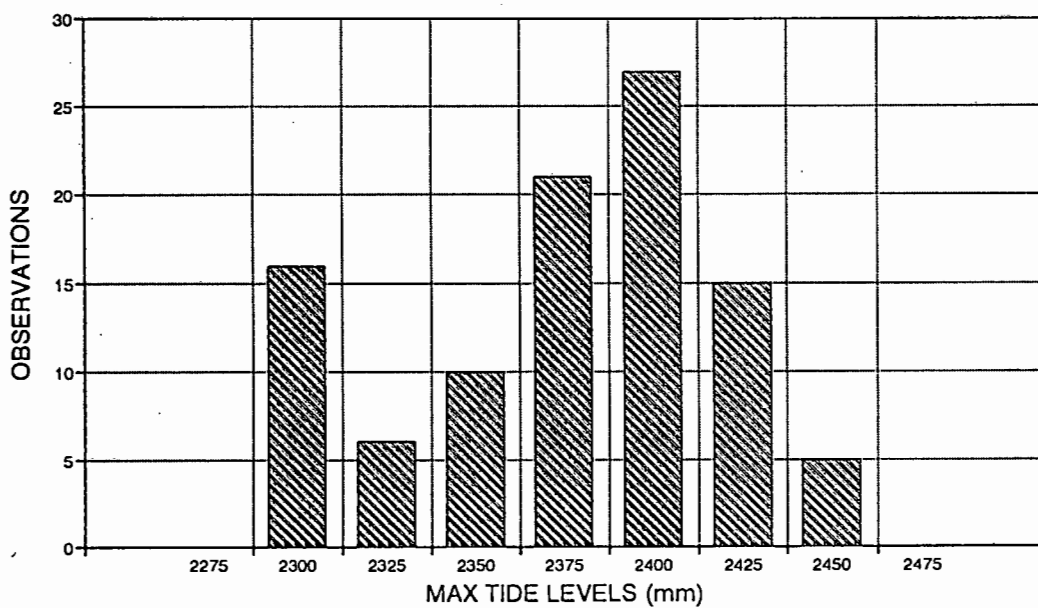


FIGURE 6.1.6
ANNUAL TIDAL MAXIMA TIME SERIES

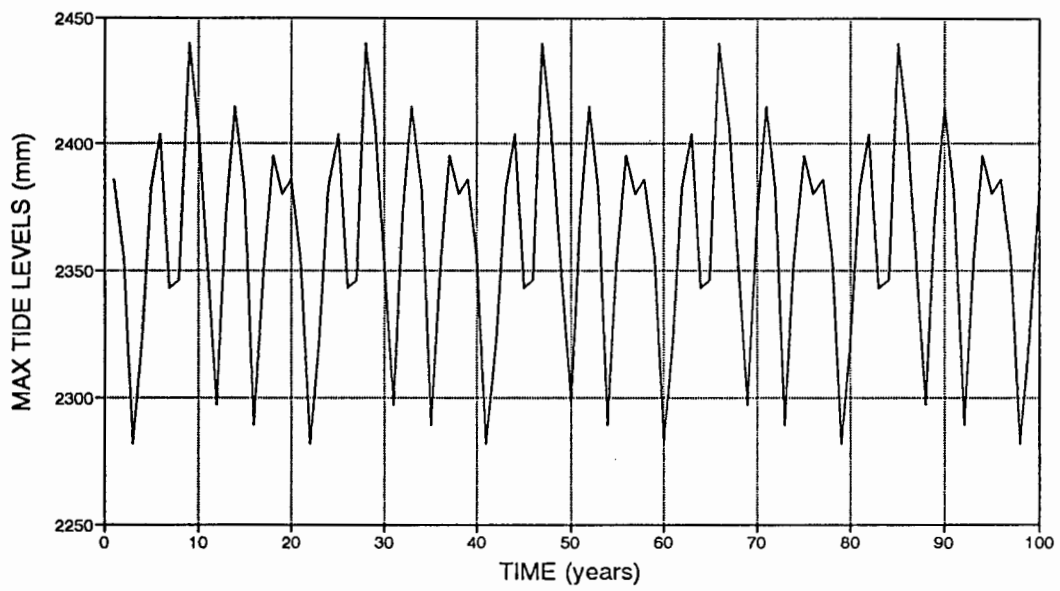


FIGURE 6.1.7
ANNUAL TIDAL MINIMA TIME SERIES

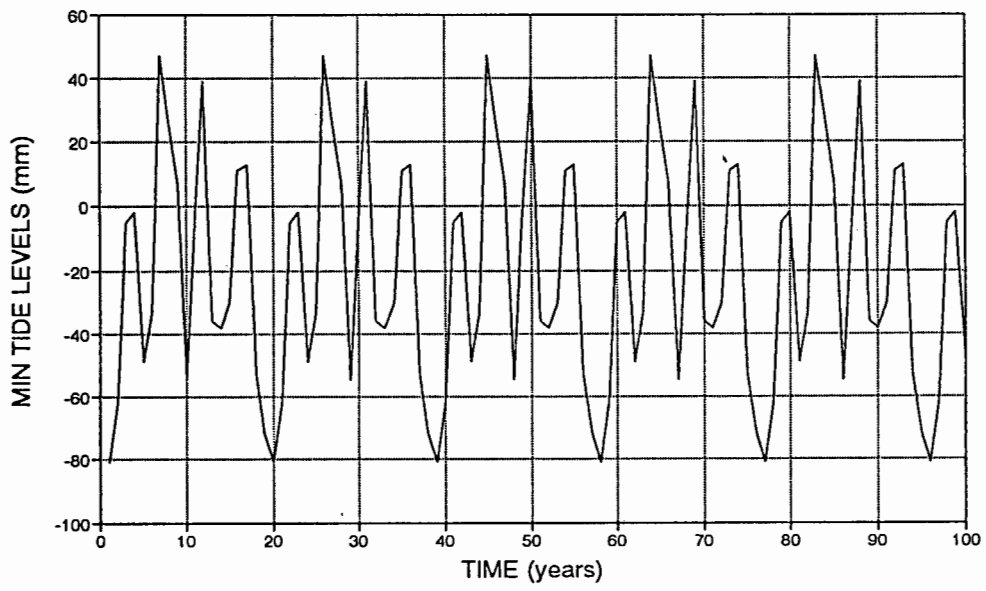


TABLE 6.1.5

SUMMARY OF TIDAL CHARACTERISTICS

LEVELS	RICHARDS BAY	MOSSEL BAY	PORT NOLLOTH	SIMONS BAY
LAT	-0.11	-0.01	-0.19	0.07
MLWS	0.19	0.25	0.09	0.32
MLWN	0.83	0.84	0.55	0.78
ML	1.09	1.13	0.87	1.06
MHWN	1.35	1.41	1.20	1.34
MHWS	1.99	2.00	1.66	1.80
HAT	2.37	2.42	2.02	2.14
RANGE	2.37	2.42	2.02	2.14
MSL	0.90	0.90	0.90	0.90

6.2. SHELF WAVES

Measurements of shelf waves in southern Africa were obtained from the existing tide gauge records for the 3 ports around the coast. The extent and quality of these records are variable but generally sufficient to characterize the nature of the events. The basic methodology followed for the analysis of shelf wave data can be summarized as follows:

1. All sea level data for a particular tide station were plotted out to form an hourly time series. From the time series plot it was possible to visually inspect the record for anomalies and lost data. Most data available for South African ports had been subject to quality control with missing data being denoted by 888 values.
2. Shelf wave data were obtained from the time series of tidal residuals. In this study tidal residuals were calculated by subtracting the predicted tidal values from the measured sea level values as recorded by the tide gauge.

Early studies on shelf waves in southern Africa had made use of the Doodson filter to calculate tidal residuals. The Doodson filter calculates the mean average over 39 hourly measurements. De Cuevas (1985) supplies more detail in this regard. The results from these studies gave some indication of the mean (or average) time series for tidal residuals. If the tidal values are considered as being the difference between the observed sea level and the tidal residual, it would appear that the Doodson filter approach overpredicts the magnitude of the tidal values. This can be illustrated by considering the extreme values predicted by the TOGA model, the SAN Hydrgrapher and the Doodson filter for 1985. These results can be listed as follows (see table 6.2.1.).

TABLE 6.2.1.

COMPARISON OF ANALYSIS METHODS

	MAXIMUM(mm)	MINIMUM(mm)
1. Doodson filter (max) (1985)	2450	-66
2. H.A.T. (SAN Hydrographer)(1988)	2410	-1
3. TOGA package (1985)	2404	-33

(Mossel Bay Data)

It can be seen that the Doodson filter tends to overpredict the values of the tide thus under predicting the tidal residual. This has fairly large implications when undertaking extreme value analysis on the data. The predicted tidal values are therefore used for the residual analysis.

3. In order to isolate independent shelf wave events it was necessary to undertake a partial duration series or threshold analysis on the residual time series. The event duration, magnitude and recurrence interval thresholds may be set. The most important criteria here was to isolate events resulting from the same forcing mechanisms, which were large enough to have some impact on the combined effects of tide, shelf waves and other processes.
4. The definition of independent events was made in terms of the recurrence interval, the magnitude and the duration. The threshold analysis had to take all these parameters into consideration based on an understanding of the natural process under review. In the case of shelf waves, the following information was known :
 1. Event magnitudes in excess of 100 mm will be the only events in combination with sufficiently large tides, to affect the maximum sea levels.

2. Events shorter than 7 hours are not likely to be induced by macro atmospheric phenomena related to shelf waves.
3. Intervals between events are generally in excess of 12 hours. This can be inferred by considering the average propagation speeds of moving shelf wave generating systems around our coast which rarely follow very closely on one another.

The introduction of this additional information, made it possible to set up a three parameter conditional threshold analysis in order to define statistically independent events. These conditions could be summarized as follows :

1. Mag \geq 100 mm
2. Dur $>$ 7 hours
3. Interval \geq 12 hours

These thresholds should be set as low as possible so as to obtain the maximum number of statistically independent events. A randomness test was used to check the results of the threshold analysis in order to ensure that no dependency structure existed.

5. In order to generalize the information set obtained above it was desirable to fit known distributions to event magnitude, duration and interval data. Goodness of fit tests were then possible using graphical, Chi-Square and Kolmogorov - Smirnov methods.

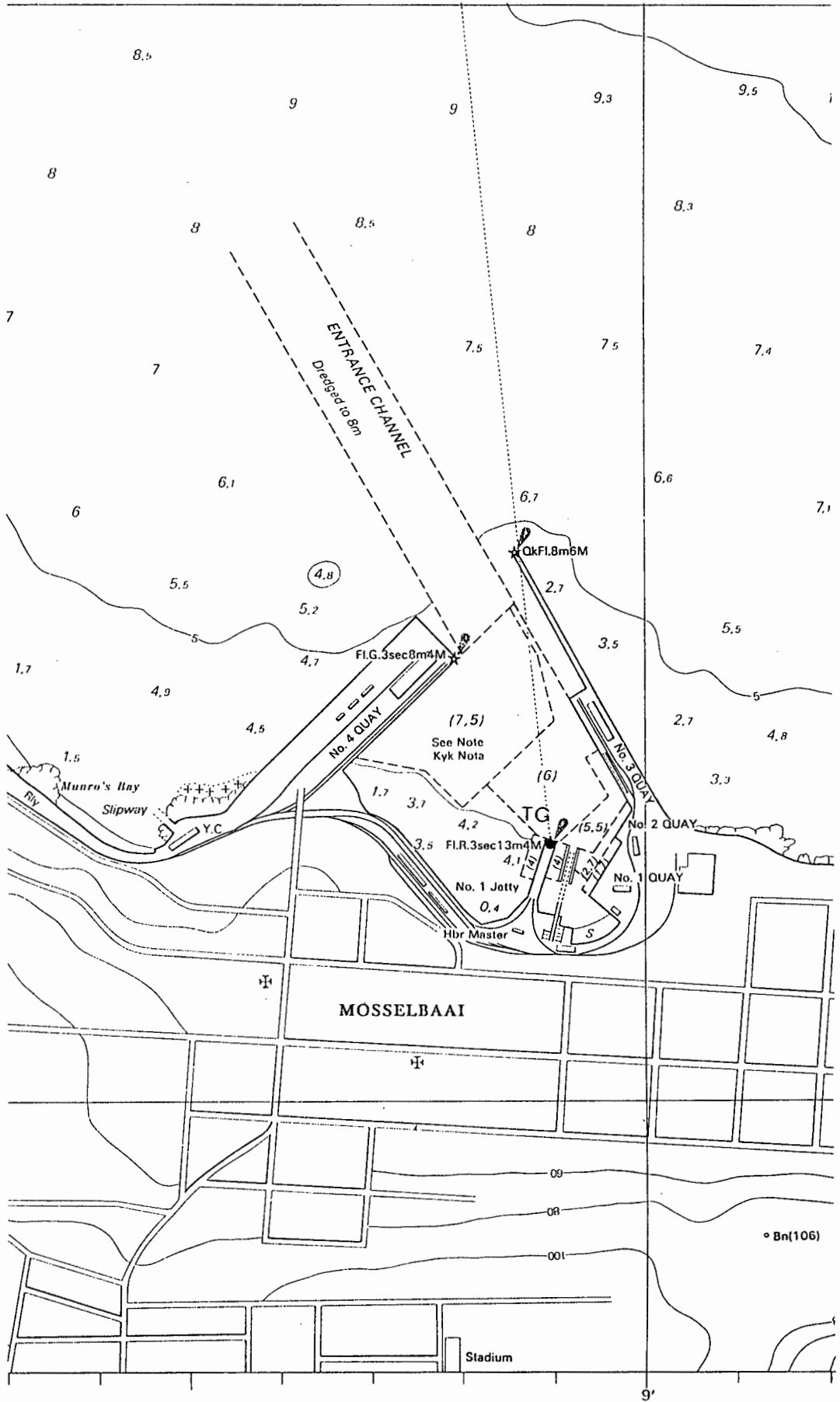
This procedure is illustrated by means of the analysis undertaken for Mossel Bay.

6.2.1. *Data Analysis for Mossel Bay*

The sea level records for Mossel Bay were obtained from the Department of Oceanography, UCT for the period 1980-1988. The tide gauge (TG) for Mossel Bay is situated inside a protected harbour at the end of the Vincent Jetty.(see figure 6.2.1.1.) The harbour entrance is situated beyond the breaker zone. An assessment of the 9 year record indicated that 7920 hours consisted of missing data. Missing data are important in the determination of the intervals between storms.

FIGURE 6.2.1.1.

LOCATION MAP : MOSSEL BAY TIDE GAUGE



The threshold analysis of the Mossel Bay data set indicated that the duration and interval limitations had the more significant effect on the sampling procedure. Thus the minimum event magnitude, in this instance, was measured as 124mm. The threshold analysis was undertaken for three different duration levels, namely 19 hours, 13 hours and 7 hours. The results for Mossel Bay are summarized below:

Threshold magnitude = 100mm

TABLE 6.2.1.1.

THRESHOLD ANALYSIS (1980-1988)

DURATION (hrs)	19		13		7	
	MAG	DUR	MAG	DUR	MAG	DUR
Mean	319	38	301	32.8	270	23
Stdev	103	19	100	18.8	98	18
Min	157	20	149	14	124	8
Max	899	111	899	111	899	111
Skewness	1.81	1.39	1.81	1.59	1.72	1.91

In addition to setting a duration threshold it was necessary to set a recurrence interval threshold in order to ensure that events were truly independent. Randomness tests were undertaken on these threshold level combinations. The tests are based on standard run tests undertaken by the **STATGRAPHICS** version 2.6. software package (1986). The level of significance of the result is given on a scale of 0-1 (0 = deterministic; 1 = random). The results of these tests are given in table 6.2.1.2.

TABLE 6.2.1.2.

RANDOMNESS TESTS

No: of events	THRESHOLD			MAGNITUDE		DURATION	
	Mag	Dur	Int	Run 1	Run 2	Run 1	Run 2
174	100	19	1	0.254	0.920	0.937	0.975
242	100	13	1	0.221	0.270	0.962	0.933
417	100	7	1	0.003	0.710	0.333	0.0104
319	100	7	6	0.002	0.156	0.080	0.1978
288	100	7	12	0.021	0.093	0.402	0.758
262	100	7	24	0.215	0.024	0.709	0.598
224	100	7	48	0.460	0.116	0.740	0.189
194	100	13	12	0.1161	0.7531	0.573	0.885

A level of significance larger than 0.01 is considered as acceptable in terms of variable randomness. As the primary aim of the threshold analysis is to obtain the greatest number of independent events for a specific time series it can be seen from the table that thresholds set at magnitude = 100mm, duration = 7 hours and recurrence intervals = 12 hours present the optimum choice. The results correspond to a large extent with our hypothesis regarding characteristics of the natural processes.

The intervals between events play a major role in the characterization of the independent event time series. An analysis of the intervals were carried out by calculating the actual observed intervals from the threshold analysis results described previously. The interval between an event was defined as the time period between the end of one independent event and the start of the next independent event. This can be formulated as follows:

$$\text{Event Interval (N) (hrs)} = [\text{Cumulative time (start) event(N)} - \text{Event duration(N)}] - [\text{Cumulative time (end) event (N-1)}]$$

The analysis of the nine years of data for Mossel Bay (1980–1988) resulted in the following statistics for event intervals (based on (100,7,12) threshold data)

INTERVAL STATISTICS

Mean interval	194.50	hours
Stdev	202.90	hours
Min interval	13.00	hours
Max interval	2017.00	hours
No of events	273	

The best fit for the available data was obtained from an exponential distribution for an expected value = 194.5 hours. The Chi squared value = 0.885 and KS = 0.208 indicate a significant relationship between the observed and fitted distribution. The major short coming in the interval data set is that missing data and the threshold analysis distorts the expected value. Consequently, a more robust approach was needed. The average number of events per year multiplied by the average duration equals the total expected event hours per year. It is thus possible to determine the total duration of the non events or calm periods per year. The average duration of these "calms" is then equal to the total period divided by the number of events per year. The following example illustrates the above procedure:

AVERAGE DURATION OF INTERVALS:

$$\begin{aligned} \text{Total hours of observed data (1980–1988)} &= 78912 \text{ hours} \\ \text{Missing data :} &= 7920 \text{ hours} \\ \text{Rate of event occurrence /hr} &= \frac{\text{No of events}}{(\text{Observed hours})-(\text{Missing hours})} \\ &= 0.00404 \text{ events per hour} \\ &= 35.4 \text{ events per year} \end{aligned}$$

If the average duration is equal to 32.50 hours, (see table 6.2.1.3.) then :

Total event hours per year = 32.50 x 35.4
 = 1150.50 hours
 Total non event hours per year = 8766-1151
 = 7615 hours

Accordingly, the average duration of intervals between events

$$= \frac{\text{Total non events (hr)}}{\text{(No events per year)}}$$

Expected hr/interval = 215.11 hours

This value rather than the previous mean will give a more realistic assessment of the expected duration of intervals. The statistics for the Mossel Bay data are summarized in Table 6.2.1.3.

TABLE 6.2.1.3.

STATISTICAL ANALYSIS : MOSSEL BAY (1980 -1988)

	MAG (100mm)	DUR (7 hours)	INT (12 hours)
Mean	285.30	32.50	194.50
Std	99.90	26.40	202.90
Min	124.00	8.00	13.00
Max	696.00	147.00	2017.00
No: of events	287	287	273

In order to extrapolate the measured data set beyond the actual observed values (in this instance, 9 years of record) it was necessary to fit distributions to the available observations. Whilst the magnitude values related strongly to a log normal and extreme I distribution the duration values showed no significant relationship to any of the distributions tested. (ie: gamma, normal, log normal, exponential or weibull). The results of the chi-square test for the log normal and extreme I distributions for event magnitude are given in table 6.2.1.4.

TABLE 6.2.1.4.

DISTRIBUTION FITTING

MAGNITUDE No of events	LOG NORMAL Chi-square	EXTREME I
174	0.18	fail
242	0.107	fail
417	0.022	0.7736
288	0.03	0.320

The above results confirm that an extreme I distribution would be appropriate for magnitude calculations if a single outlier of 899mm was ignored. Closer inspection of this event reveals a short term (8 hours) peak superimposed on a long term (82 hours) increase in sea level.

The initial modelling exercises did not take this outlier into consideration and made use of an extreme I distribution for event magnitude. The results obtained were extrapolated to determine 1/50 and 1/100 year shelf wave magnitudes based on the extreme I distribution on the assumption that on average 35.4 independent events of magnitude larger than 100mm, duration greater than 7 hour and recurrence interval exceeding 12 hours, occur each year. According to Tawn and Vassie (1989) the return period may be calculated as follows:

$$\begin{aligned}
 T &= 1 / (1-F^N(x)) \\
 &\approx 1/N (1-F(x)) \qquad \qquad \qquad -6.1.
 \end{aligned}$$

- N = number of independent events per year
- F(x) = cumulative probability of occurrence
- T = annual return period

Thus for : N = 35.4 events per year
 and T = 100 years

$$F(x) = 0.999718$$

Therefore the predicted value of x according to an extreme I distribution is

$$x_{100} = 863\text{mm}$$

Similarly, the 1/50 year event $x_{50} = 810\text{mm}$

It is interesting to note the impact of the threshold analysis parameter on the 1/100 and 1/50 events. These can be summarized as follows:

TABLE 6.2.1.5.

EXTREME VALUE PREDICTIONS

(Extreme I distribution)

	Threshold Parameters (Mag, Dur, Int)	No: of events	1/50	1/100
1.	(100,13,1)	242	825	879
2.	(100,13,12)	197	833	888
3.	(100,7,1)	417	824	877
4.	(100,7,12)	287	810	863

The differences between these results can be explained in terms of the effect of the inclusion of dependent events, generally smaller than the expected values, which reduces the 1/100 year prediction. This can be seen for 1 & 2 compared with 3. The values in 4 are generally lower due to the removal of the outlier.

It is common practice in most engineering applications to apply confidence limits to highlight the uncertainty relating to the available data set. The level of confidence associated with a particular set of data is directly related to the number of statistically independent identically distributed events (N) measured. Within this framework, it can be seen that the most appropriate set of events is derived from the threshold limits (100,7,12), which represents the largest N fulfilling the stated requirements. Carter and Challenor (1986) present a method for evaluating a one sided upper confidence limit for the extreme type I distribution. Figure 6.2.1.2a illustrates the fitted event magnitude distribution with its associated 95% upper confidence limit. A method will be proposed in chapter 7 for incorporating the one sided 95% confidence limit into the simulation model in order to reflect the extent and quality of the initial data set.

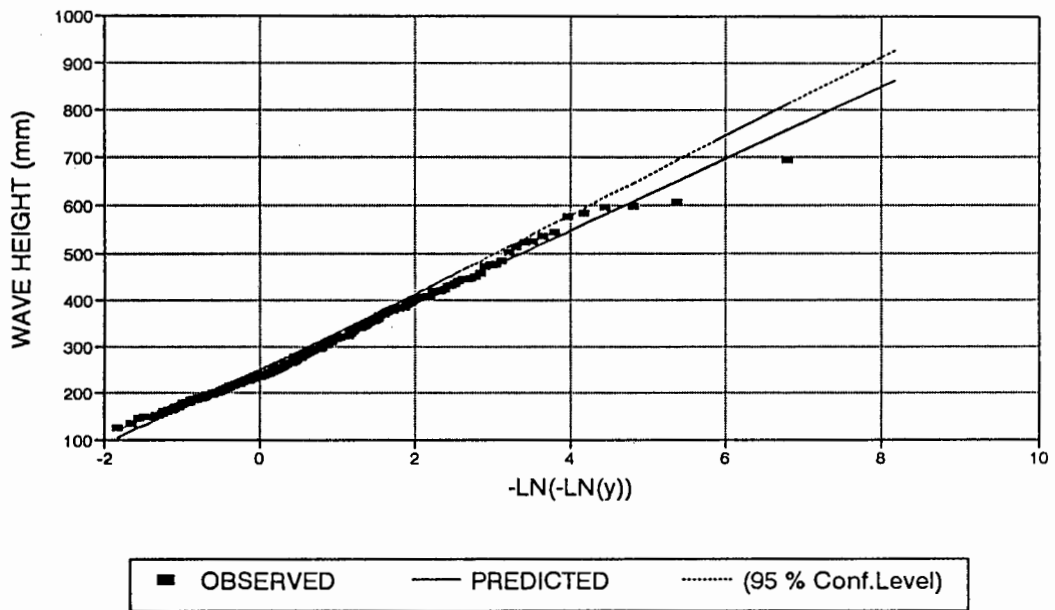
As no obvious distribution could be fitted to the duration of events it was decided to use actual measured durations as representative of the total sum of possible event durations according to the actual distribution obtained. This is discussed in more detail in chapter 7.

The results of the data analysis are summarized in table 6.2.1.6. and figure 6.2.1.2.. The analysis of data from the ports of Richards Bay and Port Nolloth were undertaken in the same manner as described for Mossel Bay. In each case only site specific information will be given. The results are summarized in the same format as table 6.2.1.6.

6.2.2. Data Analysis for Richards Bay

The data coverage for Richards Bay consists of two periods from 1977 – 1985 and 1989 – 1990. These records were supplied by the SAN Hydrographer. The measurements were obtained from a Kent analogue tide gauge (TG) recorder situated in protected waters inside the harbour entrance (see figure 6.2.2.1.). A total of nine years of hourly data were obtained from the Hydrographer. An amount of 20 132 hours comprised of missing data. A further 804 hours of data were rejected on the basis of a visual inspection of the records. The results of the data analysis procedure are summarized in table 6.2.2.1. and figure 6.2.2.2. The threshold analysis was undertaken for events exceeding a wave height of 100mm, duration of 6 hours and intervals between events of 12 hours.

FIGURE 6.2.1.2(a)
SHELF WAVE MAGNITUDE



MOSSEL BAY
 SHELF WAVE LEVELS
 SUMMARY DATA SHEET

TABLE 6.2.1.6

DATA SET	SOURCE	TYPE	DURATION
	UCT	KENT	1980-88
DATA QUALITY			
TOTAL HOURS MEASURED		78912	
MISSING DATA HOURS		7920	
THRESHOLD ANALYSIS			
	MAGNITUDE	DURATION	INTERVAL
	(mm)	(hrs)	(hrs)
	100	7	13
No. OF EVENTS	287		
RATE OF EVENT			
OCCURENCE (events/yr)	35.4		
SUMMARY STATISTICS			
	MAGNITUDE	DURATION	INTERVAL
	(mm)	(hrs)	(hrs)
AVERAGE	285.30	32.50	194.50
STANDARD DEVIATION	99.90	26.40	202.90
MINIMUM	124	8	13
MAXIMUM	696	147	2017
INTERVAL ANALYSIS			
TOTAL EVENT HOURS PER YEAR		1152.45	hrs/yr
CALMS PER HOURS PER YEAR		7613.55	hrs/yr
EXPECTED HOURS PER INTERVAL		215.08	hrs
EXPECTED CALMS PER HOUR		0.00465	events/hr
RANDOMNESS TEST			
	MAGNITUDE	DURATION	INTERVAL
TREND	0.021	0.40	-
CYCLE	0.093	0.76	-
DISTRIBUTION FITTING			
	EXTREME I	LOG NORMAL	GAMMA
MAGNITUDE	0.32	fail	fail
	NORMAL	EXPONENTIAL	WEIBULL
DURATION	fail	-	-
EXTREME EVENT PREDICTIONS			
MAGNITUDE (mm)			
1:5 (year)	-		
1:10 (year)	-		
1:50 (year)	810		
1:100 (year)	863		

FIGURE 6.2.1.2
MOSSEL BAY SHELF WAVES

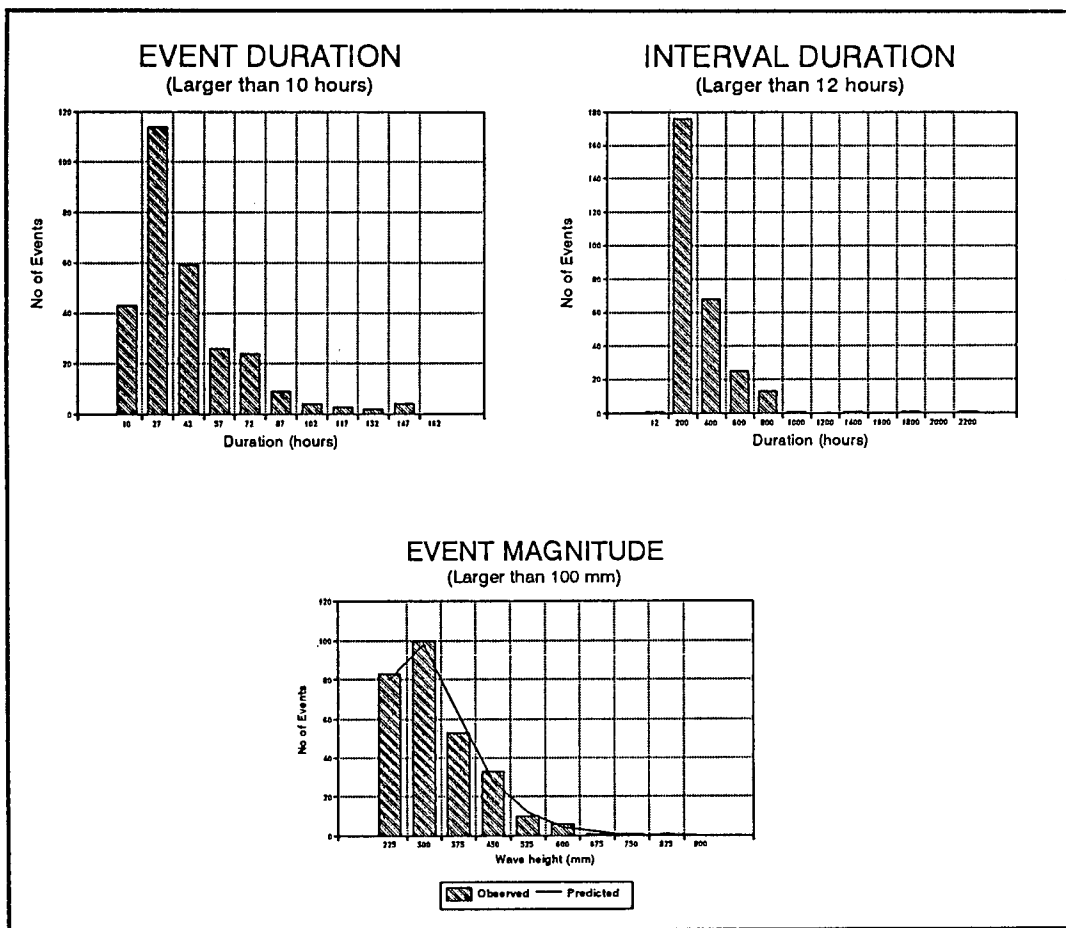
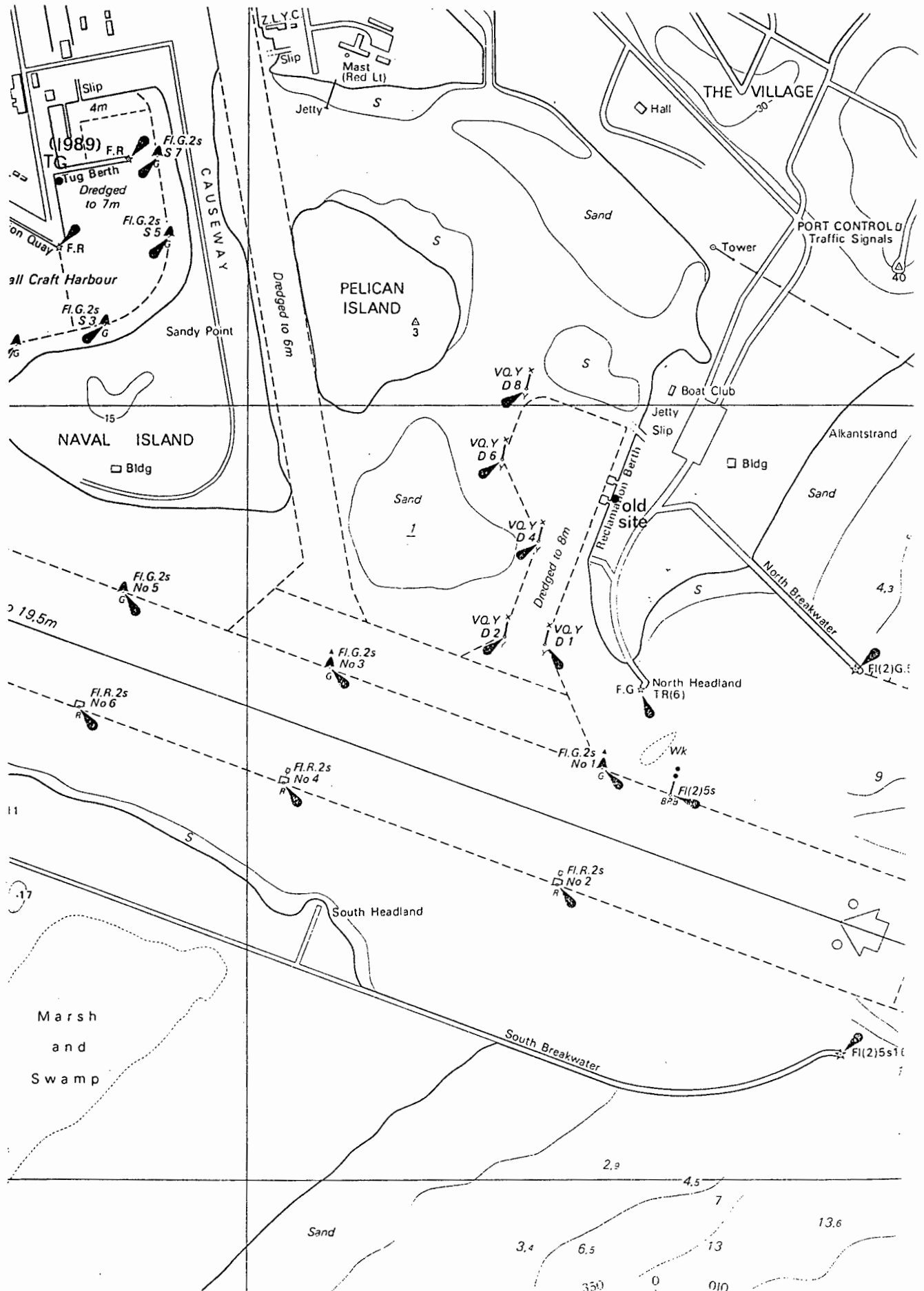


FIGURE 6.2.2.1.

LOCATION MAP : RICHARDS BAY TIDE GAUGE

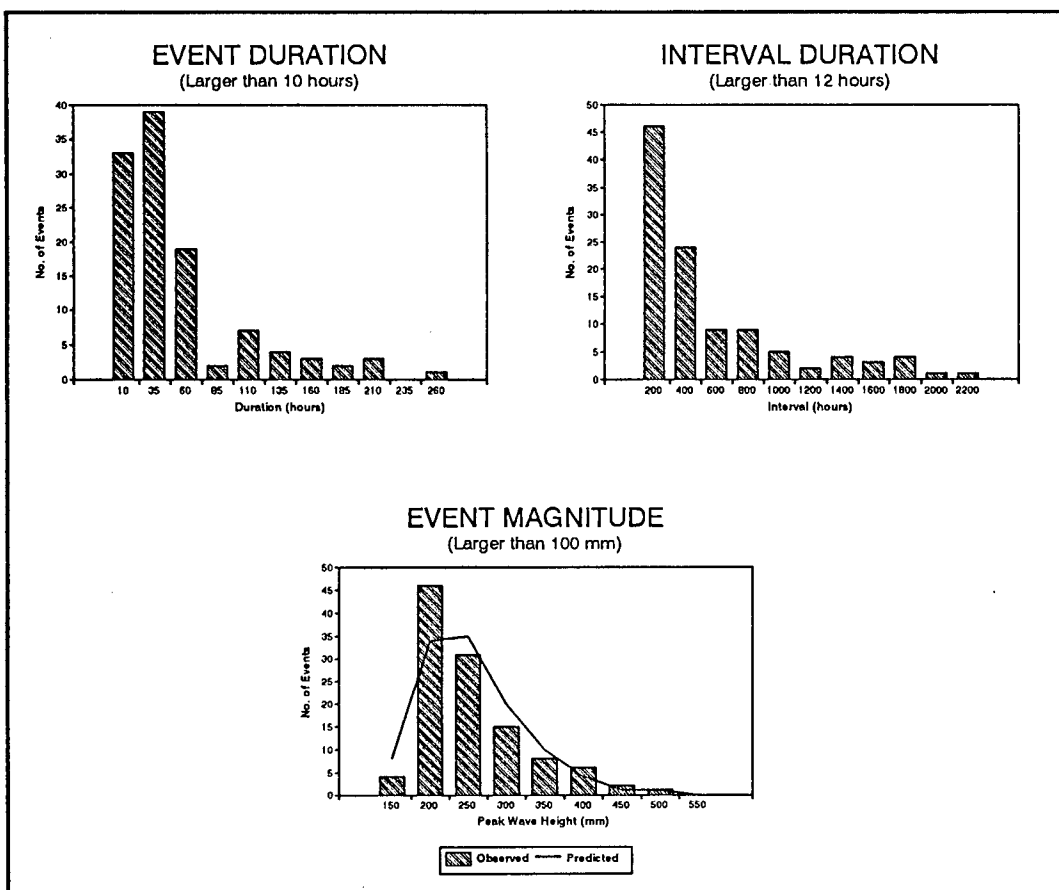


RICHARDS BAY
SHELF WAVE LEVELS
SUMMARY DATA SHEET

TABLE 6.2.2.1

DATA SET	SOURCE	TYPE	DURATION
	SAN	KENT	1977-83
DATA QUALITY			1989-90
TOTAL HOURS MEASURED		78864	
MISSING DATA HOURS		20936	
THRESHOLD ANALYSIS	MAGNITUDE	DURATION	INTERVAL
	(mm)	(hrs)	(hrs)
	100	7	13
No. OF EVENTS	113		
RATE OF EVENT			
OCCURENCE (events/yr)	17.09		
SUMMARY STATISTICS			
	MAGNITUDE	DURATION	INTERVAL
	(mm)	(hrs)	(hrs)
AVERAGE	230.36	42.62	559.44
STANDARD DEVIATION	68.21	50.30	795.13
MINIMUM	135	8	12
MAXIMUM	458	239	1350
INTERVAL ANALYSIS			
TOTAL EVENT HOURS PER YEAR		728.79	hrs/yr
CALMS PER HOURS PER YEAR		8037.2	hrs/yr
EXPECTED HOURS PER INTERVAL		470.28	hrs
EXPECTED CALMS PER HOUR		0.00213	events/hr
RANDOMNESS TEST			
	MAGNITUDE	DURATION	INTERVAL
TREND	0.706	0.85	-
CYCLE	0.243	0.10	-
DISTRIBUTION FITTING			
	EXTREME I	LOG NORMAL	GAMMA
MAGNITUDE	0.05	0.03	fail
	NORMAL	EXPONENTIAL	WEIBULL
DURATION	fail	fail	fail
EXTREME EVENT PREDICTIONS			
MAGNITUDE (mm)			
1:5 (year)			
1:10 (year)	473		
1:50 (year)	559		
1:100 (year)	596		

FIGURE 6.2.2.2
RICHARDS BAY SHELF WAVES



Shelf waves at Richards Bay are characterized by an expected magnitude of 230 mm and expected durations of 45 hours (moderate long duration events). The expected number of annual occurrences is 17 (more than one per month) with a total of 728 event hours per year. The 1/50 and 1/100 shelf wave magnitude is expected to be 559 mm and 596 mm respectively.

6.2.3. *Data analysis for Port Nolloth*

Port Nolloth sea level data from 1958–1990 were obtained from the SAN Hydrographer. A Kent analogue tide gauge (TG) recorder was used to collect the information. The tide gauge is situated within the harbour limits (see figure 6.2.3.1.) Of a total of 33 years of hourly sea level record 19316 hours were classified as missing data. A further 2185 hours for 1958–59 were rejected during the residual analysis due to an apparent phase shift in the data for this period.

Based on the results obtained for Mossel Bay the threshold analysis was undertaken for events exceeding wave heights of 100mm, duration of 6 hours and interval between events of 12 hours. The results of the data analysis for Port Nolloth are summarized in table 6.2.3.1. and figure 6.2.3.2.

The shelf wave events recorded at Port Nolloth are characterized by an expected wave magnitude of 243 mm and expected duration of 27 hours (moderate short duration events). The expected number of annual occurrences is 15 (similar to Richards Bay) with only 407 event hours per year. This may reflect the observation that shelf waves are generated in this region as opposed to Richards Bay where smaller, longer period events are measured. The 1/50 and 1/100 shelf wave magnitude is expected to be 634 mm and 678 mm respectively.

6.2.4. *Minimum Shelf Wave Levels for Mossel Bay*

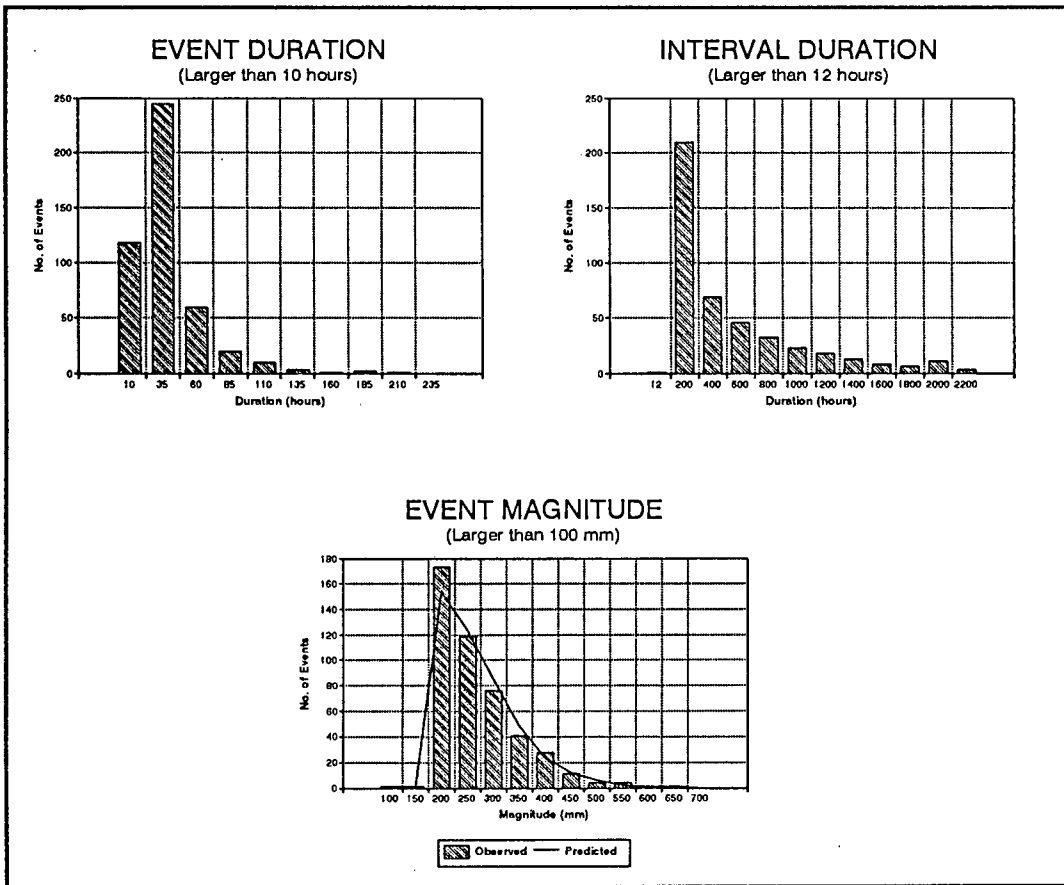
In order to assess the nature of minimum sea level stands or maximum drawdown levels, Mossel Bay data were reanalyzed for events exceeding –100 mm, 7 hours duration and recurrence intervals of 12 hours. Apart from obvious changes required in the threshold analysis, the procedure remained exactly the same as before. The same

PORT NOLLOTH
 SHELF WAVE LEVELS
 SUMMARY DATA SHEET

TABLE 6.2.3.1

DATA SET	SOURCE	TYPE	DURATION
	SAN	KENT	1958-1990
DATA QUALITY			
TOTAL HOURS MEASURED		289422	
MISSING DATA HOURS		21501	
THRESHOLD ANALYSIS			
	MAGNITUDE	DURATION	INTERVAL
	(mm)	(hrs)	(hrs)
	100	7	12
No. OF EVENTS	459		
RATE OF EVENT OCCURENCE (events/yr)			
	15.05		
SUMMARY STATISTICS			
	MAGNITUDE	DURATION	INTERVAL
	(mm)	(hrs)	(hrs)
AVERAGE	242.80	27.04	553.67
STANDARD DEVIATION	82.94	24.86	789.16
MINIMUM	131	8	12
MAXIMUM	707	193	5364
INTERVAL ANALYSIS			
TOTAL EVENT HOURS PER YEAR		407	hrs/yr
CALMS PER HOURS PER YEAR		8359	hrs/yr
EXPECTED HOURS PER INTERVAL		555.41	hrs
EXPECTED CALMS PER HOUR		0.0018	events/hr
RANDOMNESS TEST			
	MAGNITUDE	DURATION	INTERVAL
TREND	0.287	0.24	-
CYCLE	0.616	0.01	-
DISTRIBUTION FITTING			
	EXTREME I	LOG NORMAL	GAMMA
MAGNITUDE	0.05	fail	fail
	NORMAL	EXPONENTIAL	WEIBULL
DURATION	fail	0.14	fail
EXTREME EVENT PREDICTIONS			
MAGNITUDE (mm)			
1:5 (year)	-		
1:10 (year)	530		
1:50 (year)	634		
1:100 (year)	678		

FIGURE 6.2.3.2
PORT NOLLOTH SHELF WAVES



data set as used in section 6.2.1. was analyzed for minimum levels. The results of the data analysis are summarized in table 6.2.4.1. and figure 6.2.4.1. If the results of the maximum drawdown and maximum elevation are compared, it would appear that sea level fluctuations are more responsive to negative waves than positive waves. The physical reason for this is not clear at this stage. The lack of symmetry, however, can no doubt be related to the forced nature of shelf wave events. The expected event magnitude is nearly –300 mm with an expected duration of 42 hours (large long duration events). It is expected that 38 events will occur per annum (at least 3 per month) resulting in 1589 event hours per year. There will be significant periods of time when predicted sea level will not be the same as observed sea level in Mossel Bay. The estimated 1/50 and 1/100 event would appear to be –1088 mm and –1153 mm respectively. This is a significant deviation when viewed in terms of harbour operations.

6.2.5. *Minimum Shelf Wave Levels for Richards Bay*

Minimum levels for Richards Bay were analyzed in terms of thresholds set at –100 mm, 7 hours and 12 hours for event magnitude, duration and recurrence interval respectively. The results are summarized in table 6.2.5.1. and figure 6.2.5.1. The results reflect, to a large extent the characteristics of the positive wave events. The expected event magnitude is –237 mm with an expected duration of 54 hours (in the order of 4 days). It is expected that 19 events will occur per annum (nearly 2 per month) with an expected value of 1089 event hours per year. The estimated 1/50 and 1/100 year events are –613 mm and –654 mm respectively.

6.2.6. *Minimum Shelf Wave Levels for Port Nolloth*

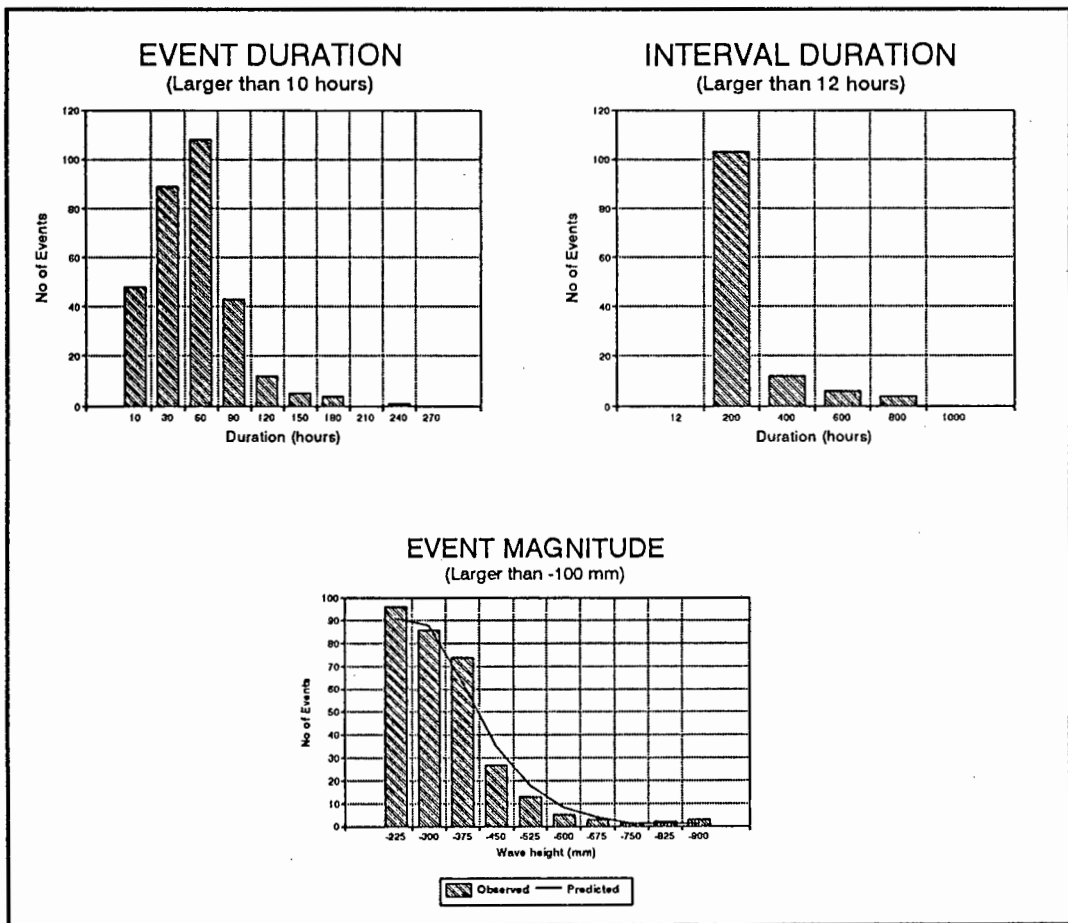
Threshold levels were set at –100 mm, 7 hours and 12 hours for the analysis of minimum shelf wave levels at Port Nolloth. The results are summarized in table 6.2.6.1. and figure 6.1.6.1. Minimum shelf wave levels at Port Nolloth can be expected to have an average magnitude of –242 mm and average duration of 38 hours (3 days). An expected number of 22 events should occur per year (nearly 2 per month) with an expected 827 event hours. The negative and positive shelf wave characteristics reflect the same characteristics or trends if compared with Mossel Bay and Richards Bay. The 1/50 and 1/100 event magnitudes are –609 mm and –649 mm respectively marginally less than Richards Bay.

MOSSEL BAY
 MINIMUM SHELF WAVE LEVELS
 SUMMARY DATA SHEET

TABLE 6.2.4.1

DATA SET	SOURCE	TYPE	DURATION
	SAN	KENT	1980-88
DATA QUALITY			
TOTAL HOURS MEASURED		78912	
MISSING DATA HOURS		7920	
THRESHOLD ANALYSIS			
	MAGNITUDE	DURATION	INTERVAL
	(mm)	(hrs)	(hrs)
	-100	7	13
No. OF EVENTS	310		
RATE OF EVENT			
OCCURENCE (events/yr)	38.278		
SUMMARY STATISTICS			
	MAGNITUDE	DURATION	INTERVAL
	(mm)	(hrs)	(hrs)
AVERAGE	-298.15	41.52	169
STANDARD DEVIATION	120.25	32.99	177
MINIMUM	-141	8	13
MAXIMUM	-856	244	1664
INTERVAL ANALYSIS			
TOTAL EVENT HOURS PER YEAR		1589.31	hrs/yr
CALMS PER HOURS PER YEAR		7176.68	hrs/yr
EXPECTED HOURS PER INTERVAL		187.48	hrs
EXPECTED CALMS PER HOUR		0.00533	events/hr
RANDOMNESS TEST			
	MAGNITUDE	DURATION	INTERVAL
TREND	0.078	0.28	0.58
CYCLE	0.540	0.20	0.45
DISTRIBUTION FITTING			
	EXTREME I	LOG NORMAL	GAMMA
MAGNITUDE	0.05	fail	fail
	NORMAL	EXPONENTIAL	WEIBULL
DURATION	fail	fail	0.401
EXTREME EVENT PREDICTIONS			
MAGNITUDE (mm)			
1:5 (year)	-850		
1:10 (year)	-937		
1:50 (year)	-1088		
1:100 (year)	-1153		

FIGURE 6.2.4.1
MOSEL BAY MINIMUM SHELF WAVES

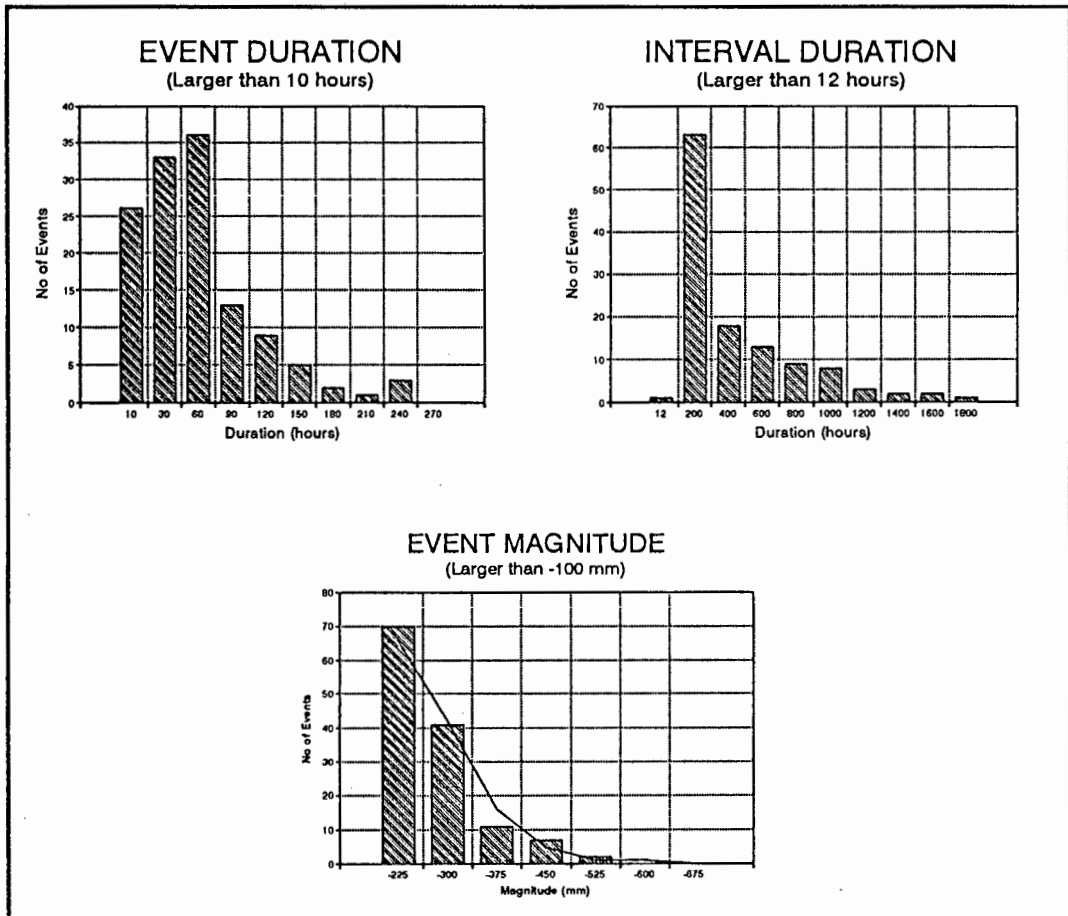


RICHARDS BAY
 MINIMUM SHELF WAVE LEVELS
 SUMMARY DATA SHEET

TABLE 6.2.5.1

DATA SET	SOURCE	TYPE	DURATION
	SAN	KENT	1977-83
DATA QUALITY			1989-90
TOTAL HOURS MEASURED		78864	
MISSING DATA HOURS		20936	
THRESHOLD ANALYSIS	MAGNITUDE	DURATION	INTERVAL
	(mm)	(hrs)	(hrs)
	-100	7	13
No. OF EVENTS	131		
RATE OF EVENT			
OCCURENCE (events/yr)	19.82		
SUMMARY STATISTICS			
	MAGNITUDE	DURATION	INTERVAL
	(mm)	(hrs)	(hrs)
AVERAGE	-237.30	54.93	415
STANDARD DEVIATION	76.16	61.46	524
MINIMUM	-129	8	13
MAXIMUM	-506	357	2119
INTERVAL ANALYSIS			
TOTAL EVENT HOURS PER YEAR		1089	hrs/yr
CALMS PER HOURS PER YEAR		7677	hrs/yr
EXPECTED HOURS PER INTERVAL		387.34	hrs
EXPECTED CALMS PER HOUR		0.00258	events/hr
RANDOMNESS TEST			
	MAGNITUDE	DURATION	INTERVAL
TREND	0.005	0.21	0.0085
CYCLE	0.505	0.24	0.916
DISTRIBUTION FITTING			
	EXTREME I	LOG NORMAL	GAMMA
MAGNITUDE	0.50	0.3716	0.1377
	NORMAL	EXPONENTIAL	WEIBULL
DURATION	fail	0.487	0.313
EXTREME EVENT PREDICTIONS			
MAGNITUDE (mm)			
1:5 (year)	-476		
1:10 (year)	-517		
1:50 (year)	-613		
1:100 (year)	-654		

FIGURE 6.2.5.1
RICHARDS BAY MINIMUM SHELF WAVES



PORT NOLLOTH

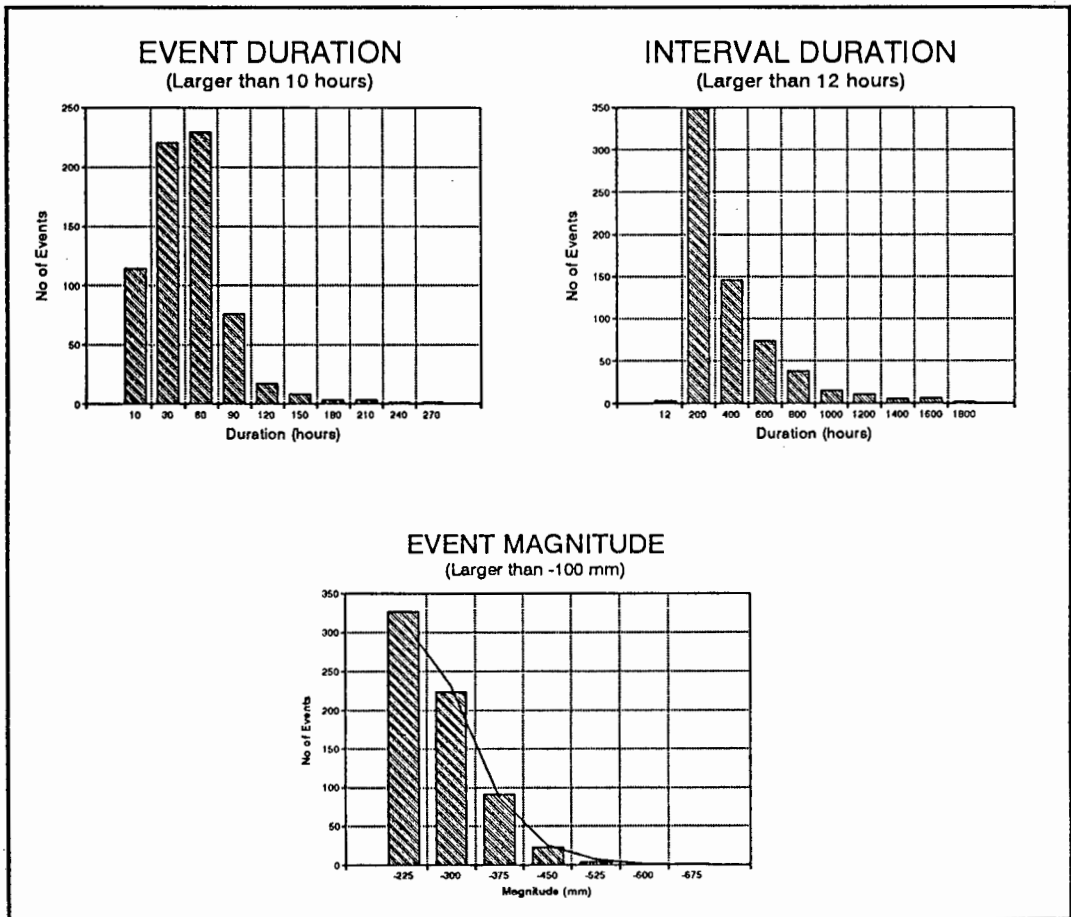
MINIMUM SHELF WAVE LEVELS

SUMMARY DATA SHEET

TABLE 6.2.6.1

DATA SET	SOURCE	TYPE	DURATION
	SAN	KENT	1958-1990
DATA QUALITY			
TOTAL HOURS MEASURED		289422	
MISSING DATA HOURS		21501	
THRESHOLD ANALYSIS			
	MAGNITUDE	DURATION	INTERVAL
	(mm)	(hrs)	(hrs)
	-100	7	12
No. OF EVENTS	672		
RATE OF EVENT			
OCCURENCE (events/yr)	21.99		
SUMMARY STATISTICS			
	MAGNITUDE	DURATION	INTERVAL
	(mm)	(hrs)	(hrs)
AVERAGE	-241.61	37.64	357
STANDARD DEVIATION	73.28	30.92	520
MINIMUM	-131	8	12
MAXIMUM	-653	265	3678
INTERVAL ANALYSIS			
TOTAL EVENT HOURS PER YEAR		827	hrs/yr
CALMS PER HOURS PER YEAR		7939	hrs/yr
EXPECTED HOURS PER INTERVAL		361.07	hrs
EXPECTED CALMS PER HOUR		0.00277	events/hr
RANDOMNESS TEST			
	MAGNITUDE	DURATION	INTERVAL
TREND	0.375	0.727	0.02
CYCLE	0.634	0.768	0.818
DISTRIBUTION FITTING			
	EXTREME I	LOG NORMAL	GAMMA
MAGNITUDE	0.35	fail	fail
	NORMAL	EXPONENTIAL	WEIBULL
DURATION	fail	fail	0.02
EXTREME EVENT PREDICTIONS			
MAGNITUDE (mm)			
1:5 (year)	-477		
1:10 (year)	-517		
1:50 (year)	-609		
1:100 (year)	-649		

FIGURE 6.2.6.1
PORT NOLLOTH MINIMUM SHELF WAVES



6.2.7. *Discussion of the results of Shelf Wave Analysis*

Shelf wave events were evaluated from the hourly tidal residual data using a three parameter threshold analysis procedure. Maximum (positive) and minimum (negative) levels were assessed independently. Mossel Bay appeared to be most responsive to shelf waves. Nearly twice as many events (larger than 100 mm) were recorded at this port. Although it is not the intention of this study to assess the physics of the waves it may be postulated that the differences between Mossel Bay and Richards Bay and Port Nolloth are related to the physiography of the continental shelf at each location. The shelf configuration coupled with the wave propagation mechanism may well result in Mossel Bay being more prone to large shelf wave events. A further point of interest is the comparison of the nature of events at Richards Bay and Port Nolloth. Richards Bay is characterized by moderate magnitude long duration events whilst Port Nolloth reflects moderate magnitude short duration events. In both cases, the number of events per year are similar. This may be explained in terms of the physical characteristics of shelf waves. Shelf waves are generated on the west coast and propagate eastwards. In a generating area it can be expected that wave period will be relatively short, as a function of the driving force. As the wave propagates eastwards it can be expected that the higher frequency components will be damped out more rapidly than lower frequency components. Hence in areas some distance from the source one may expect smaller magnitude events of larger duration. The asymmetry in the wave characteristics, noted between positive and negative waves, can be attributed to the forced nature of the shelf wave. This asymmetry would appear to be most prominent at Mossel Bay which could again relate the shelf configuration at this point.

6.3. *WAVE CLIMATE*

Wave setup is the increase in sea level associated with the transfer of momentum in the breaker zone which takes place when gravity waves impact on an open coastline. Of primary interest therefore is the analysis of deep sea wave records with a view to transferring this data to a particular location. The procedure used to analyze the available wave records can be summarized as follows:

1. All available deep sea waverider records were obtained for a particular regional location.
2. The waverider records had been subjected to basic quality control and screening as described in Roussow (1989).
3. The wave statistics indicated that seasonality played a significant role in the characteristics of wave height variability. Wave data were therefore divided into Summer (November–April) and Winter (May–October) records and analyzed separately.
4. A three parameter threshold analysis was undertaken on the available data time series with thresholds of 3 m wave height, 6 hours duration and 6 hours recurrence interval. These levels were selected in order to ensure that independent events could be identified.
5. The basic statistics relating to the occurrence rate, event magnitude and event duration were then obtained.
6. Probability distributions were fitted to event magnitude, event duration and recurrence intervals. KS and Chi-square tests were used to test their validity.

The approach followed is illustrated by way of the procedure used for Mossel Bay.

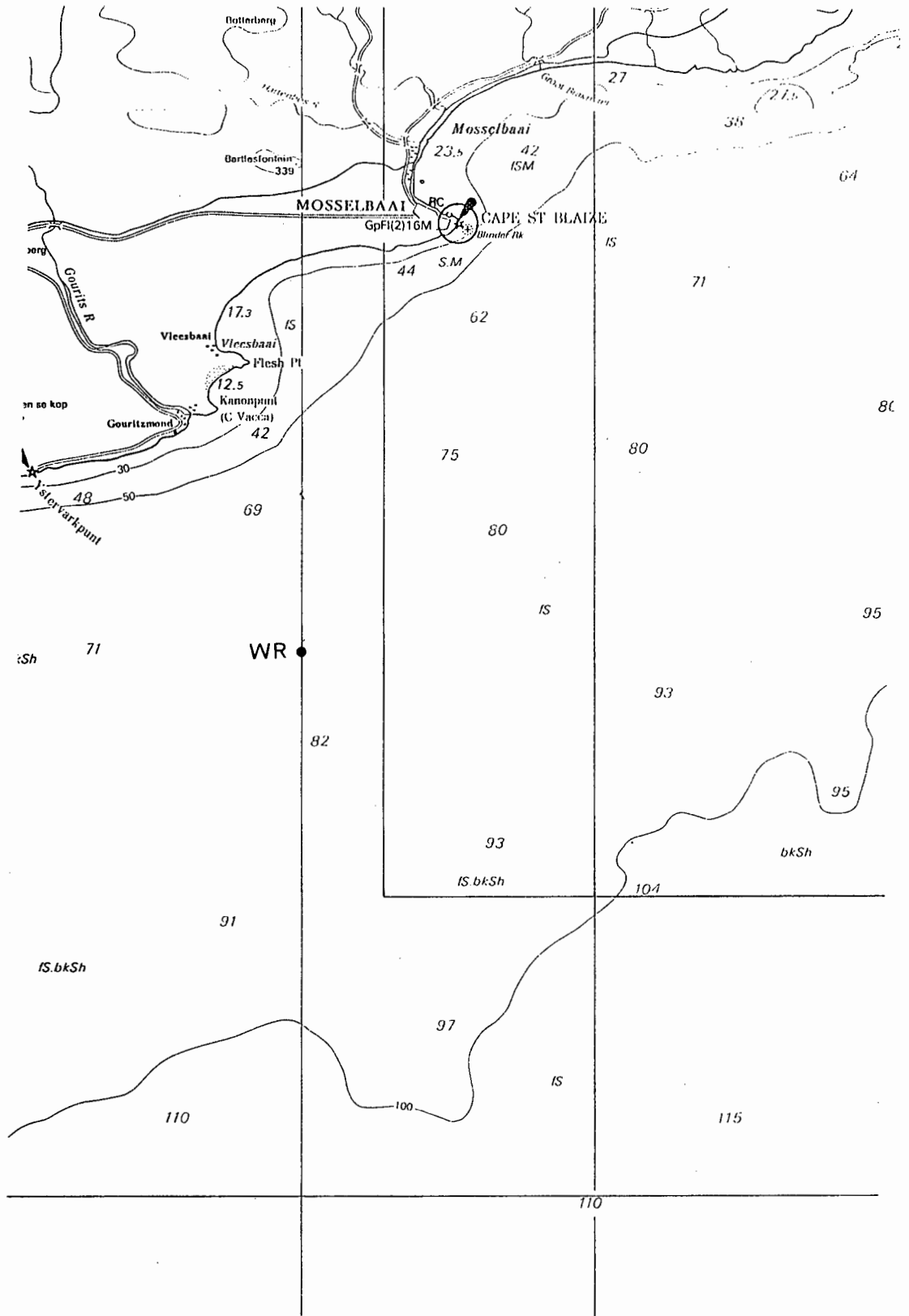
6.3.1. Data Analysis for Mossel Bay

The available waverider (WR) records for Mossel Bay are composed of observations from the Soekor exploration drilling rigs operating approximately 80–100 nautical miles south of Mossel Bay in water depth ranging between 60–110 meters (see figure 6.3.1.1.). The information has been obtained from the SEDCO K, the ACTINA and Gouritzmond waveriders. These data will henceforth be referred to as Mossel Bay waverider data.(MB waves).

The data set comprised a six hourly time series of significant wave heights for various overlapping periods between 1978 and 1991. The first step entailed combining all the data into one continuous time series, plotting the results and evaluating the extent of the missing data. The missing data set is important when evaluating the rate of occurrence of independent events. Of the 114855 hours of record evaluated, 40701 hours of missing data were found.

FIGURE 6.3.1.1.

LOCATION OF MOSSEL BAY WAVERIDER



The compiled time series was then subjected to a threshold analysis to determine the basic statistical characteristics of independent storm events larger than 3 metres. A level of 3 metres was selected as a magnitude threshold in order to clearly identify independent events. It is quite likely that at other locations lower or higher thresholds may be selected. Ultimately the choice of a particular threshold is determined by the specific characteristics of the site under consideration. If the site is located in a shelter embayment the effect of wave refraction and diffraction will tend to make events smaller than 3 metres insignificant. On the other hand on an exposed open coastline much smaller events will play a role. A rule of thumb in this regard may be to consider a sea level response larger than 100 mm at the site under consideration as being large enough to be taken into account.

The threshold analysis undertaken resulted in a new data set consisting of event durations (hours), the peak significant wave height associated with a particular event, the cumulative time of the peak event and the year in which the event occurred. By plotting the independent events, time of occurrence during the year versus the magnitude of the events, it could be clearly seen that a seasonal bias existed in the data set. It is notable that the larger storms (both in magnitude and duration) occur from May through to October. This can be seen in the accompanying figure 6.3.1.2. It was decided therefore to split the data into two seasonal entities, notably winter and summer. Winter was demarcated as being between May and October (2886-7301) and Summer between November and April (7302-8784 and 0-2885 hours).

Two different threshold levels were selected namely (mag, duration, intervals) (3,6,6) and (3,6,12). In order to check their appropriateness in terms of dependency, a randomness test as described in section 6.2 was used for both the individual summer and winter data sets and for the combined data set. The following results were obtained:

FIGURE 6.3.1.2

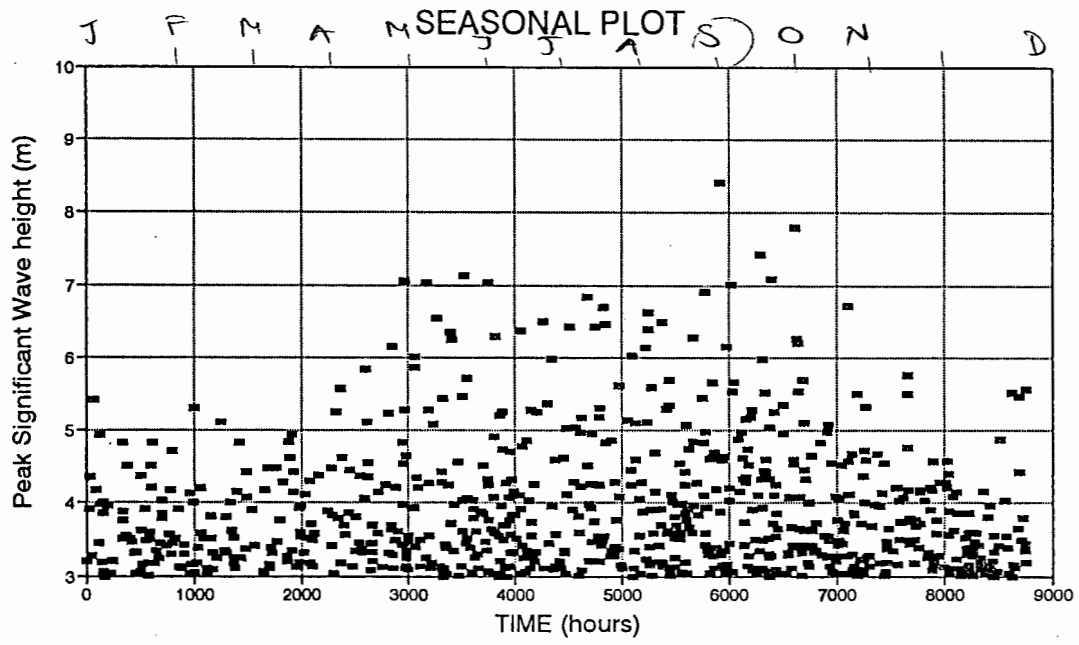


TABLE 6.3.1.1.

RANDOMNESS TEST

Threshold	No: of events	Magnitude		Duration	
		Trend	Cycles	Trend	Cycles
3,6,12	616	0.659	0.737	0.048	0.004
3,6,6 (summer)	300	0.369	0.040	1.000	0.208
3,6,6 (winter)	466	0.3318	0.429	0.8898	0.003

The randomness test indicated that the lowest threshold combination of (3,6,6) can be used for summer and winter data. The winter data on both threshold limits indicated some lack of randomness with regard to duration values. The procedure is sensitive to long term cycles in which the number of turning points (cycles) is less than those in the random sequence. There appeared to be no apparent improvement by using the higher threshold limit (3,6,12) thus the larger data sets (3,6,6) were used.

An interval analysis or assessment of the statistical characteristics of the calms between storm events was carried out on the new data set. Due to the poor data coverage over some time periods it was extremely difficult to assess the statistical properties of the intervals between events. An evaluation of the intervals, excluding those where missing data were included, indicated that these data fitted an exponential distribution. The actual data could not be used as the degree of data loss was too high. In order to assess the expected rate of occurrence of events and hence estimate the expected duration of calms the following approach was used.

The number of events per season and the number of missing hours for that year were counted and subtracted from the total possible hours. The number of events or occurrences per hour could then be calculated for each year. The average number of occurrences per hour was then assessed over the 14 year period. An example of this method for winter is given in table 6.3.1.2. The results of the interval analysis can be summarized as follows :

TABLE 6.3.1.2
SEASONAL DATA ANALYSIS (WINTER)

YEAR	EVENTS	MISSING DATA	RECORDED DATA	RATE PER HOUR
78	34	1450	2360	0.0144
79	13	3280	1135	0.0115
80	45	1428	2987	0.0151
81	38	1632	2783	0.0137
82	23	2618	1797	0.0128
83	21	1835	2580	0.0081
84	23	2472	1943	0.0118
85	32	1795	2620	0.0122
86	35	1446	2969	0.0118
87	45	202	4213	0.0107
88	44	724	3691	0.0119
89	48	561	3854	0.0125
90	52	222	4193	0.0124
91	13	0	1329	0.0098
AVERAGE	33.29	1404.64	2746.71	0.0120
AVERAGE RATE PER YEAR				105.57
AVERAGE RATE PER SEASON				52.78

	SUMMER	WINTER
Average events per hour	0.00828	0.0121
Number of events per year	72.60	105.60

The number of calms per year will be equivalent to the total number of events. Therefore if the average duration of the events is known then the expected interval may be calculated according to the following procedure:

$$\begin{aligned}
 \text{Average event duration} &= \text{ADUR (see table 6.3.1.4.)} \\
 \text{Average number of events/year} &= 72.6 \text{ (Summer)} \\
 \text{Total event hours/year} &= \text{ADUR} \times 72.6 \text{ hours} \\
 \\
 \text{Therefore total calm hours/year} &= \text{Total hours} - \text{Total event hours} \\
 \text{but : Number of calms} &= \text{Number of events}
 \end{aligned}$$

Therefore:

$$\begin{aligned}
 \text{Expected duration of calms} &= \frac{\text{Total calm hours/years}}{\text{No of calms per year}} \\
 \text{(hr/calm)} &
 \end{aligned}$$

Therefore:

$$\begin{aligned}
 \text{Expected rate of occurrence/hr} &= \frac{1}{\text{Expected duration of calms}}
 \end{aligned}$$

By using the above mentioned method the following results were obtained:

	SUMMER	WINTER
Expected duration (hr/calm)	99.92	53.96
Rate of occurrence (calm/hr)	0.01001	0.0185

These results may be compared with the alternative approach of analyzing the actual data set, including missing values for intervals. Table 6.3.1.3. illustrates the latter approach.

TABLE 6.3.1.3.

INTERVAL ANALYSIS

	TOTAL	SUMMER	WINTER
Mean	71.20	55.62	95.21
Stdev	82.52	65.11	99.37
Min	6.00	6.00	6.00
Max	594.00	594.00.	504.00
Skewness	2.23	2.95	1.569
No: of events	766	300	466

Both the summer and winter data fit an exponential distribution with a chi square statistic of 0.65 and 0.31 respectively. The two assessment approaches compare well, however the former is considered as more robust as it includes the influence of missing data. This approach will be used throughout the study.

Event magnitude and duration information obtained from the threshold analysis were analyzed with regard to its basic statistical characteristics. These can be summarized as follows (see table 6.3.1.4.):

TABLE 6.3.1.4.

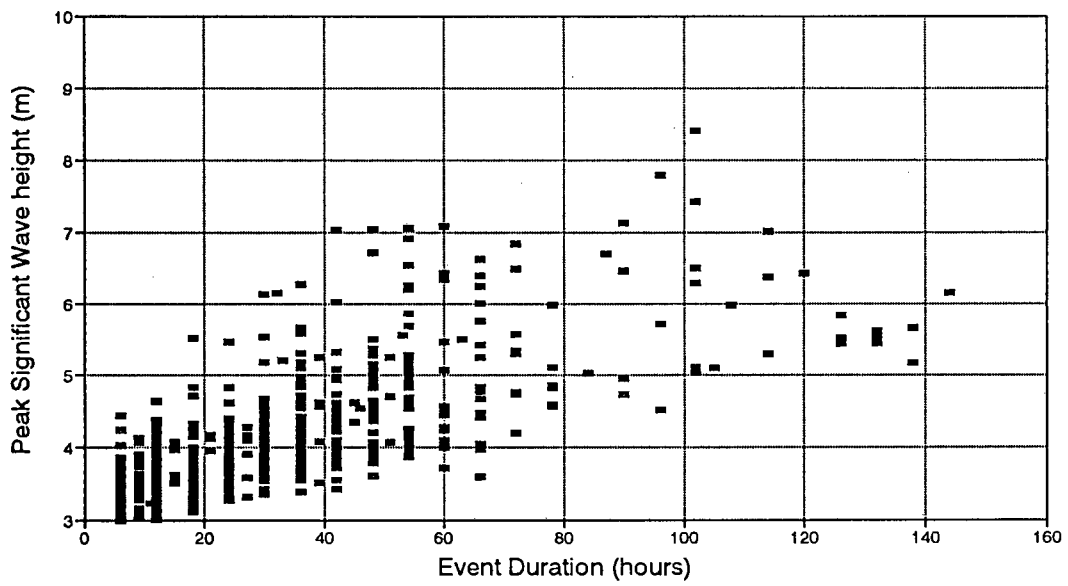
STATISTICS FOR MAGNITUDE AND DURATION

	SUMMER		WINTER	
	MAGNITUDE (m)	DURATION (hr)	MAGNITUDE (m)	DURATION (hr)
Mean	3.71	20.85	4.05	29.05
Stdev	0.619	19.17	1.014	27.45
Min	3.00	6.00	3.00	6.00
Max	6.16	126.00	8.41	144.00
Skewness	1.298	1.974	1.723	1.723
No of events	300	300	466	466

The table above appears to justify early remarks relating to the seasonal nature of the data set. Clearly, larger events of longer duration tend to occur during the winter months whilst the summer months are characterized by smaller less variable occurrences. It may be postulated that the two data sets represent the response of the sea to the different driving processes, notably the predominantly winter Ferrel Westerlies and the summer Atlantic high, respectively. Whilst this hypothesis has not been investigated it remains statistically correct to evaluate these two data sets separately.

An evaluation of the relationship between the magnitude of the peak significant wave height and the event duration indicates that, in general, large events are characterized by extended durations. This can be seen in figure 6.3.1.3. The relationship displays a large degree of variability and therefore should be assessed as a joint probability distribution. The event magnitudes for both winter and summer were fitted to a number of distributions and tested for goodness of fit. The results of these investigations indicated that the best fit in, both instances was obtained using an extreme I distribution.

FIGURE 6.3.1.3
Event Magnitude vs Duration



CHI-SQUARED RESULTS

	SUMMER	WINTER
Extreme I	0.10	0.08
Log normal	fail	fail
Gamma	fail	fail

Similarly for the event durations:

CHI-SQUARED RESULTS

	SUMMER	WINTER
Normal	fail	fail
Exponential	fail	fail
Weibull	fail	fail

In order to evaluate the magnitude of the design event the fitted data for the Extreme I and log normal distributions were extrapolated from the 11 years of observed data to estimate the 1/100 and 1/50 events. The probability of occurrences were calculated according to Tawn and Vassie (1989)

$$T = 1/(1-F^N(x)) \approx 1/N(1-F(x))$$

where

- N = Number of independent events per year
- F(x) = Cumulative probability of occurrence
- T = Annual return period

Therefore $F(x) = (1-1/N.T)$

An assessment of the available data for summer and winter taking into consideration the influence of missing data over the 14 year period resulted in expected occurrence

rates of 73 and 106 events per year. This implies that 36.5 and 53.3 events occur per season for summer and winter respectively. The recurrence probabilities for these values of N may therefore be determined using the above information.

TABLE 6.3.1.5.

EXTREME EVENT MAGNITUDE

	SUMMER	WINTER
Return periods (years)	Hmo (m)	Hmo (m)
1/10	7.00	9.10
1/50	7.40	10.36
1/100	7.73	10.91

If the winter values are considered to dominate the distribution these values may be compared with Roussow (1989) with the difference that more data has been included from 1986–1991. The results of the data analysis can be summarized in a standard format in table 6.3.1.6. and the characteristic distributions given in figure 6.3.1.4. and 6.3.1.5.

6.3.2. Data Analysis for Richards Bay

The waverider (WR) records for Richards Bay were measured in a water depth of 19 metres south east of the harbour mouth (see figure 6.3.2.1.). The data set extends from January 1980 to March 1992. Of the total of 108092 hours measured, 52591 hours of missing data were found.

The available data were subject to a threshold analysis for events exceeding 3 metres in magnitude, 6 hours in duration and at least 6 hours between events. A randomness test was undertaken on the results for these threshold limits for winter and summer data.

MOSEL BAY
DEEP SEA WAVE DATA
SUMMARY DATA SHEET

TABLE 6.3.1.6

DATA SET	SOURCE	TYPE	DURATION
	CSIR	WAVERIDER	1978-91
DATA QUALITY			
TOTAL HOURS MEASURED		114855	
MISSING DATA HOURS		40701	
THRESHOLD ANALYSIS			
	MAGNITUDE	DURATION	INTERVAL
	(m)	(hrs)	(hrs)
	3	6	6
	Summer	Winter	
No OF EVENTS	300	466	
RATE OF EVENT			
OCCURENCE (events/yr)	72.60	105.60	
SUMMARY STATISTICS			
	MAGNITUDE	DURATION	INTERVAL
SUMMER	(m)	(hrs)	(hrs)
AVERAGE	3.71	20.85	55.62
STANDARD DEVIATION	0.619	19.17	65.11
MINIMUM	3	6	6
MAXIMUM	6.16	126	594
WINTER			
AVERAGE	4.02	29.05	95.21
STANDARD DEVIATION	1.01	27.45	99.37
MINIMUM	3	6	6
MAXIMUM	8.41	144	504
INTERVAL ANALYSIS			
	Summer	Winter	
TOTAL EVENT HOURS PER YEAR	1514	3068	hrs/yr
CALMS PER HOURS PER YEAR	7252	5698	hrs/yr
EXPECTED HOURS PER INTERVAL	99.92	53.96	hrs
EXPECTED CALMS PER HOUR	0.01001	0.0185	events/hr
RANDOMNESS TEST			
	Summer	MAGNITUDE	DURATION
TREND		0.37	1.00
CYCLE		0.04	0.21
	Winter		
TREND		0.33	0.89
CYCLE		0.43	0.003
DISTRIBUTION FITTING			
	Summer	Winter	
	EXTREME I	EXTREME I	
MAGNITUDE	0.10	0.075	
	EXPONENTIAL	EXPONENTIAL	
DURATION	fail	fail	
EXTREME EVENT PREDICTIONS			
	Summer	Winter	
MAGNITUDE (m)			
1:5 (year)	-		
1:10 (year)	7.00	9.10	
1:50 (year)	7.40	10.36	
1:100 (year)	7.73	10.91	

FIGURE 6.3.1.4
MOSSEL BAY SUMMER WAVE CLIMATE

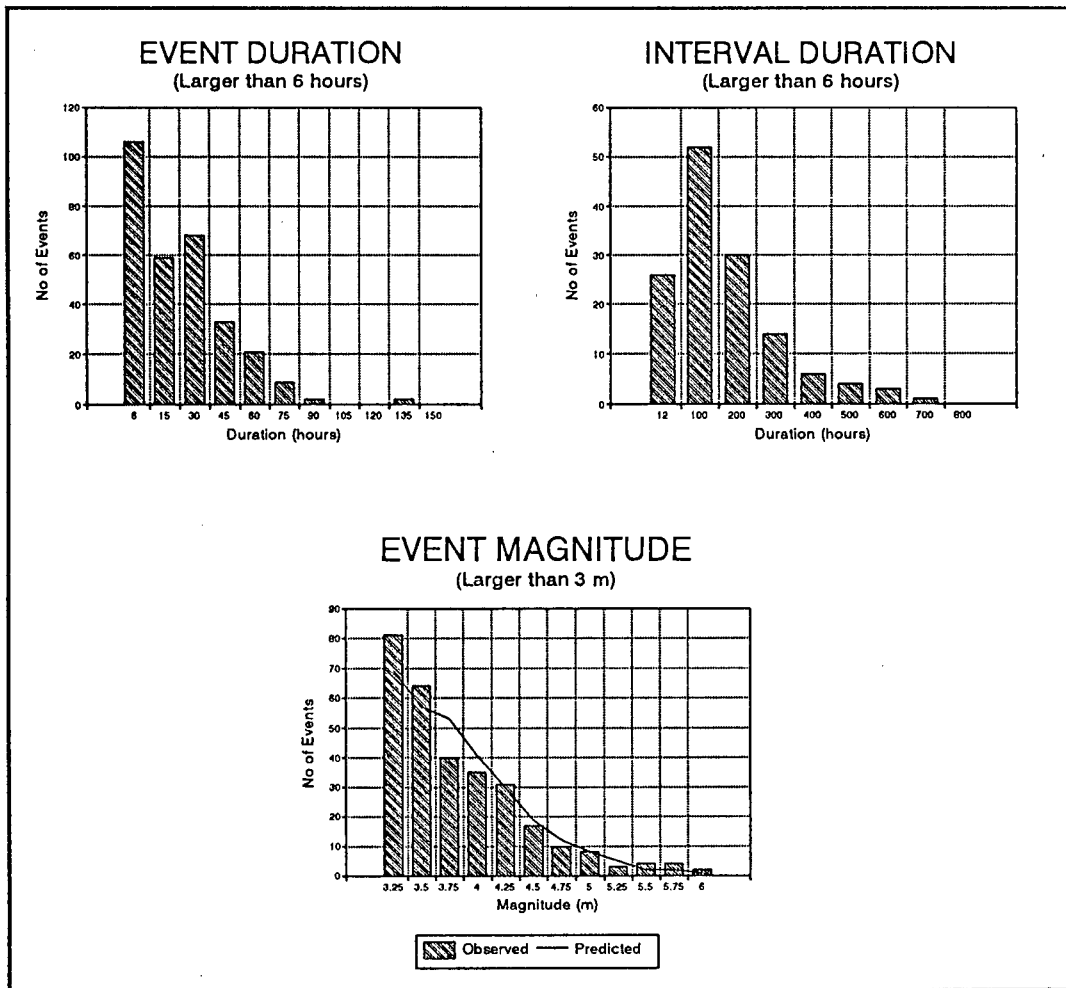


FIGURE 6.3.1.5
MOSEL BAY WINTER WAVE CLIMATE

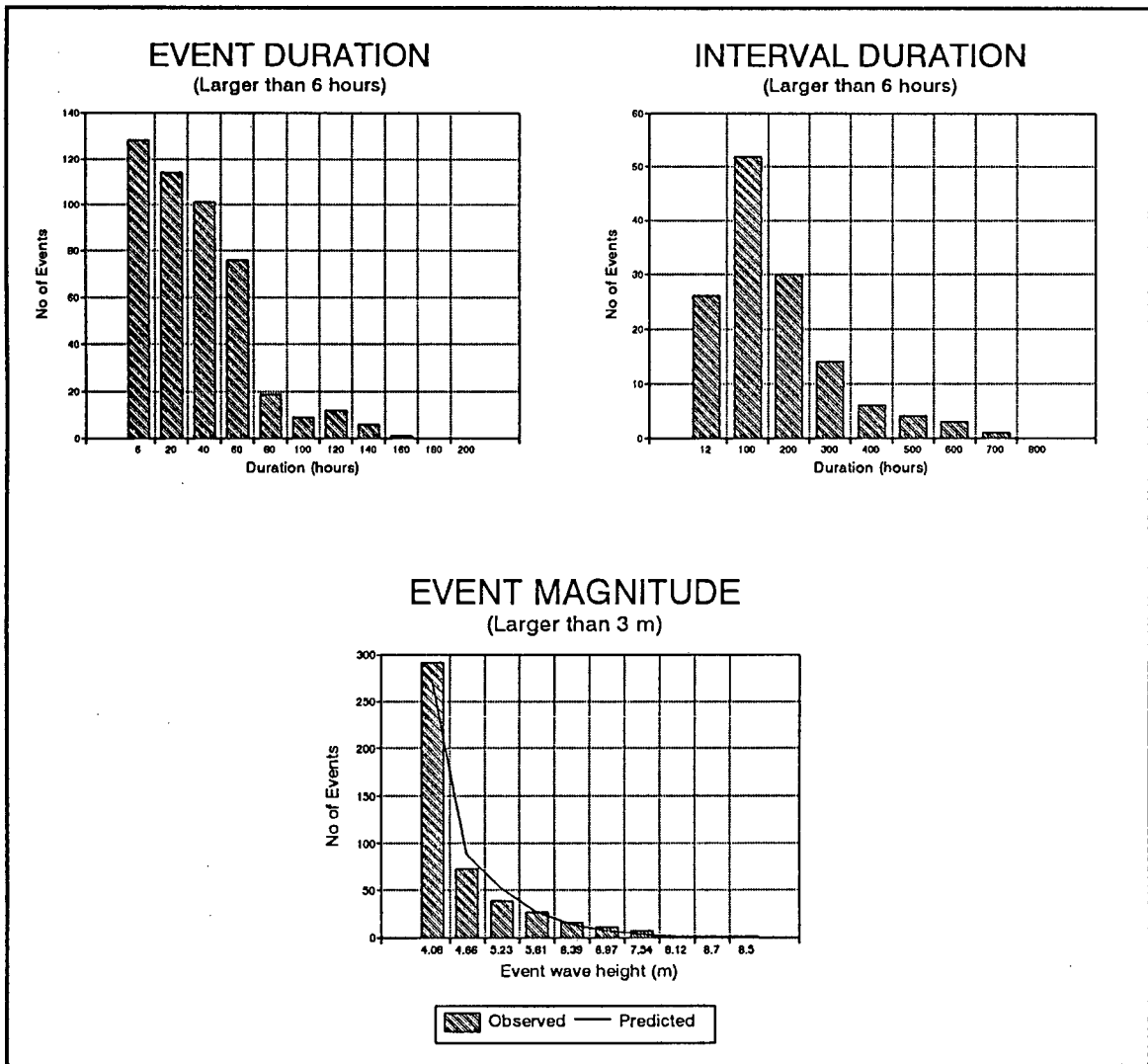
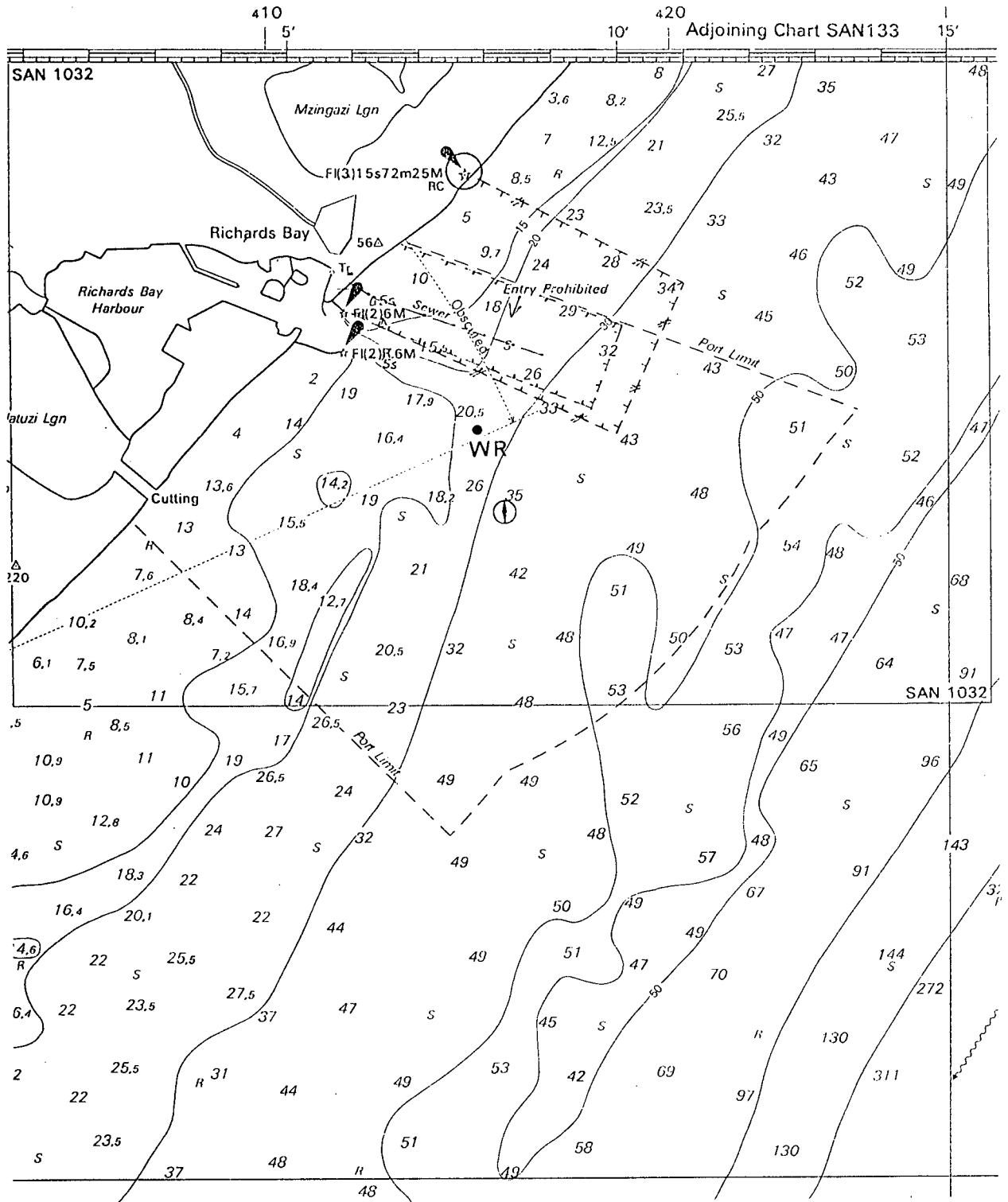


FIGURE 6.3.2.1.

LOCATION OF RICHARDS BAY WAVERIDER



The results of the data analysis are summarized in table 6.3.2.1. and figure 6.3.2.2. and figure 6.3.2.3. Wind wave storms observed at Richards Bay can be expected to have an average peak significant wave height of 3.61 m, 3.62 m (summer, winter) with an expected duration of (14, 17) hours. More than twice the number of events can be expected to occur in winter than in summer (6, 14). Similarly the exposure to waves above 3 metres indicates significantly more storm hours in winter than during the summer months. The 1/50 and 1/100 design wave estimates based on an extreme type I distribution are (5.03, 6.23) m and (5.28, 6.62) m respectively.

6.3.3. *Data Analysis for Port Nolloth*

The Port Nolloth waverider (WR) is situated in 105 metres of water 5 kilometres west of the port (see figure 6.3.3.1.). The available data set extends from February 1987 to March 1992. Of a total coverage of 45195 hours, only 8072 hours constituted missing data.

A threshold analysis was undertaken on the data based on limits set at a wave height of 3 metres, event duration at 6 hours and event intervals of 6 hours. A seasonality test indicated that larger events occurred during the winter months than in the summer months. The data were split in two data sets representing 1 November – 30 April (summer) and 1 May – 31 October (winter). The results are summarized in table 6.3.3.1. and figures 6.3.3.2 and 6.3.3.3. The expected peak significant wave height observed at Port Nolloth is (3.49 m, 3.74 m) (summer, winter) with expected durations of (16.6, 23.1) hours, for storms larger than 3 m. The rate of occurrence of storms per year is (44.70, 72.75). Almost twice as many events occur in winter compared with summer. This trend is further emphasized by the duration of storms larger than 3 m (741, 1680) hours. An estimation of the 1/50 and 1/100 events, based on an extreme I distribution, results in (5.45, 7.59) m and (5.66, 7.97) m respectively.

6.3.4. *Discussion on Wave Climate*

The analysis of wave climate in southern Africa based on three different regions gives some indication of the underlying characteristics. The largest storms were measured on the south Cape coast. This agrees with the proposition that wind waves are generated by the eastward moving Ferrel westerlies which occur more frequently during winter.

RICHARDS BAY
DEEP SEA WAVE DATA
SUMMARY DATA SHEET

TABLE 6.3.2.1

DATA SET	SOURCE	TYPE	DURATION
	CSIR	WAVERIDER	1980-92
DATA QUALITY			
TOTAL HOURS MEASURED		108092	
MISSING DATA HOURS		52591	
THRESHOLD ANALYSIS			
	MAGNITUDE	DURATION	INTERVAL
	(m)	(hrs)	(hrs)
	3	6	6
	Summer	Winter	
No OF EVENTS	24	47	
RATE OF EVENT			
OCCURENCE (events/yr)	6.14	15.93	
SUMMARY STATISTICS			
	MAGNITUDE	DURATION	INTERVAL
SUMMER	(m)	(hrs)	(hrs)
AVERAGE	3.62	14.17	-
STANDARD DEVIATION	0.461	16.34	-
MINIMUM	3.03	6	-
MAXIMUM	4.53	84	-
WINTER			
AVERAGE	3.61	17.17	-
STANDARD DEVIATION	0.725	19.15	-
MINIMUM	3	6	-
MAXIMUM	6.44	84	-
INTERVAL ANALYSIS			
	Summer	Winter	
TOTAL EVENT HOURS PER YEAR	87	274	hrs/yr
CALMS PER HOURS PER YEAR	8679	8492	hrs/yr
EXPECTED HOURS PER INTERVAL	1414	533	hrs
EXPECTED CALMS PER HOUR	0.0007	0.0019	events/hr
RANDOMNESS TEST			
Summer	MAGNITUDE	DURATION	INTERVAL
TREND	0.99	0.39	-
CYCLE	0.11	0.18	-
Winter			
TREND	0.56	0.58	
CYCLE	0.93	0.93	
DISTRIBUTION FITTING			
	Summer	Winter	
	EXTREME I	EXTREME I	
MAGNITUDE	0.05	0.20	
	EXPONENTIAL	EXPONENTIAL	
DURATION	0.29	0.12	
EXTREME EVENT PREDICTIONS			
MAGNITUDE (m)	Summer	Winter	
1:5 (year)	-		
1:10 (year)	4.46	5.32	
1:50 (year)	5.03	6.23	
1:100 (year)	5.28	6.62	

FIGURE 6.3.2.2
RICHARDS BAY SUMMER WAVE CLIMATE

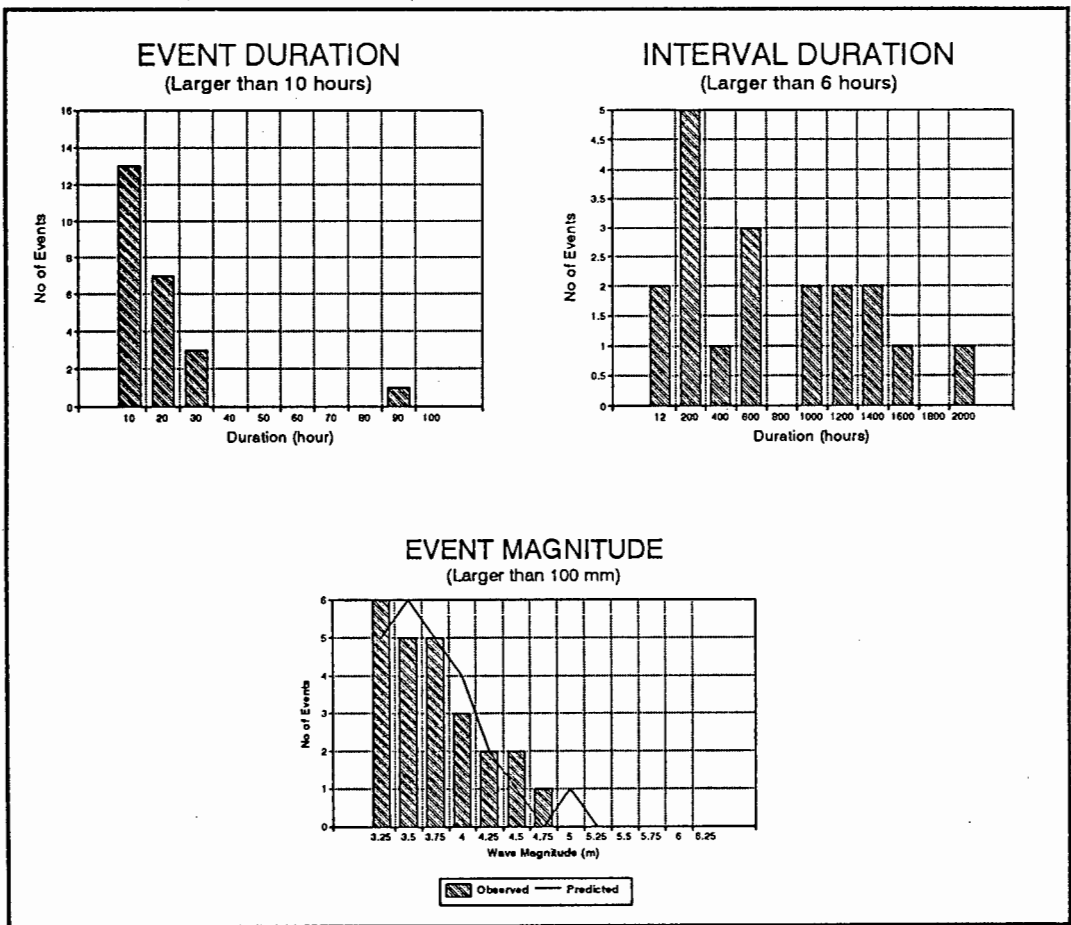


FIGURE 6.3.2.3
RICHARDS BAY WINTER WAVE CLIMATE

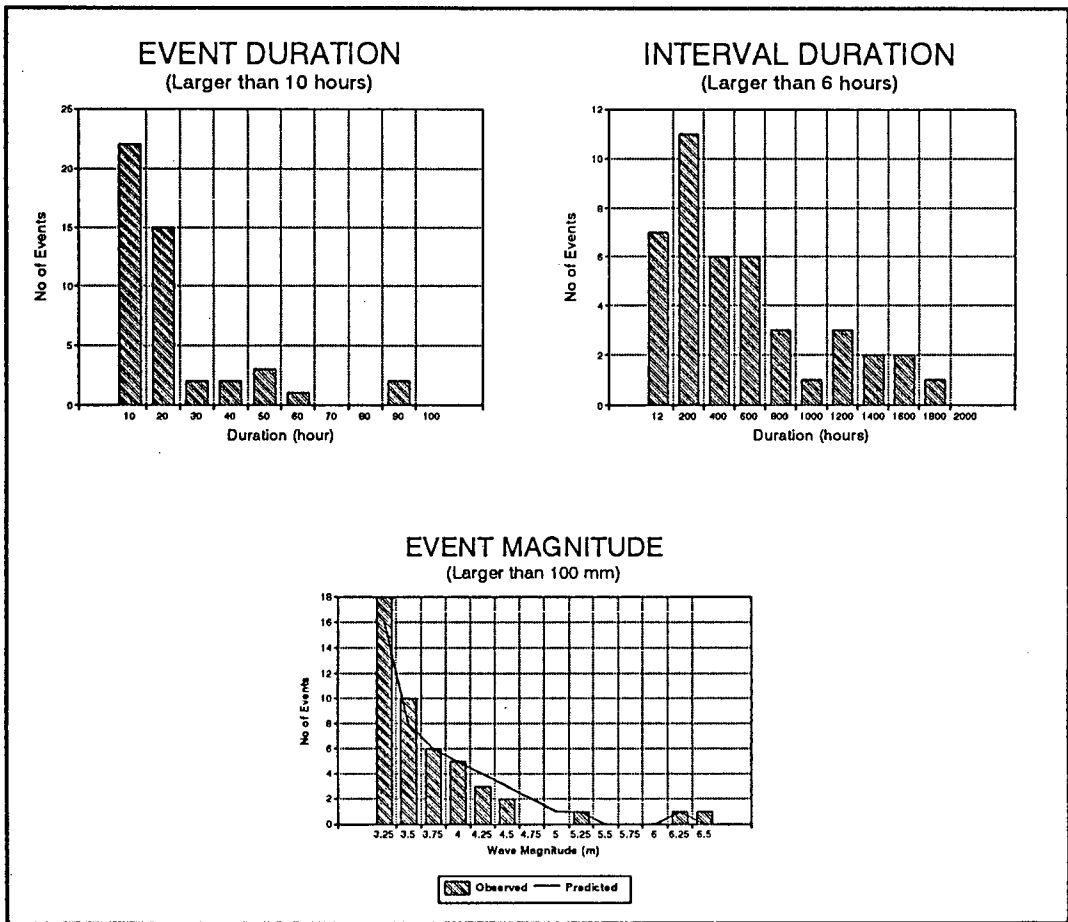
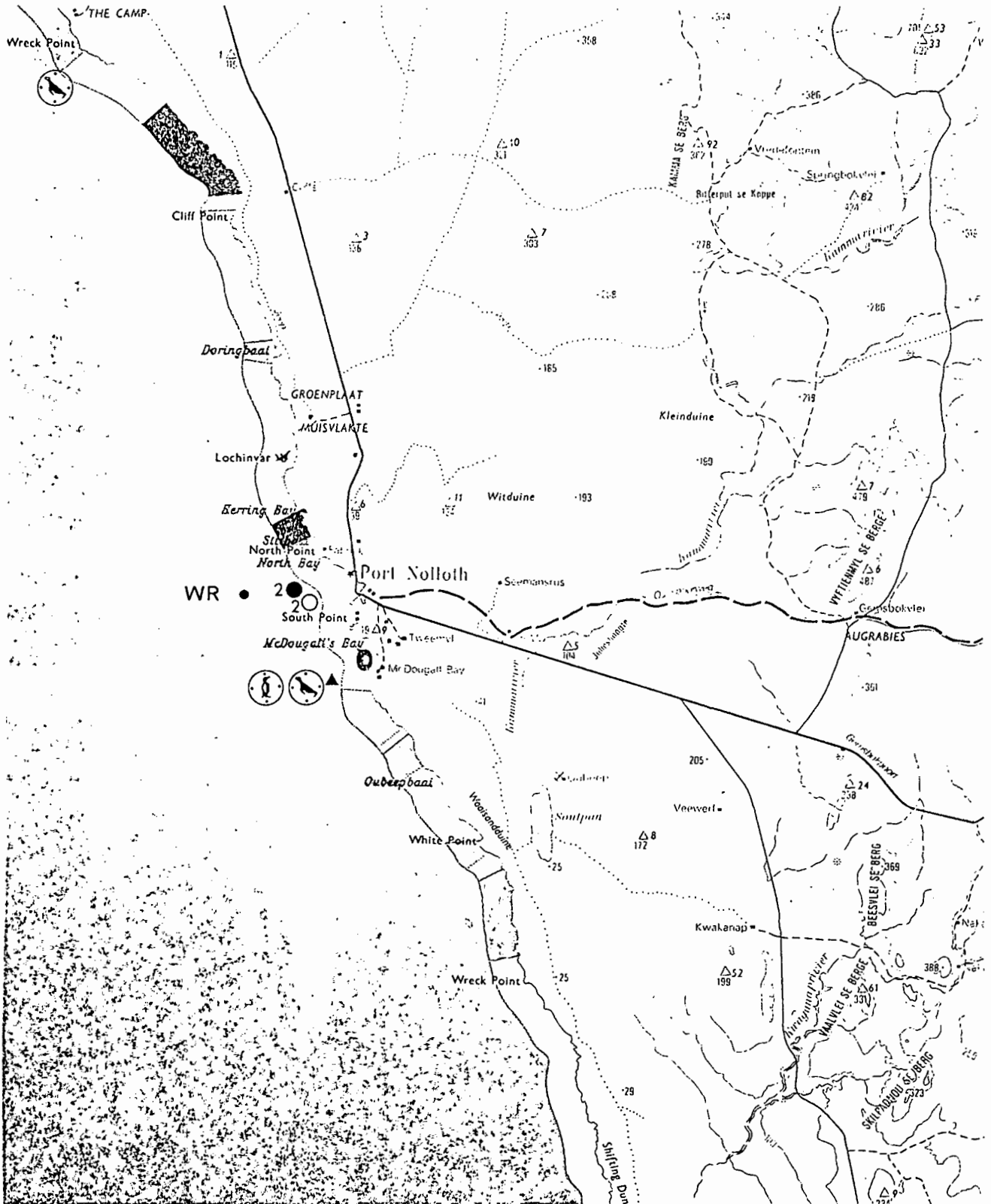


FIGURE 6.3.3.1.

LOCATION OF PORT NOLLOTH WAVERIDER



PORT NOLLOTH

DEEP SEA WAVE DATA
SUMMARY DATA SHEET

TABLE 6.3.3.1

DATA SET	SOURCE	TYPE	DURATION
	CSIR	WAVERIDER	1987-92
DATA QUALITY			
TOTAL HOURS MEASURED		45195	
MISSING DATA HOURS		8072	
THRESHOLD ANALYSIS			
	MAGNITUDE	DURATION	INTERVAL
	(m)	(hrs)	(hrs)
	3	6	6
	Summer	Winter	
No OF EVENTS	76	166	
RATE OF EVENT			
OCCURENCE (events/yr)	44.70	72.75	
SUMMARY STATISTICS			
	MAGNITUDE	DURATION	INTERVAL
SUMMER	(m)	(hrs)	(hrs)
AVERAGE	3.485	16.57	-
STANDARD DEVIATION	0.392	12.30	-
MINIMUM	3	6	-
MAXIMUM	4.97	66	-
WINTER			
AVERAGE	3.735	23.1	-
STANDARD DEVIATION	0.714	17.85	-
MINIMUM	3	6	-
MAXIMUM	5.95	84	-
INTERVAL ANALYSIS			
	Summer	Winter	
TOTAL EVENT HOURS PER YEAR	741	1299	hrs/yr
CALMS PER HOURS PER YEAR	8025	7467	hrs/yr
EXPECTED HOURS PER INTERVAL	180	97	hrs
EXPECTED CALMS PER HOUR	0.0056	0.0103	events/hr
RANDOMNESS TEST			
Summer	MAGNITUDE	DURATION	INTERVAL
TREND	0.56	0.84	-
CYCLE	0.96	0.22	-
Winter			
TREND	0.59	0.24	
CYCLE	0.98	0.16	
DISTRIBUTION FITTING			
	Summer	Winter	
	EXTREME I	EXTREME I	
MAGNITUDE	0.578	0.05	
	EXPONENTIAL	EXPONENTIAL	
DURATION	0.32	0.107	
EXTREME EVENT PREDICTIONS			
MAGNITUDE (m)	Summer	Winter	
1:5 (year)	-		
1:10 (year)	4.95	6.69	
1:50 (year)	5.45	7.59	
1:100 (year)	5.66	7.97	

FIGURE 6.3.3.2
PORT NOLLOTH SUMMER WAVE CLIMATE

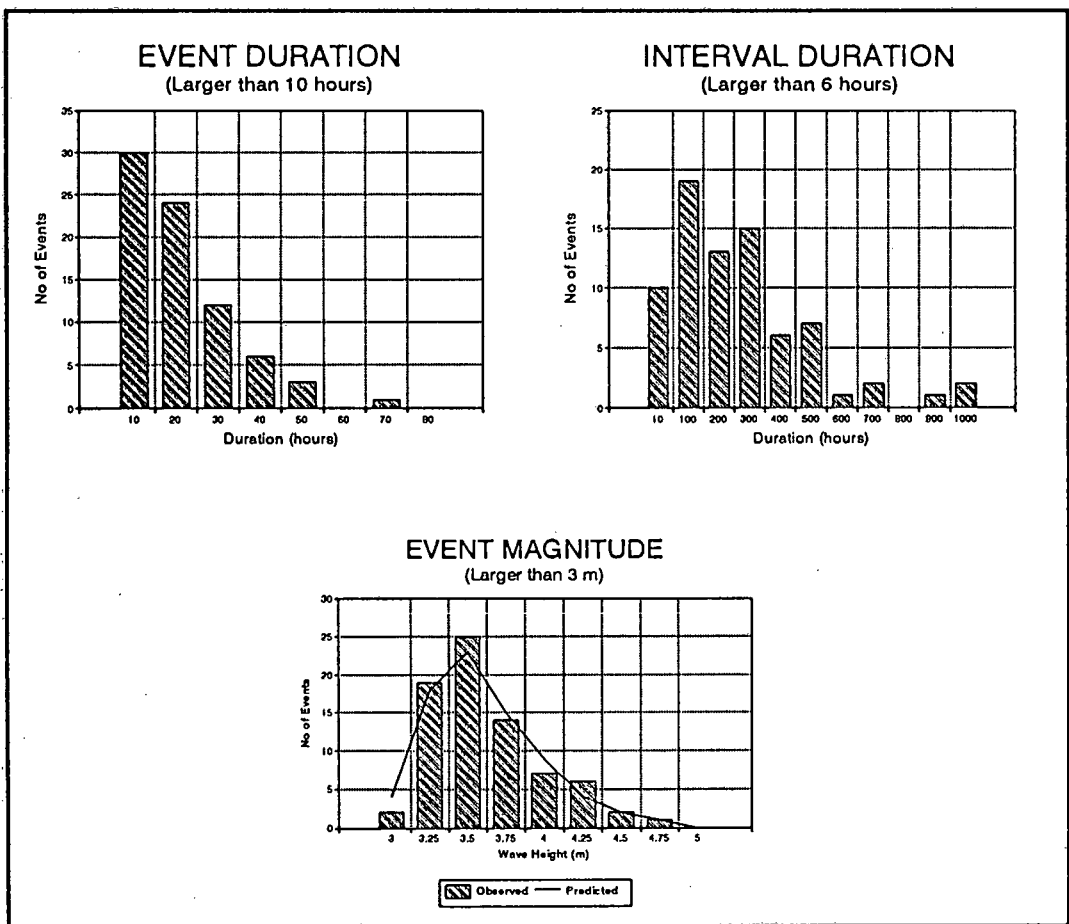
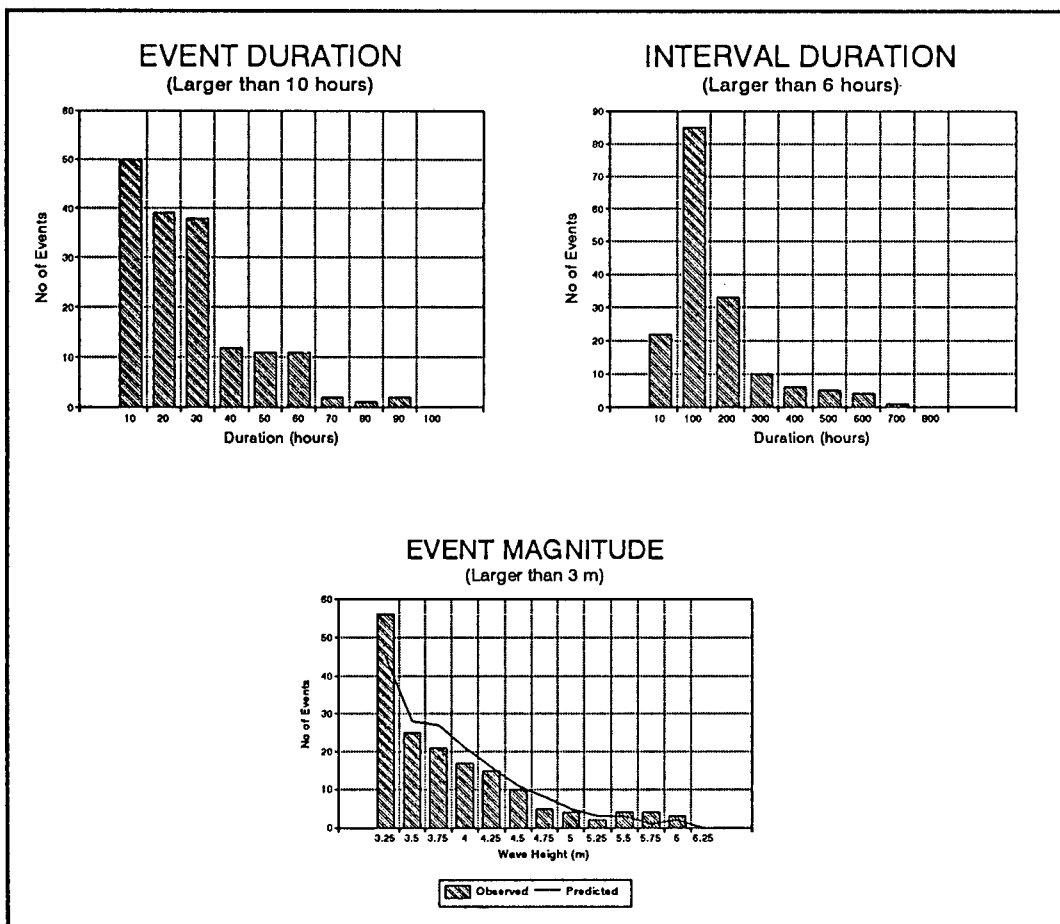


FIGURE 6.3.3.3
PORT NOLLOTH WINTER WAVE CLIMATE



It can be seen that the rate of occurrence, magnitude and duration of storm decreases as one progresses further to the north east and north west. There is a significant difference between the storm hours above the 3 m threshold at the different locations on the coast. Storm hours at Mossel Bay are approximately twice as long as at Port Nolloth and an order of magnitude larger than at Richards Bay. There is therefore a significant difference in the wave climate along these coastlines which is not reflected in the assessment of wave height alone. The magnitude of the storm hours will determine to a large extent the probability of events occurring simultaneously. It is these combined events which ultimately determine the extreme sea level for the southern African coastline.

Rossouw (1989) undertook an extensive assessment of wave climate in Southern Africa. It is not the purpose of this section to repeat any of that work here. Suffice to point out that the same general observations were made regarding a regional wave height distribution. A major difference perhaps relates to the analysis approach used. The major focus of this study has been the assessment of general storm characteristics in terms of peak event magnitude, duration and rate of occurrence.

6.4. *EDGE WAVES*

Limited records of long period edge waves are available in southern Africa. The available measurements of these phenomena are in analogue format, having been obtained from the existing tide gauges around the southern African coast. Shillington (1985) presents the most comprehensive listing of these events and it is from this source that basic edge wave characteristics, as known today, have been compiled. Due to this distinct lack of data, it was not possible to undertake any rigorous data analysis as in the previous sections. The underlying properties of long period edge waves have therefore been inferred from existing data sets.

The average rate of occurrence per year is thought to be 4-5 events which appear to take place predominantly around equinoxes. Equinoxes refer to the period during the year when the sun crosses the celestial equator (ie: Duration of day = duration of night). Time of the year is around the 22nd of September and the 20th of March.

The average amplitude of edge wave events would appear to be between 200–300mm, with maximum wave height of 1200 mm and 1000 mm having been measured at Port Nolloth and at Mossel Bay respectively. These two ports would appear to be most sensitive to long period edge wave events. Typical wave periods measured vary between 10–60 minutes. The duration of these events would appear on average to vary between 6–12 hours with events as short as 2 hours and as long as 48 hours having been measured. Table 6.4.1. summarizes some of the published data on edge waves in southern Africa. At this point in time most of the data resides in analogue form on marigrams held by the SAN Hydrographer. Analysis of these data would not seem to be justified within the context of this study. The approach taken in chapter 7 will be to use assumed probability distributions to represent the statistical characteristics of long period edge waves with a view to combining these phenomena with other more frequently occurring events.

The assumption made with regard to the rate of occurrence of edge waves is that an expected number of 5 events per year occur along the entire coastline from Port Nolloth to Port Elizabeth. Table 6.4.2. summarizes the available data with regard to intervals between events.

TABLE 6.4.2.

INTERVAL ANALYSIS

AVERAGE EVENTS PER HOUR	0.00057
NUMBER OF EVENTS PER YEAR	5
EXPECTED DURATION OF CALMS	1745
RATE OF OCCURRENCE (CALMS) PER HOUR	0.00057

TABLE 6.4.1.

<i>LARGE SCALE EDGE WAVE EVENTS OFF SOUTHERN AFRICA</i>					
REFERENCE	DATE	MAX WAVE (mm)	AVERAGE WAVE PERIOD (min)	TIME	PORT
Darbyshire(1963a)	29/09/58	762	70min	—	Cape Town
Darbyshire(1963a)	1958-61	—	20-30	—	Witsands
Darbyshire(1963b)	17/10/61	—	20min	9 hrs	Port Nolloth
Darbyshire(1963b)	17/10/61	—	30min	7 hrs	Strompneus Bay
Darbyshire(1963b)	17/10/61	—	30min	12 hrs	Cape Town
Darbyshire(1963b)	18/03/62	457	20min	12 hrs	Cape Town
Shillington(1985)	11/05/81	250	20min	10 hrs	Simonstown
Shillington(1985)	11/05/81	600	20min	12 hrs	Mossel Bay
Shillington(1985)	11/05/81	450	20min	10 hrs	Port Elizabeth
Shillington(1985)	16/04/81	220	20min	8 hrs	Simonstown
Shillington(1985)	16/04/81	1000	12min	7 hrs	Mossel Bay
Shillington(1985)	16/04/81	250	20min	7 hrs	Port Elizabeth
Shillington(1985)	29/09/83	800	15min	12 hrs	Mossel Bay
Shillington(1988)	01/06/86	1200	20min	16 hrs	Port Nolloth

The available statistics on edge waves may be inferred as follows :

TABLE 6.4.3.

STATISTICAL ANALYSIS

	MAGNITUDE (mm)	DURATION (hr)
MEAN	250	8
STANDARD DEVIATION	not known	not known
MINIMUM	100	6
MAXIMUM	1200	12
NO OF EVENTS/YR	5	5

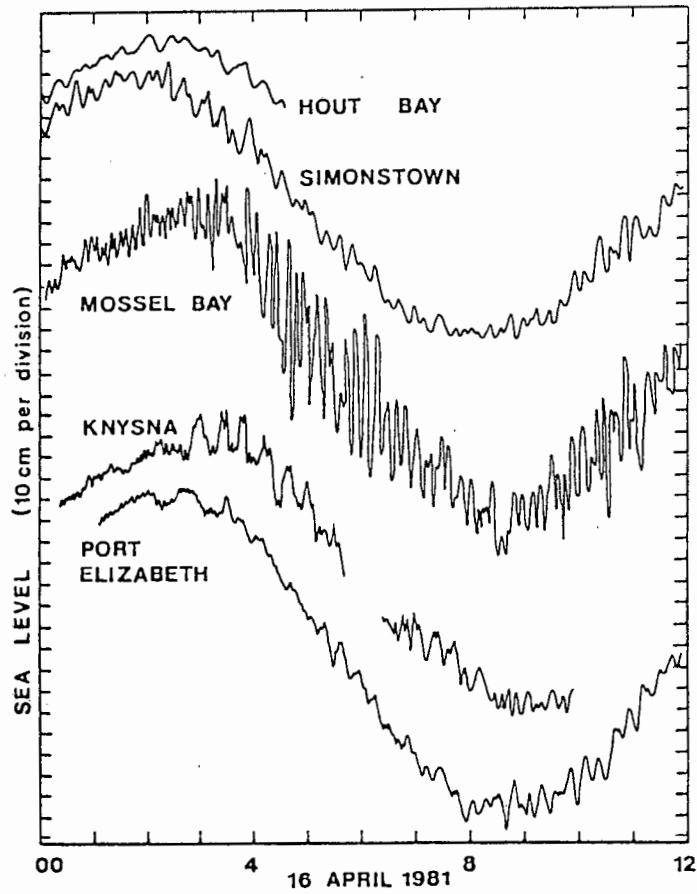
Due to the lack of data, Port Nolloth and Mossel Bay are assumed as having the same characteristics whilst Richards Bay is considered as not experiencing edge waves. It should be noted that although Port Nolloth and Mossel Bay appear to respond to edge waves in a similar fashion, it is not necessarily correct to assume that intermediate ports such as Simons Bay and Cape Town will display the same characteristics. More detailed work is required in this regard. Figure 6.4.1. represents a typical edge wave event along the southern African coastline.

6.5. INTERACTION BETWEEN PROCESSES

The interaction between various processes has a particular bearing on the manner in which these phenomena are modelled and hence the nature of the results. In this section the results of three investigations are described in respect of meteorologically induced events. Wind wave versus shelf wave events, and local wind speed and wind waves versus shelf wave events were subject to a comprehensive correlation analysis.

FIGURE 6.4.1.

TYPICAL EDGE WAVE EVENT ON THE COAST OF SOUTH AFRICA
(Shillington (1985))

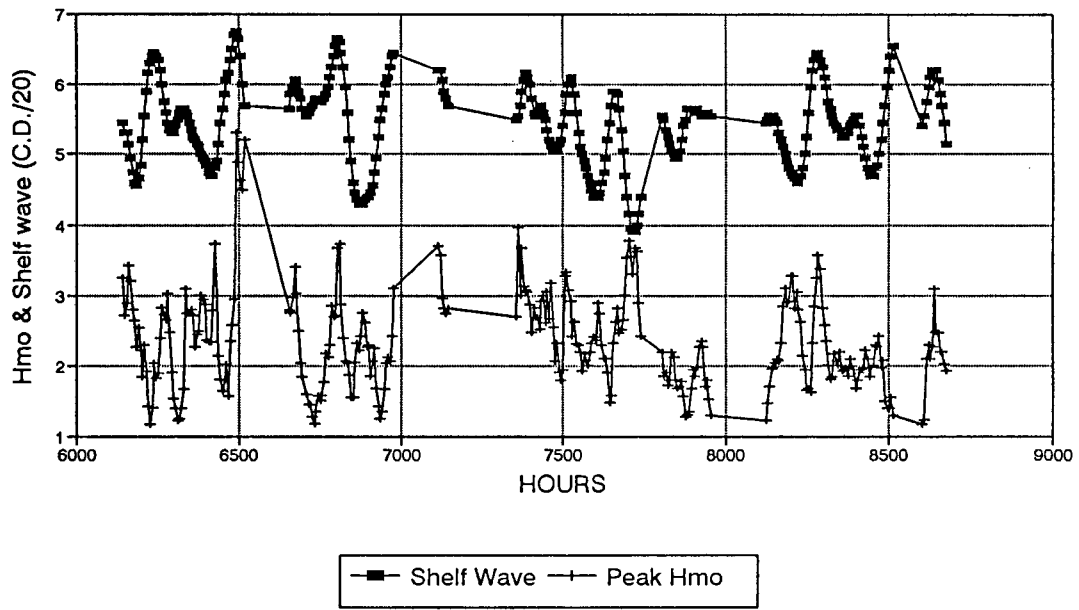


6.5.1. *Wind Waves and Shelf Waves*

In order to develop a stochastic model for evaluating combined probabilities of occurrence for different events, it is necessary to develop an understanding of the relationship between the different responses. Both shelf waves and gravity waves emanate from meteorological phenomena. From a physics viewpoint gravity waves are essentially associated with wind stress whilst shelf waves relate more closely to moving low pressure systems (De Cuevas (1985)). The approach taken here was to compare the available joint time series for wind and shelf wave height values with available synoptic charts. The primary objective was to develop some overall understanding of the relationship with a view to modelling the process. The major limitation related to the lack of sufficient information and the actual complexity of the processes under consideration.

The correlation analysis was limited to the assessment of Mossel Bay data. A further limitation was the shortage of data where gravity wave values, shelf wave records and synoptic weather maps were continuous for extended periods of time. An evaluation was made of the 1987 and 1988 six hourly time series. A visual assessment indicated that increases in shelf wave height generally preceded an increase in Hmo values. It would appear that not all increases in Hmo (storm events) are associated with a change in shelf wave height whilst most shelf wave activity is associated with a change in Hmo values (see figure 6.5.1.1). If the joint time series is compared with the prevailing synoptic conditions, there would appear to be a correlation between the frontal system and Hmo and shelf wave magnitude. From the data analyzed, namely large events for 1987 and 1988, peak significant wave height would appear to lag peak shelf wave height by some 5-60 hours, whilst the frontal system itself would appear to lead the peak shelf wave height by 5-24 hours. Only two shelf wave events were noted which could not be associated with Hmo and frontal systems. In this instance there appeared to be a stronger relationship to the passing of a coastal low. In spite of this visual evaluation, no clear relationship between peak shelf wave magnitude and peak significant wave height could be identified. It would appear that the magnitude of shelf wave events relate to the location of the moving low pressure cell offshore. The closer it approaches the actual measuring station the larger the induced event. On the other hand storm events (peak Hmo) appear to relate to the extent of the development of the frontal

FIGURE 6.5.1.1
JOINT TIME SERIES PLOT (Mossel Bay)



system. Intuitively, one would expect that both Hmo and shelf wave height would also relate directly to the magnitude of pressure gradient, the propagation speed, duration and fetch. These parameters could not be investigated in detail due to insufficient data. Cross correlation analysis runs were undertaken for several years of joint records for Mossel Bay. Some of these results are given in figure 6.5.1.2. No inter annual trend could be identified from this exercise in spite of the apparent existence of weak relationships.

Apart from the apparent relationship seen in figure 6.5.1.1, the joint time series analysis was not able to demonstrate any significant relationship. If one considers the general statistical properties, presented in earlier sections, then further information regarding this relationship may be inferred. Some of these characteristics are summarized in table 6.5.1.

TABLE 6.5.1.

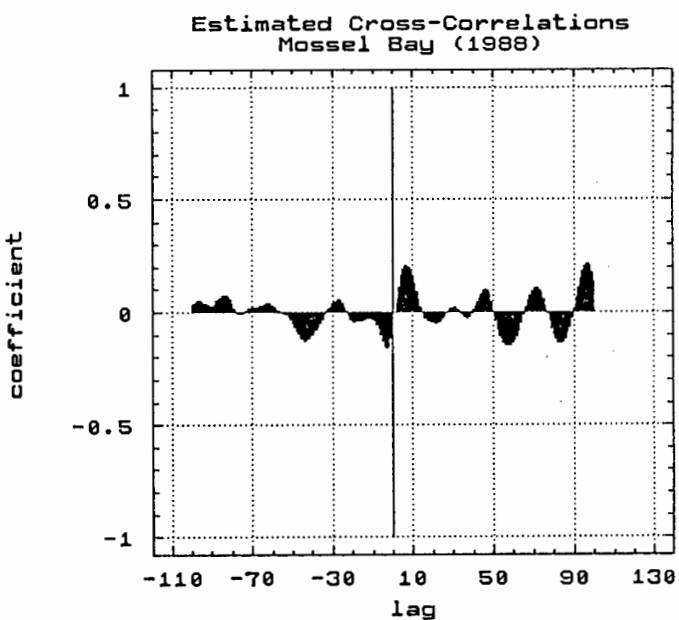
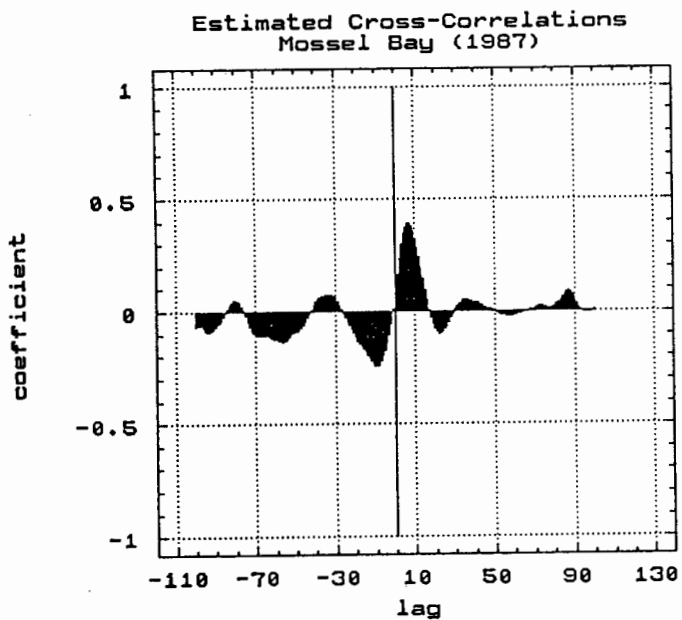
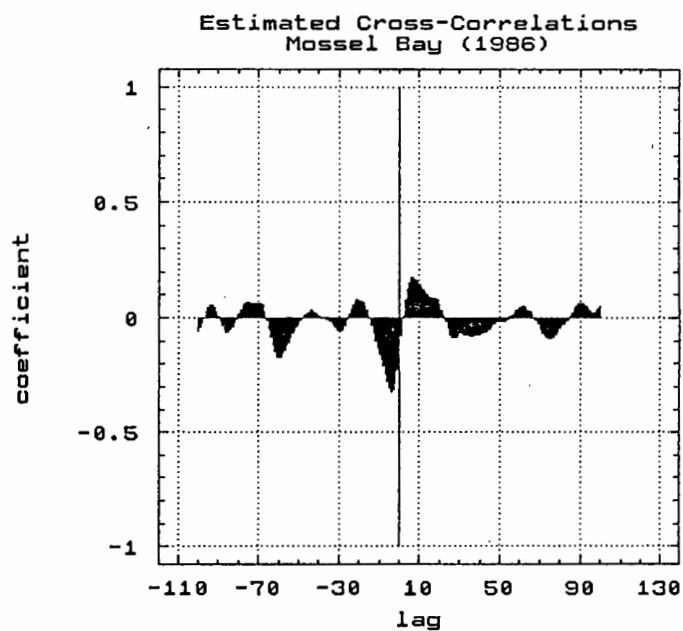
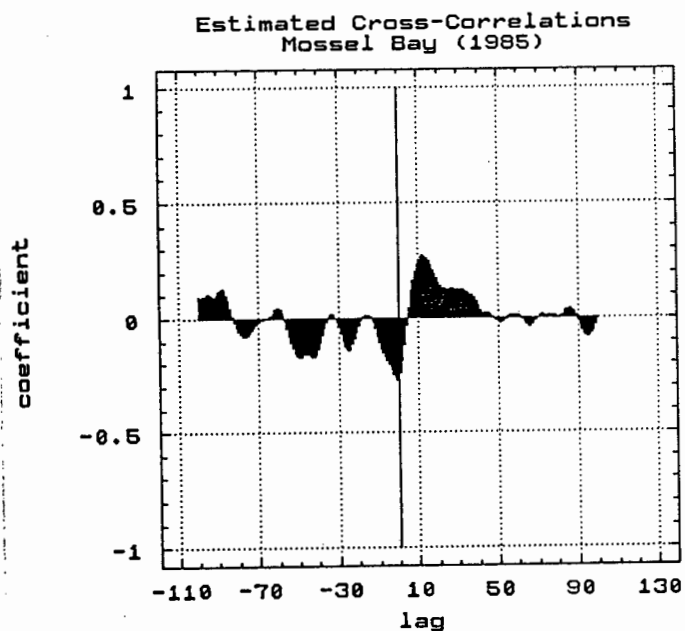
COMPARISON OF STATISTICAL PROPERTIES

AVERAGE PROPERTIES	MOSSEL BAY		RICHARDS BAY		PORT NOLLOTH	
	Shelf	Wind	Shelf	Wind	Shelf	Wind
	Waves	Waves	Waves	Waves	Waves	Waves
No Event/year	35.4	80	17	12	15	59
Duration (hrs)	32	25	43	16	27	20
Intervals (hrs)	215	154	470	950	555	130

Whilst the expected values cannot be used to preclude any form of relationship it would appear that no obvious trend in the occurrence rate or duration of these events exists. This may indicate that the scale of the generating or response mechanisms are different. On this basis it will be assumed that no significant relationship exists between shelf waves and gravity waves in this region. For the purpose of modelling these processes they will be considered as statistically independent.

FIGURE 6.5.1.2.

CROSS CORRELATION RESULTS



6.5.2. *Local Wind Speed, Shelf Waves and Wind Waves*

It is known that wind stress over water induces surface currents in the direction of the wind and horizontal forces on the water surface. In order to assess the relationship between these parameters, hourly wind speed data from Gouriqua (some 30 kms south west of Mossel Bay) were compared with tidal residuals from Mossel Bay harbour and significant wave height from the Gouritzmond waverider. Data were analyzed for January and June 1987. The compiled data sets representing wind speed versus shelf wave height and wind speed versus peak significant wave height are plotted in figures 6.5.2.1. and 6.5.2.2. respectively. These results indicate that no apparent relationship can be identified between these parameters on a one to one basis. A time series plot of wind speed, wave height and tidal residual (figure 6.5.2.3.) does not indicate that any significant time lag relationship exists.

It would appear that local wind speed should be considered as independent from shelf waves and significant wave height. This can be attributed to the fact that waves measured at Mossel Bay are generally the result of distant storms and therefore are unlikely to relate to local wind speed. Similarly, shelf waves are the result of synoptic scale processes which are not necessarily related to local winds. It should however be expected that specific storm events will occur when all three parameters occur simultaneously. At the present level of available data it is not possible to develop any empirical relationship between these parameters. It is expected that at, specific locations, some form of dependency will exist between wind speed/wind waves and shelf waves whilst at others they will be essentially independent of one another. Wind effects, although important at specific locations will not be included in the modelling procedure. The omission of local wind effects is primarily aimed at limiting the scope of this study to the most significant processes affecting sea level. This by no means implies that in specific circumstances wind, sea level and waves will not play a major role in coastal engineering design.

6.5.3. *Edge Waves, Shelf Waves and Wind Waves*

The assessment of the relationship between edge waves and shelf waves, and edge waves

FIGURE 6.5.2.1
Shelf Wave Height vs Windspeed (Jan 87)

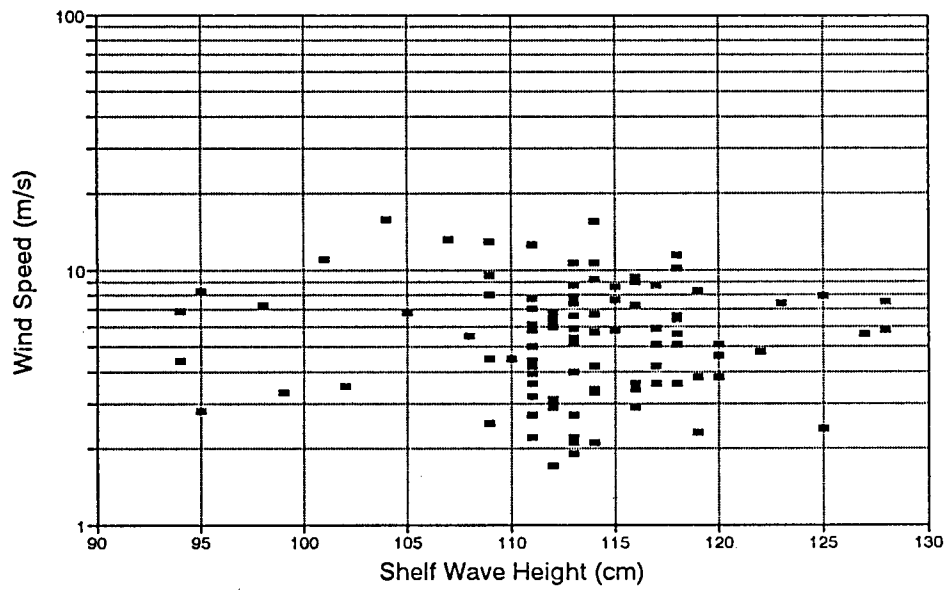


FIGURE 6.5.2.2
Wind Wave Height vs Windspeed (Jan 87)

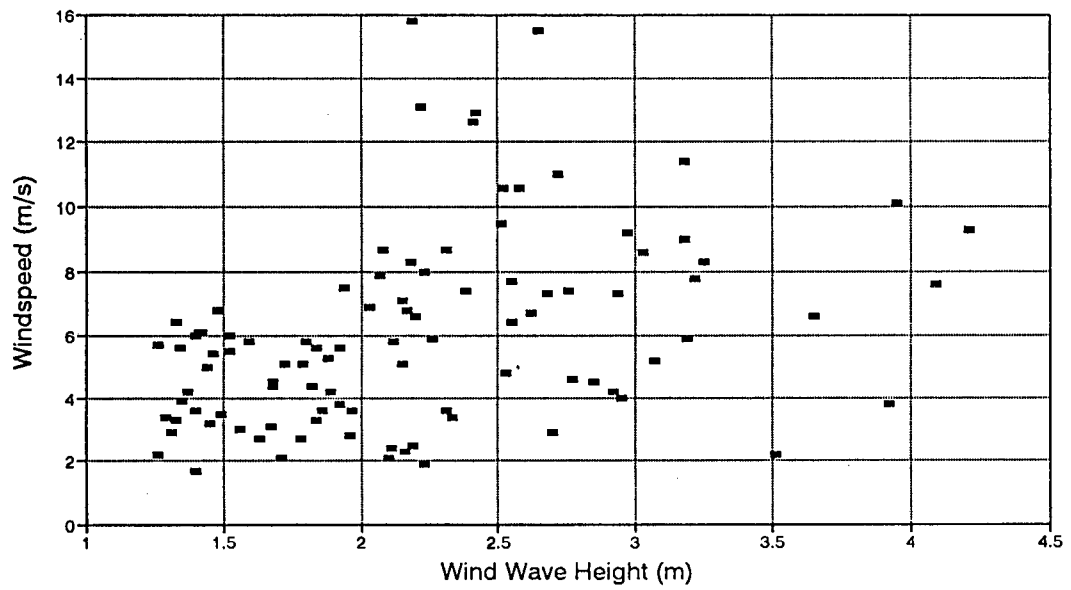
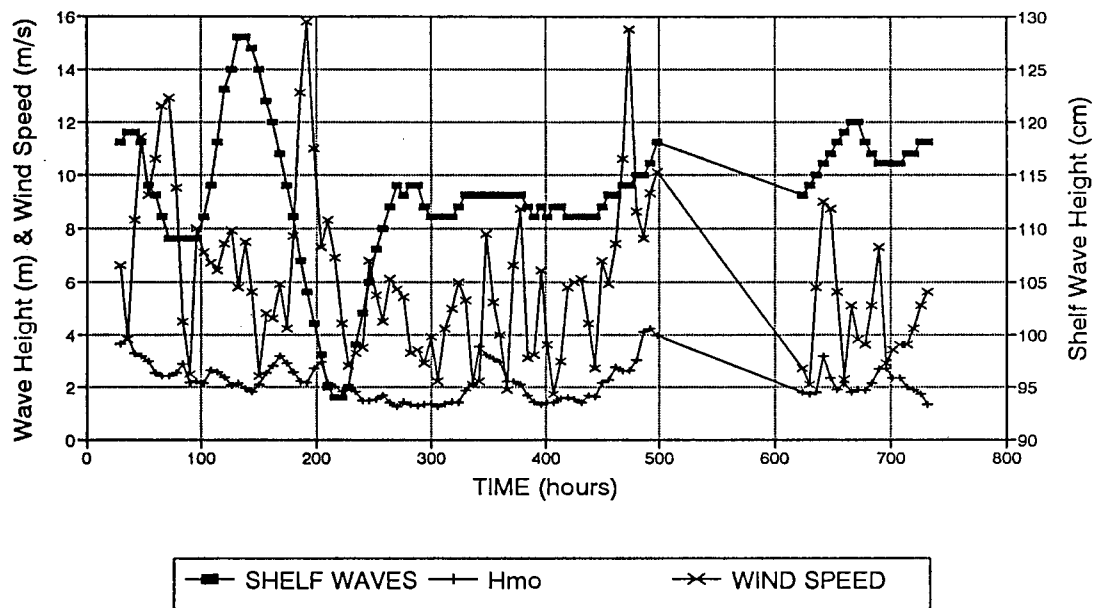


FIGURE 6.5.2.3
TIME SERIES PLOT (Jan 87)



and wind waves is clearly restricted by the lack of sufficient data for the former process. Within this framework the following inferences can be made. If the events (post 1980) are compared with the available shelf waves and wind wave records for these periods it can be seen that no apparent relationship exists between these processes.

Shillington (1985) confirms that edge waves are not generally associated with storm conditions. If it is considered that edge waves are generated by rapidly moving micro pressure oscillations along the continental shelf then it may be concluded that wind waves, which are a function of wind speed, duration and fetch are not likely to be generated by the same process. Once again it would appear that the major differences between these processes relates to both time and space scales. On this basis edge waves will be considered as statistically independent of other events.

SUMMARY

This chapter has concentrated on the assessment of the four major processes affecting design sea level in southern Africa. The primary aim has been to characterize the underlying properties with a view to the development of a stochastic convolution model. Chapter 7 describes the development, structure and verification of this model.

CHAPTER 7

MODEL DEVELOPMENT AND VERIFICATION

The primary objective of this thesis, as set out in previous chapters, is to develop a full probabilistic approach for the assessment of design sea level, with specific reference to southern Africa. This chapter describes the model which has been developed with a view to attaining this goal. The model combines all the known information with regard to the natural processes affecting sea level (such as tides, shelf waves, wind waves, and edge waves), for a specific location, in order to obtain a quantitative assessment of design conditions.

Chapter 7 is divided into four main sections. Section 1 is essentially of an introductory nature with a view to setting out the basis of the model. Section 2 systematically addresses the issues relating to the validation of components of the model whilst section 3 discusses the validation of the convolution model itself. Section 4 compares the results obtained using the model with those obtained using a conventional engineering approach to design sea level and finally section 5 considers the sensitivity of the model to record duration.

7.1. BASIS OF THE MODEL

The model has been designed to combine known probability distributions, based on observed data, for both deterministic and stochastic processes affecting sea level. A Monte Carlo simulation technique is used to generate hourly values for sea level for each process. These randomly distributed events are characterized by fitted or actual observed distributions for event magnitude, duration and recurrence interval. The model is capable of setting dependency criteria between processes and event characteristics based on the nature of conditions at the site under consideration. For the purposes of this study, processes will be considered as statistically independent. Yearly time series of hourly events are simulated and then integrated according to their dependency structure. The model is run over several million hours until satisfactory convergence, in the results, is obtained. The model is capable of outputting any design return period of interest (ie: 1/50 or 1/100 year) for sea level, wind wave heights, edge wave heights or any combination thereof, thus providing the full range of environmental design conditions. The model takes into account the 18.6 year sun/lunar cycle in

describing the full extent of the tidal level distribution. Furthermore the entire dependency structure is accounted for by using nineteen years of serially correlated data.

The stochastic processes affecting sea level, notably shelf waves, wave setup and edge waves are dealt with in the same manner and hence only the overall approach will be described. Probability distributions, based on the data analyzed for these processes in chapter 6 are used to describe the magnitude and duration of events and the recurrence intervals between these events. The model assumes that events occur according to a triangular time series distribution (ie: the maximum values occur equidistant from the commencement and termination of the event). Accordingly random events, corresponding to the particular distribution, are generated to form an hourly time series for each process. These hourly time series can then be combined with the hourly tidal values, for a particular year, to obtain a simulated sea level. A flow diagram is given in figure 7.1.1..

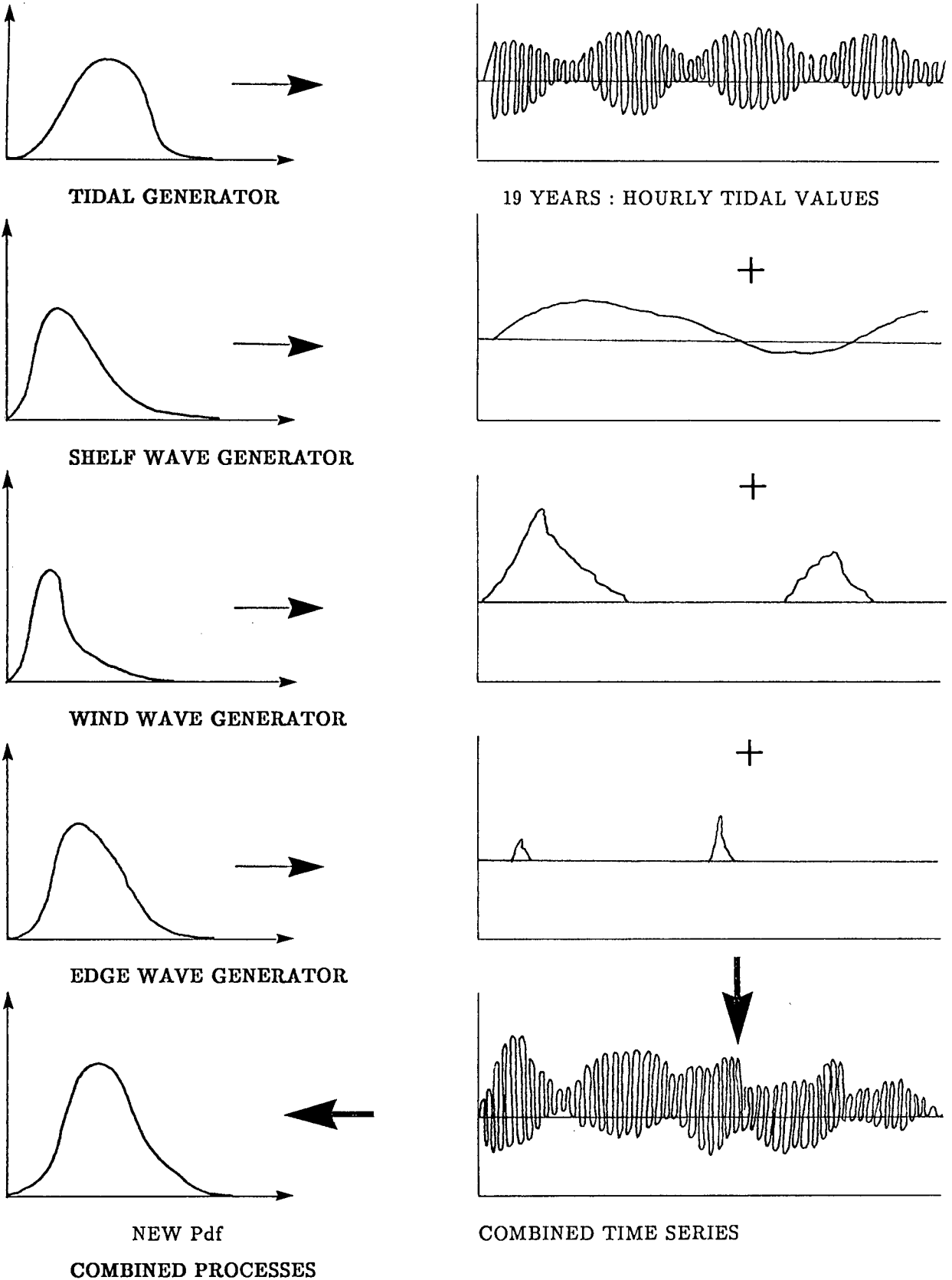
The maximum simulated sea level values for each year are stored for model runs over a period of 100 years. The resulting annual maximum values are then sorted in ascending order and averaged with previous simulation runs using the same parameters. The greater the number of variables, the greater the number of simulations required.

The model makes use of the actual joint cumulative distributions between event magnitude and duration for the generation of synthetic events for wave setup and shelf waves. Due to the lack of recorded edge wave data, assumptions have been made regarding the relationship between event magnitude and duration. A linear relationship $y = mx + c$ was used based on the available statistics obtained from Shillington (1980).

The model can accommodate various forms of dependency between different processes. This may be achieved by incorporating known conditional probability distributions between events or by observed deterministic relationships. As the model requires a high degree of statistical accuracy regarding the simulation of the natural processes additional useful information may be extracted which greatly enhances the value of the design sea level, notably the associated design wave height.

FIGURE 7.1.1.

MODEL LOGIC DIAGRAM



The manner in which different processes are combined is a function of the specific site and structure under consideration. An enclosed harbour scenario will not be affected by wave setup in the same way as an open coastline, similarly, a breakwater will be somewhat different to a slender offshore structure design. These aspects will be dealt with in more detail regarding the application model and interpretation of the results.

7.2. MODEL COMPONENT VALIDATION

The purpose of this section is to describe the validation of the model. The model will be assessed in terms of the validity of its individual components and their ability to realistically simulate natural processes in order to obtain design information. The primary components of the model may be listed as follows:

1. *Tidal generator*
2. *Random number generator*
3. *Shelf wave event generator*
4. *Wind wave event generator*
5. *Edge wave event generator*

These process generators may be verified against available field data. The second major area of testing regards the integration of individual events with a view to generating a simulated hourly sea level record. Of primary interest will be the correlation between sea level observations at various ports and the simulated sea level at these ports.

The approach taken will be to use standard goodness of fit tests such as Chi square and Kolmogorov - Smirnov tests to evaluate the appropriateness of simulated and observed relationships.

7.2.1 *Tidal Generator.*

The model used for the prediction of hourly tidal values has been discussed earlier. The hourly values generated for Mossel Bay have been compared with an alternative model used by the S.A.N. Hydrographer. On the basis of the work undertaken in chapter 6 the hourly predicted tide values are considered adequate for the purposes of the model.

7.2.2. *Random Number Generator*

Numerous random number generators are available for use on micro computers. As the stochastic simulation model is to a large extent dependent on random numbers for the definition of events, it is important that true randomness is maintained. A number of problems exist relating to frequently used algorithms. The most common problem relating to random number generators or pseudo random generators is that the algorithms used tend to repeat themselves after a specific number of values. The algorithm used by the model is of the mixed congruential family of generators based on the form:

$$U_{i+1} = (a U_i + c) \pmod{m}, i = 1 \dots n \quad (1)$$

The maximum sequence of random numbers without repetition is determined by the constant m . Alexander (1985). The mixed multiplicative congruential generator used in the model random number subroutine takes the form :

$$U_{i+1} = \text{Mod}(D, M) / M$$

$$\begin{aligned} \text{where } M &= 2^{20} \\ T &= 2^{10} + 3 \\ D &= X_i * T \\ X_1 &= \text{Seed value} \\ X_{i+1} &= \text{Mod}(D, M) \\ \text{Mod}(D, M) &= D - (\text{INT}(D/M) * M) \end{aligned}$$

Tests for randomness are discussed by Alexander (1985). These can be summarized as follows:

1. Successive events in a sequence of numbers must be statistically independent.
2. The generated values must be uniformly distributed over the specific range.
3. Values must not repeat themselves within a given data set.

Alexander (1985) proposes two tests which can be used to assess the above criteria. The test for randomness entails comparing the number of times n that $U_{i+1} > U_i$ with the number of times m that $U_{i+1} < U_i$. The values of m and n should, after sufficient repetition be equal. This test was carried out on the model generator using 10 000 and 50 000 simulations. The following results were obtained:

(10 000)	$n = 49.64\%$	(50 000)	$n = 50.04\%$
	$m = 50.36\%$		$m = 49.96\%$

The Kolmogorov - Smirnov and Chi square tests were used to test the uniformity of the random number distribution based on 999 simulations. The results may be summarized as follows:

Chi square test	Chi square	=	30.5519 with 23 d.f.
	Significance level	=	0.134205
Kolmogorov - Smirnov test	D PLUS	=	0.0196888
	DN	=	0.0240026
	D MINUS	=	0.0240026
	Significance level	=	0.999376

These results indicate a significant degree of agreement.

In order to assess the last requirement, it is necessary to evaluate the number of random numbers used by the model for each 100 year cycle per type of process simulated. Table 7.2.2.1 illustrates the number of random values used per run.

TABLE.7.2.2.1.

RANDOM NUMBERS PER RUN

PROCESS	EXPECTED No OF EVENTS /YEAR	No OF RANDOM NUMBERS /100 YRS
Shelf waves	36	18 000
Wind waves	80	40 000
Edge waves	5	15 000

As the model uses a new seed value for each 100 year cycle or upper limit of 70000 – 75000 random numbers in series are required for each run. As the value of m relates to the repetition interval for pseudo random number generators it can be seen that only after 2^{20} random numbers does this become problematic. Repetition is therefore not considered to be a problem with regard to the random number generator used.

7.2.3. *Shelf Wave Event Generator*

The model makes use of three basic characteristics to define the nature of a shelf wave event, namely the event recurrence interval, magnitude and duration. The generation of an hourly time series is accomplished using an idealized triangular distribution with the maximum magnitude equidistant between the initiation and termination of the events. A flow chart of the shelf wave generation routine is given in Figure 7.2.3.1.

Event recurrence intervals are generated by fitting the distribution for the observed data to an exponential distribution. If Mossel Bay data for 1980–88 are used the following comparison between generated and observed data are listed in table 7.2.3.1.

FIGURE 7.2.3.1.

SCHMATIC LAYOUT
SHELF WAVE GENERATOR

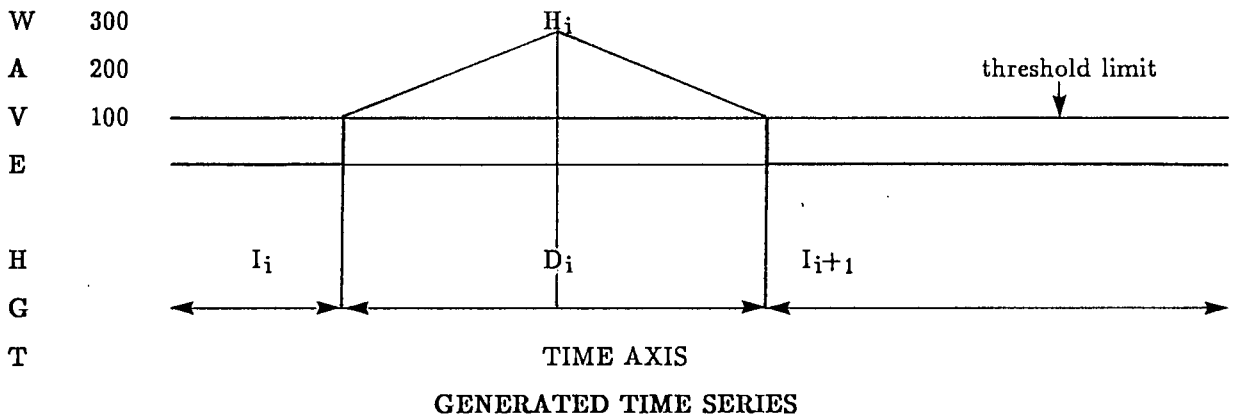
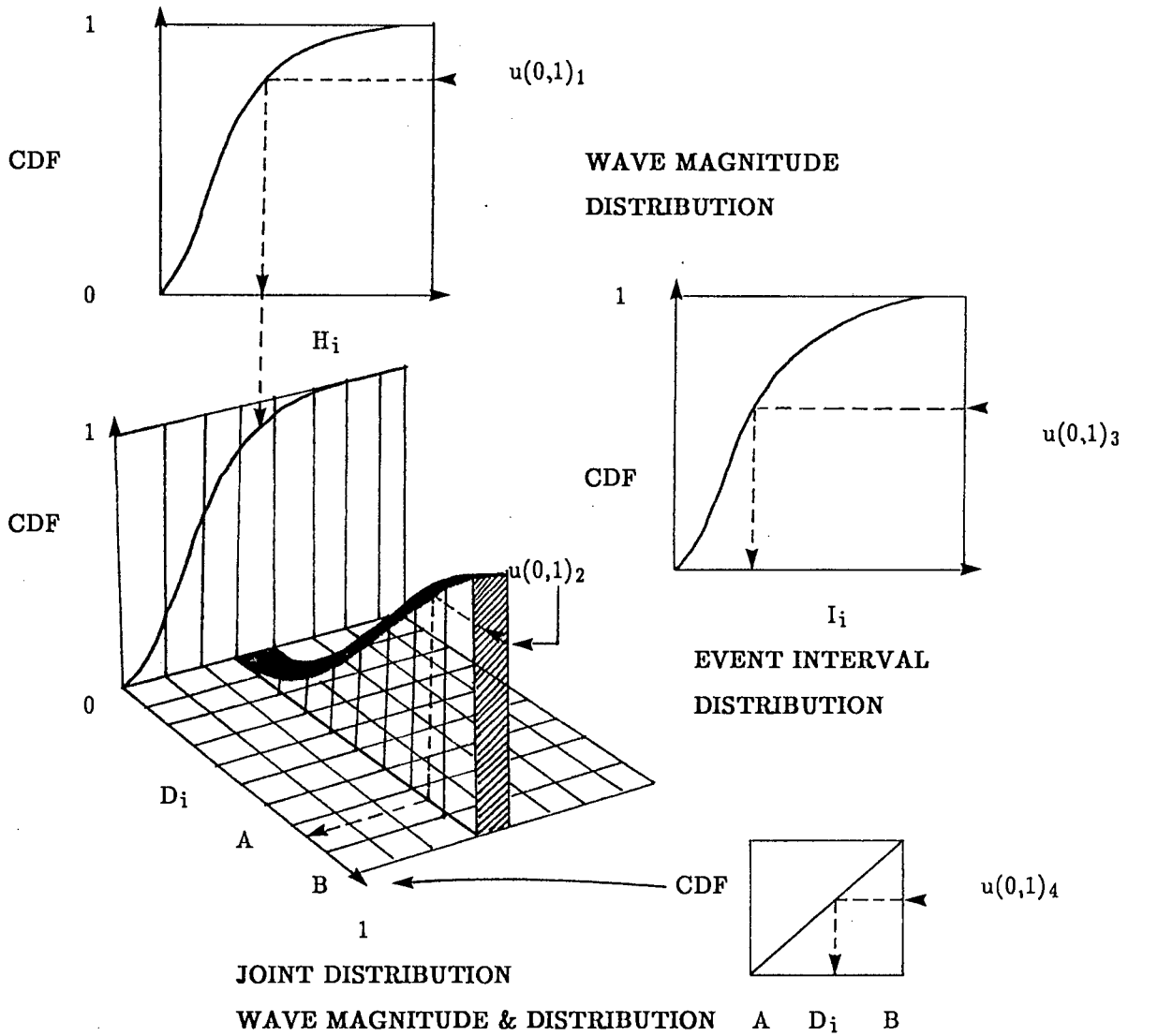


TABLE.7.2.3.1

INTERVAL GENERATION (hrs)

	OBSERVED DATA (273)	GENERATED DATA (1000)
Mean	195.00	198.00
Std	203.00	194.00
Max	2017.00	1513.00
Min	13.00	0.50
Skewness	3.43	2.07

The Chi square goodness of fit = 0.885
 and Kolmogorov - Smirnov (KS) = 0.208

These results are considered acceptable in terms of the nature of the interval value distribution. The actual occurrence rate used by the model cannot be calculated from the available interval data due to missing data and the technique used for the threshold analysis. The expected or mean rate of occurrence is calculated in chapter 6 based on the total number of independent events per year and their expected duration. The performance of the exponential distribution in the model will be discussed later in this section.

The simulation of event magnitude and duration was carried out by using the observed joint probability distribution between wave height and duration. (see table 7.2.3.1 (a)). In order to test the model's ability to accurately simulate a specific distribution, 10000 values were generated using these observed values. This resulted in the joint probability distribution given in Table 7.2.3.1.(b). It can be seen that, for all intents and purposes, these two distributions are the same. It is apparent that the model can replicate a given observed distribution, however it is clear that a limited data set will not include all possible values of wave magnitude and duration. Therefore it would be more realistic to use a fitted distribution for one of the variables. From chapter 6 it can be seen that event magnitude follows an extreme type I distribution. Therefore, the approach taken has been to use the fitted extreme type I distribution for magnitude to

SHELF WAVE HEIGHT AND DURATION DATA

(Mossel Bay)

OBSERVED VALUE DISTRIBUTION

TABLE 7.2.3.1(a)

DUR (hrs)	EVENT MAGNITUDE (mm)									
	225	300	375	450	525	600	675	750	825	900
10	0.1007	0.0208	0.0208	0.0069	0	0	0	0	0	0
27	0.1319	0.184	0.059	0.0139	0.0035	0.0035	0	0	0	0
42	0.0556	0.066	0.0486	0.0139	0.0104	0.0069	0.0035	0	0	0
57	0	0.0278	0.0278	0.0208	0.0069	0.0069	0	0	0	0
72	0	0.0313	0.0208	0.0243	0.0069	0	0	0	0	0
87	0	0.0139	0	0.0174	0	0	0	0	0	0
102	0	0	0.0035	0.0069	0.0035	0	0	0	0	0
117	0	0	0	0.0035	0.0035	0	0	0	0	0.0035
132	0	0.0035	0	0.0035	0	0	0	0	0	0
147	0	0	0.0035	0.0035	0	0.0035	0	0.0035	0	0

ACTUAL MAGNITUDE

PREDICTED VALUE DISTRIBUTION

TABLE 7.2.3.1(b)

DUR (hrs)	EVENT MAGNITUDE (mm)									
	225	300	375	450	525	600	675	750	825	900
10	0.0954	0.0209	0.0216	0.0076	0	0	0	0	0	0
27	0.1293	0.1835	0.064	0.0131	0.0034	0.0035	0	0	0	0
42	0.0564	0.0624	0.0487	0.0143	0.0103	0.0083	0.0031	0	0	0
57	0	0.0286	0.027	0.0218	0.0072	0.0065	0	0	0	0
72	0	0.0312	0.0214	0.0257	0.0071	0	0	0	0	0
87	0	0.0145	0	0.0176	0	0	0	0	0	0
102	0	0	0.0035	0.0077	0.0037	0	0	0	0	0
117	0	0	0	0.0032	0.0036	0	0	0	0	0.0024
132	0	0.0032	0	0.0046	0	0	0	0	0	0
147	0	0	0.0036	0.0031	0	0.0027	0	0.0043	0	0

FITTED MAGNITUDE

PREDICTED VALUE DISTRIBUTION

TABLE 7.2.3.1(c)

DUR (hrs)	EVENT MAGNITUDE (mm)									
	225	300	375	450	525	600	675	750	825	900
10	0.0997	0.0183	0.0215	0.0057	0.0003	0	0	0	0	0
27	0.1325	0.17	0.072	0.0124	0.005	0.0045	0.0002	0	0	0
42	0.056	0.0623	0.0527	0.0126	0.0123	0.0051	0.0035	0	0	0
57	0	0.0251	0.0305	0.0191	0.009	0.0063	0.001	0	0	0
72	0	0.0293	0.0242	0.0213	0.0082	0	0	0	0	0
87	0	0.0126	0.0024	0.0162	0.001	0	0	0	0	0
102	0	0	0.0036	0.0065	0.004	0	0	0	0	0
117	0	0	0.001	0.0032	0.0036	0	0	0.0013	0.0012	0.0004
132	0	0.0037	0	0.0042	0	0	0	0	0	0
147	0	0	0.0035	0.0031	0.0011	0.0024	0.0027	0.0014	0	0

generate synthetic wave magnitude. As there is no obvious relationship between event magnitude and duration the joint probability distribution between magnitude and duration, based on the observed data, is used to define the event duration once magnitude has been established. Table 7.2.3.1(c) illustrates the predicted distribution for magnitude and duration using this approach. It should be noted that although the general pattern of the distribution is maintained, differences do exist because of the fitted extreme type I distribution used for the definition of wave magnitude.

An alternative approach here could be to determine some form of relationship between duration and magnitude for each location. However, due to the uncertainty relating to our understanding of the causative processes and hence the parameters influencing event duration, it is quite possible that a fitted relationship will introduce subjective bias into the model, thus the current approach of using the observed distribution.

A final test carried out on the shelf wave generator was to assess the actual hourly time series generated for a given number of years. Then, using the same data analysis techniques as described in chapter 6, an assessment was made of the statistical properties of the two data sets. By simulating 78894 hours of data using the model the following comparisons can be made between the observed and predicted values.

TABLE 7.2.3.2

OBSERVED vs PREDICTED STATISTICS

	OBSERVED DATA			PREDICTED DATA		
	Mag(mm)	Dur(hr)	Int(hr)	Mag(mm)	Dur(hr)	Int(hr)
Mean	285.30	32.50	194.50	286.90	32.80	219.63
Std	99.91	26.41	202.90	95.99	24.40	204.83
Min	124.00	8.00	13.00	108.00	9.00	12.00
Max	696.00	147.00	2017.00	657.00	145.00	1571.00

It can be seen that a good correlation exists between the statistics of the observed and predicted data. The discrepancy between the mean observed and predicted interval duration between events can be related to the impact of missing data on the original data set but compares well with the alternative method proposed for assessing intervals. Both the magnitude and interval values are fitted to given distributions, extreme type I and exponential respectively and therefore it can be expected that if the original data fits, that the generated data will be correct. The duration data are generated from the observed distribution. The observed and predicted distributions for duration are given in figure 7.2.3.2. This fit appears good.

If it is accepted that statistically independent shelf wave events can be characterized by a magnitude, duration and recurrence interval then it can be concluded, from this section, that shelf waves can be modelled in such a way as to maintain the overall statistical characteristics of the observed data. Shelf waves are simulated in the convolution model on this basis.

7.2.4. *Wind Wave Event Generator*

The approach to the validation of the wind wave generator used in the model is largely the same as the procedure used in section 7.2.3. The wind wave event time series is characterized by the event magnitude, duration and recurrence interval. (see figure 7.2.4.1.) The model can allow for transformation of deep sea wind wave data to a nearshore location by the inclusion of wave directions, refraction, diffraction, setup and runup coefficients. These factors are site specific. This section will deal exclusively with ability of the model to generate deep sea wind waves. It has been shown in chapter 6 that wind wave data possess a well defined seasonality bias. Consequently, most discussion is in terms of the independent summer and winter data sets.

An analysis of the available waverider data for Mossel Bay, with regard to the recurrence period or intervals between events exceeding 3 metres in wave height and 6 hours in duration for summer and winter data sets produced the following results (see table 7.2.4.(a)).

FIGURE 7.2.3.2
Observed vs Predicted Durations

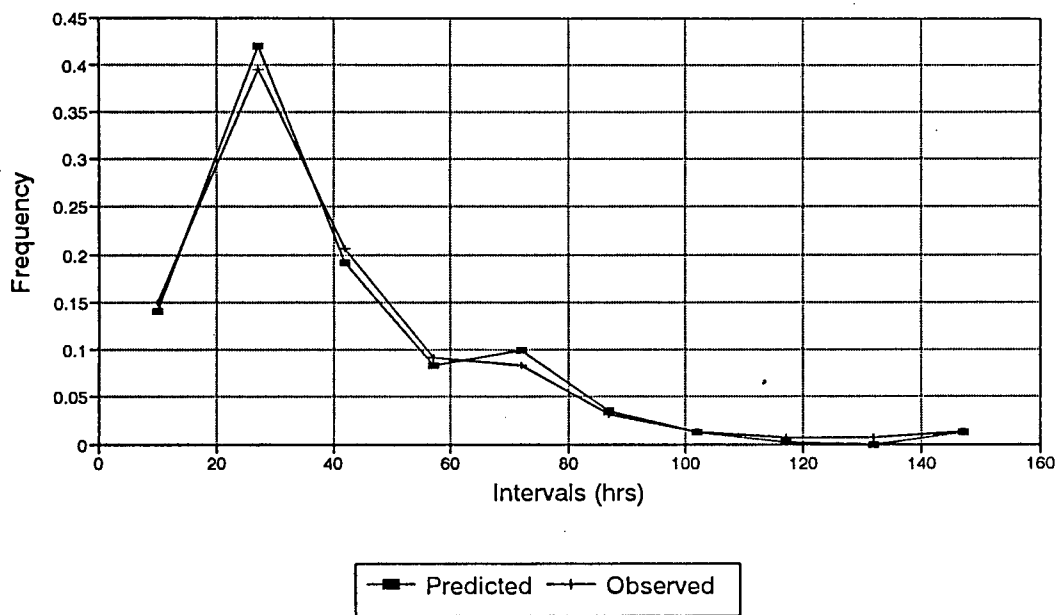
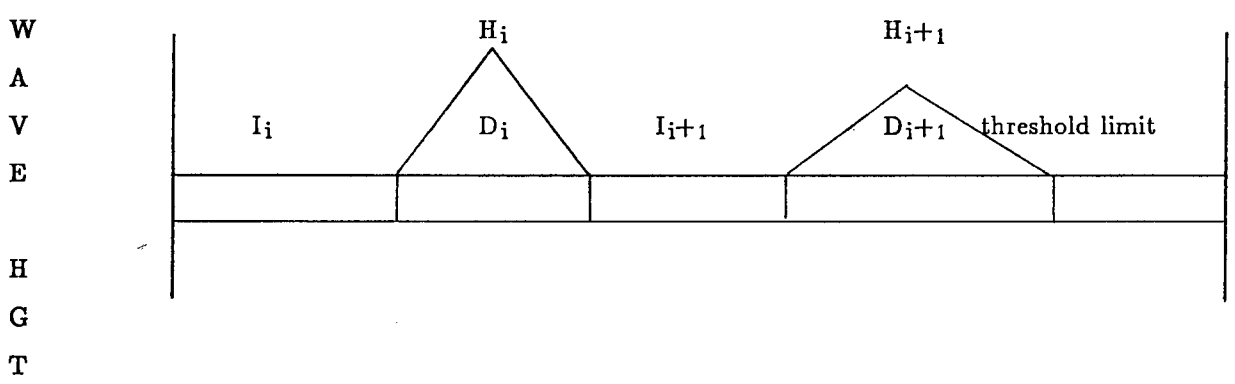
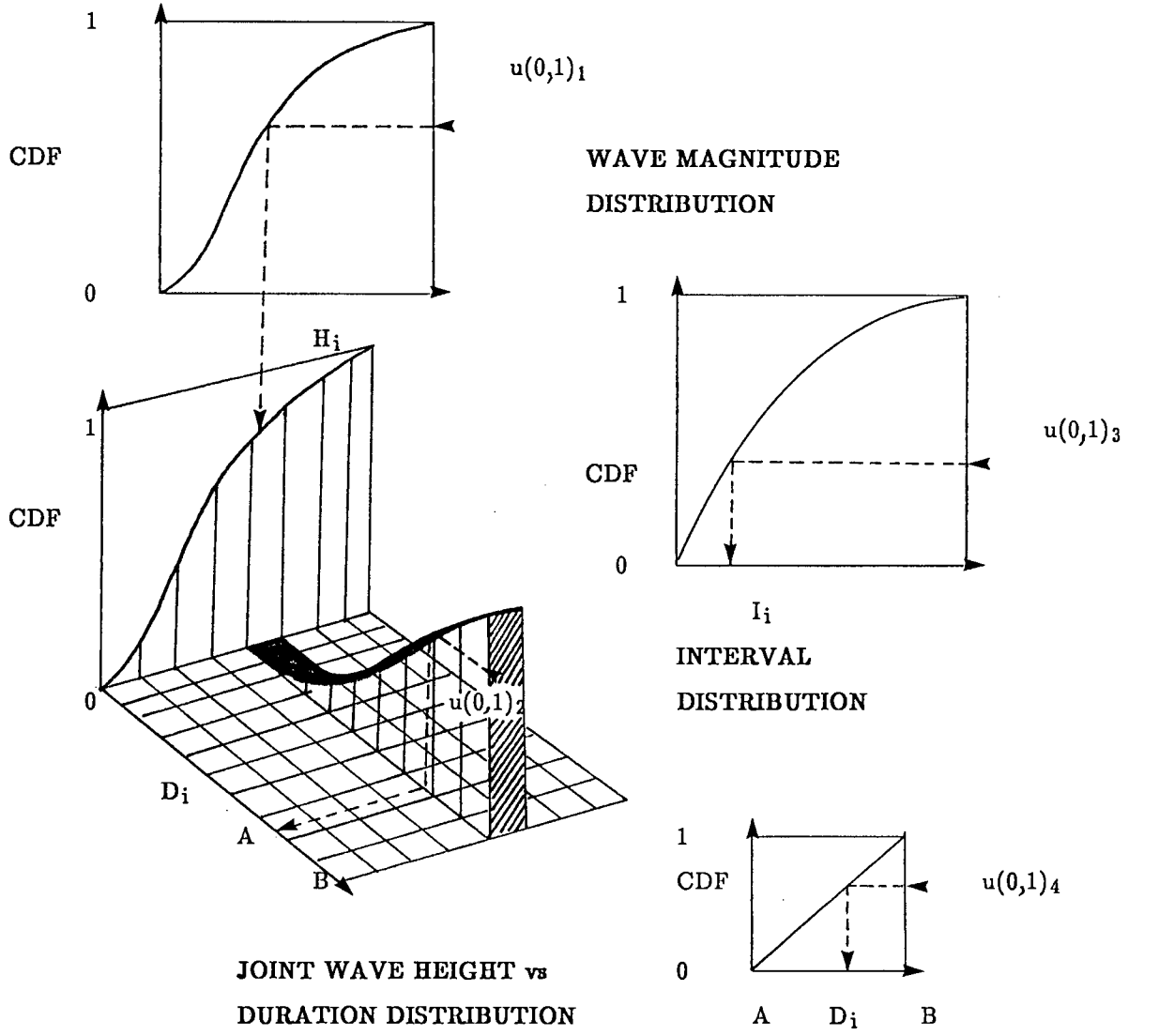


FIGURE 7.2.4.1.

SCHEMATIC LAYOUT
WIND WAVE GENERATOR



GENERATED TIME SERIES

TABLE 7.2.4.(a).

OBSERVED INTERVAL STATISTICS (1978 - 1991)

	SUMMER	WINTER
Mean	55.62	95.21
Std	65.11	99.37
Min	6.00	6.00
Max	594.00	504.00
Skewness	2.95	1.57
No of events	300	466

Both data sets could be fitted to an exponential distribution with Chi square significant levels 0.650 and 0.314 respectively. These can be compared with 1000 generated values given in Table 7.2.4.(b).

TABLE 7.2.4.(b).

PREDICTED INTERVAL STATISTICS

	SUMMER	WINTER
Mean	55.61	92.78
Std	55.45	91.61
Min	0	0
Max	432.00	591.00
Skewness	1.95	1.81
No of events	1000	1000

This confirms that the nature of interval distribution follows an exponential distribution. The estimated mean value used suffers from the effects of missing data points in the original data. The alternative approach described in chapter 6 has therefore been used whereby the expected value of the summer and winter interval

SUMMER
WIND WAVE HEIGHT AND DURATION DATA
(Mossel Bay)
OBSERVED VALUE DISTRIBUTION
TABLE 7.2.4.1(a)

DUR (hrs)	EVENT MAGNITUDE (m)									
	3.25	3.69	4.13	4.57	5.02	5.47	5.91	6.36	6.80	7.24
6	0.2367	0.0967	0.02	0	0	0	0	0	0	0
15	0.02	0.13	0.0333	0.0133	0	0	0	0	0	0
30	0.0133	0.0867	0.0567	0.0567	0.0133	0	0	0	0	0
45	0	0.0167	0.0333	0.0333	0.0133	0.01	0	0.0033	0	0
60	0	0.0033	0.0267	0.02	0.01	0.0067	0.0033	0	0	0
75	0	0	0	0.01	0.0067	0.0033	0.01	0	0	0
90	0	0	0	0	0.0033	0.0033	0	0	0	0
105	0	0	0	0	0	0	0	0	0	0
120	0	0	0	0	0	0	0	0	0	0
135	0	0	0	0	0	0	0.0067	0	0	0

ACTUAL MAGNITUDE
PREDICTED VALUE DISTRIBUTION
TABLE 7.2.4.1(b)

DUR (hrs)	EVENT MAGNITUDE (m)									
	3.25	3.69	4.13	4.57	5.02	5.47	5.91	6.36	6.80	7.24
6	0.2198	0.0919	0.019	0	0	0	0	0	0	0
15	0.0195	0.133	0.0345	0.0128	0	0	0	0	0	0
30	0.015	0.0899	0.0578	0.0531	0.0141	0	0	0	0	0
45	0	0.0172	0.0352	0.0368	0.0138	0.0092	0	0.0029	0	0
60	0	0.0038	0.0303	0.0215	0.0092	0.0061	0.0032	0	0	0
75	0	0	0	0.0115	0.0075	0.0029	0.0147	0	0	0
90	0	0	0	0	0.0033	0.0031	0	0	0	0
105	0	0	0	0	0	0	0	0	0	0
120	0	0	0	0	0	0	0	0	0	0
135	0	0	0	0	0	0	0.0074	0	0	0

FITTED MAGNITUDE
PREDICTED VALUE DISTRIBUTION
TABLE 7.2.4.1(c)

DUR (hrs)	EVENT MAGNITUDE (m)									
	3.25	3.69	4.13	4.57	5.02	5.47	5.91	6.36	6.80	7.24
6	0.2106	0.1078	0.0301	0	0	0	0	0	0	0
15	0.0152	0.1196	0.0472	0.0126	0	0	0	0	0	0
30	0.012	0.0838	0.071	0.0502	0.0124	0	0	0	0	0
45	0	0.0132	0.04	0.0277	0.019	0.0039	0	0.001	0.0014	0.001
60	0.0003	0.0027	0.0311	0.018	0.0139	0.0047	0.0015	0.001	0	0
75	0	0	0.0017	0.0096	0.0092	0.0067	0.0031	0.0018	0	0
90	0	0	0	0.0002	0.0057	0.0018	0	0	0	0
105	0	0	0	0	0	0	0	0	0	0
120	0	0	0	0	0	0	0	0	0	0
135	0	0	0	0	0	0.005	0.0023	0.001	0	0

This compares well with the estimate given in chapter 6 of 89.3 events per year. (52.8 + 36.3). This result is of importance in as much as failure to generate realistic event duration and recurrence interval values will effect the number of events generated per year as duration and recurrence interval are interrelated in the time series.

Finally a test was undertaken as to the ability of the model to reflect the correct duration distribution. Figure 7.2.4.2. illustrates the observed and predicted results. The chi square level of significant = 1.0 whilst the KS test exceeds a significance level of 0.20. Based only on 14 years of data, this would appear highly satisfactory.

7.2.5. *Edge Wave Generator*

The limited data available on long period edge waves in southern Africa necessitates making assumptions with regard to their basic characteristics. Using the information summarized in chapter 6 it was possible to develop an edge wave generator for the model.

Figure 7.2.5.1. illustrates a schematic layout of the edge wave generator. Three random numbers are necessary to define the basic characteristics of an individual wave event. An event is defined in terms of the event magnitude, duration and recurrence interval. The following assumptions are made with regard to each component of the process :

1. Wave magnitude is represented by an extreme I distribution.
2. Recurrence intervals are based on an exponential distribution.
3. A linear relationship exists between event magnitude and duration.

The assumptions with regard to wave magnitude were based on past observations that large events generated by natural processes occur according to this distribution. As more work is undertaken on this subject of edge waves, so it will be possible to review this assumption. Similarly the assumption that there exists a linear relationship between event magnitude and duration was based on the observation that large events tend to persist for longer periods. The introduction of variability into the relationship in the form of a uniform distribution about the defined $y = mx + c$ slope would appear to be a possible way of making the results appear more realistic.

FIGURE 7.2.4.2
OBSERVED vs PREDICTED DURATIONS

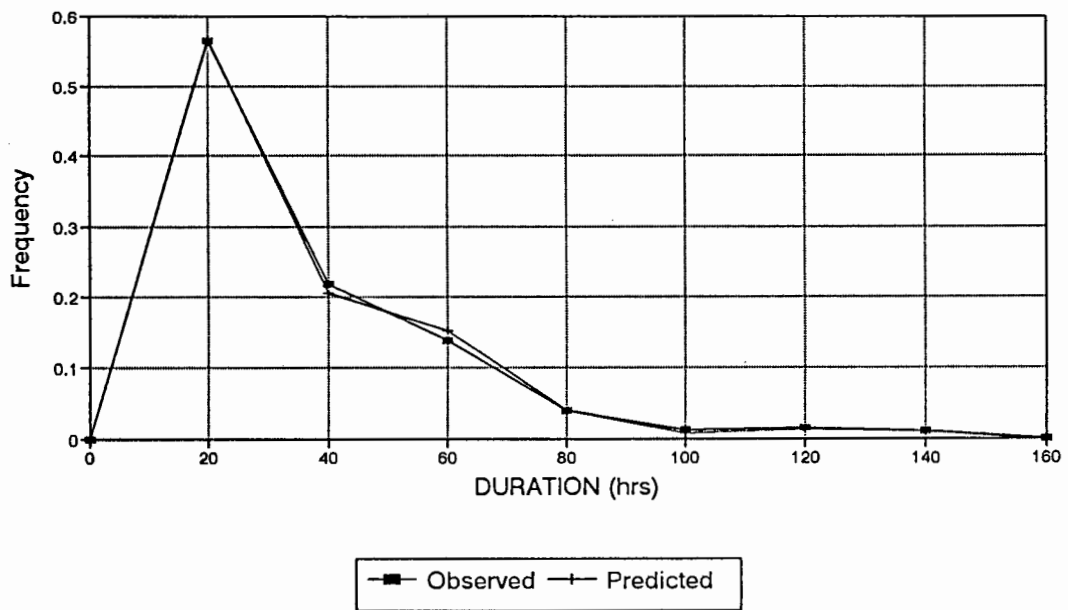
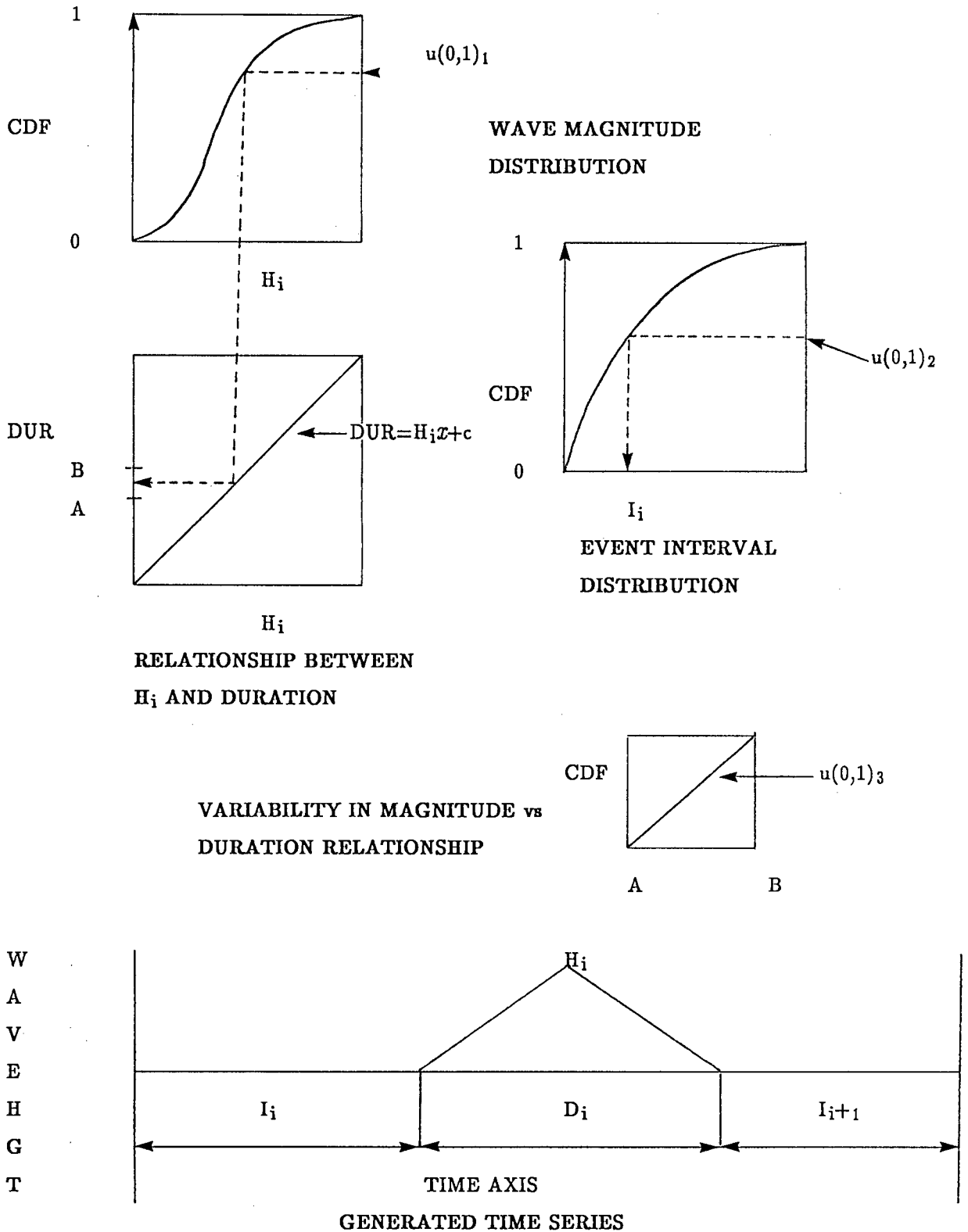


FIGURE 7.2.5.1.

SCHEMATIC LAYOUT
EDGE WAVE GENERATOR



The interval between independent events is most often modelled using an exponential distribution. Similarly, in this instance it is applicable to use this distribution based on our current knowledge of the expected number of events per year.

A further refinement which could be used in the model would be to introduce the observed tendency for events to occur during equinoxes more often than other periods of the year. This has not been included at this stage. Using the data assimilated for Mossel Bay, as described in chapter 6, the model was run with a view to reproducing the same basic characteristics of the data set. Table 7.2.5.1. summarizes these results :

TABLE 7.2.5.1.
MOSSEL BAY
OBSERVED VERSUS PREDICTED STATISTICS

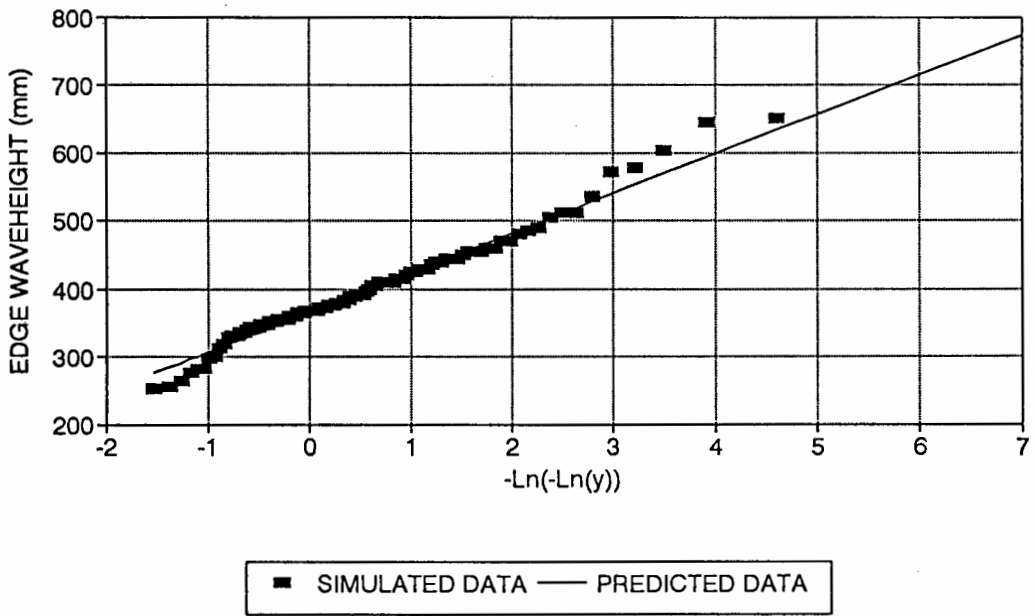
	OBSERVED DATA			PREDICTED DATA		
	Mag (mm)	Dur (hr)	Int (hr)	Mag (mm)	Dur (hr)	Int (hr)
MEAN	250	8	1753	249.1	7.5	1419
STD	100	—	—	98	0.84	—
MIN	100	6	—	35	6	—
MAX	1200	12	—	862	12	—
EVENTS/YR		5			5.4	

It can be seen from table 7.2.5.1. and figure 7.2.5.2. that the model produces the same basic characteristics assumed to represent edge waves at Port Nolloth and Mossel Bay. It should be cautioned that the assumptions pertaining to the type of distribution and the nature of edge waves themselves are all important in the interpretation of the resulting simulated data. The ultimate significance of edge waves will become more apparent in the combined model discussed in section 7.3.

7.3. CONVOLUTION MODEL VALIDATION

This section reviews the validation process with regard to the convolution model itself. The components of the model discussed in the previous section are placed into a single

FIGURE 7.2.5.2
MODEL PREDICTIONS (MB 100 years)



model in order to verify their combined responses. As no dependency structure could be identified during data analysis, all processes discussed in this section will be assumed as statistically independent.

7.3.1 *Astronomical Tide Plus Shelf Waves*

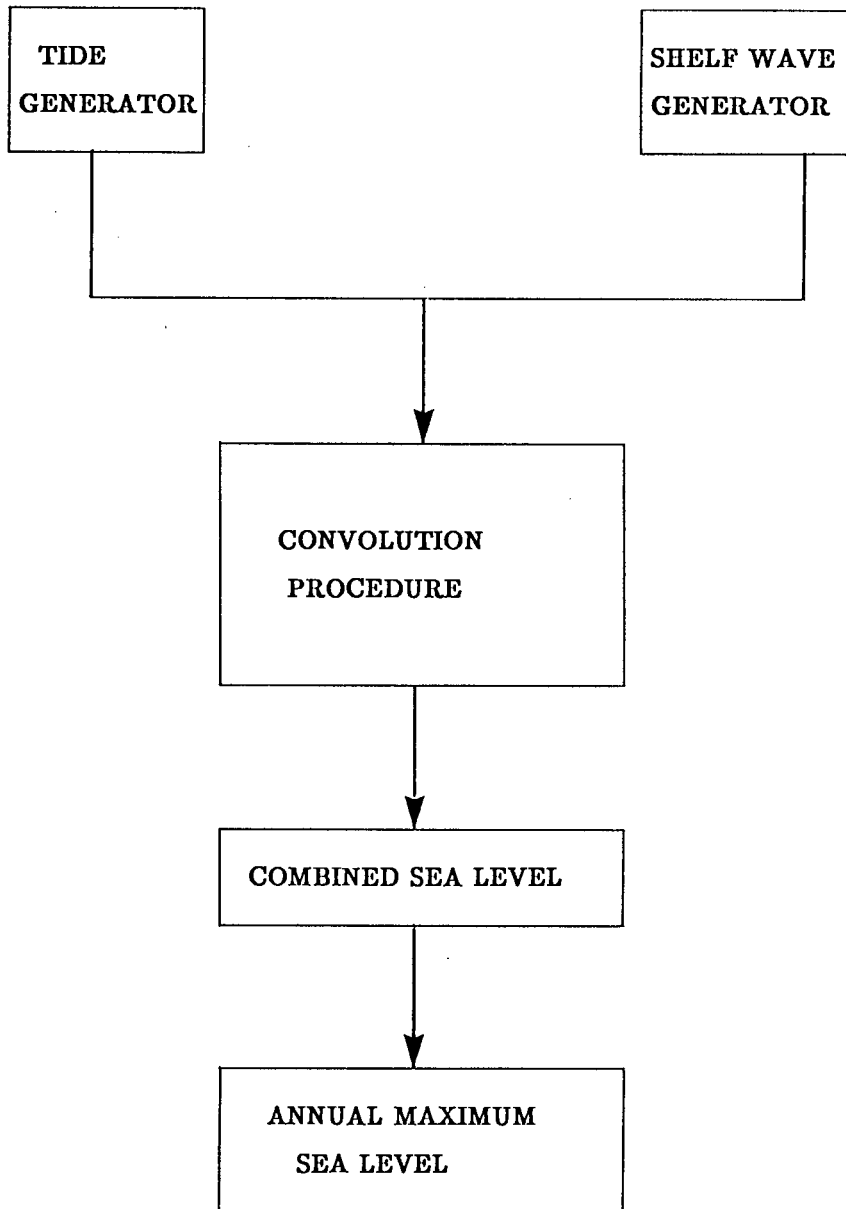
The model used to combine astronomical tides and shelf waves is illustrated in figure 7.3.1.1. The convolution model, in this instance, uses linear superposition to combine hourly values for tide and shelf waves in order to define the expected sea level. A sorting routine selects the maximum annual sea level and its associated tidal and shelf wave components. The model is run over a simulated period of 100 years. This is defined as one complete run. The annual maximum sea level results obtained from each run are sorted in ascending order and then averaged over the total number of runs carried out. The number of runs required is a function of the number of components (ie : shelf waves; edge waves; wind waves etc) which are combined in the model. The model can be run any number of times until an acceptable degree of convergence in the results is obtained. In the case of these two components (tide + shelf waves) it was found that 5 runs, 500 years of simulated data, were adequate for obtaining convergence.

In order to assess validity of the results obtained, the convolution model can be compared with results obtained using an extreme type I distribution based on observed annual maximum sea levels. The level of confidence in the predicted values is largely dependent on the available data set. The validation of this model was carried out using eighteen years of observed sea level data from Simons Bay (1970-1988). Simons Bay was selected as this port has historically not been notably responsive to long period edge waves and thus may be considered as reflecting primarily tide and shelf wave combinations. Figure 7.3.1.2 illustrates these results. The results for the model are given in figure 7.3.1.3. A comparison between both observed and predicted values is given in table 7.3.1.1..

FIGURE 7.3.1.1.

FLOW DIAGRAM

ASTRONOMICAL TIDE PLUS SHELF WAVES



MODEL versus OBSERVED DATA

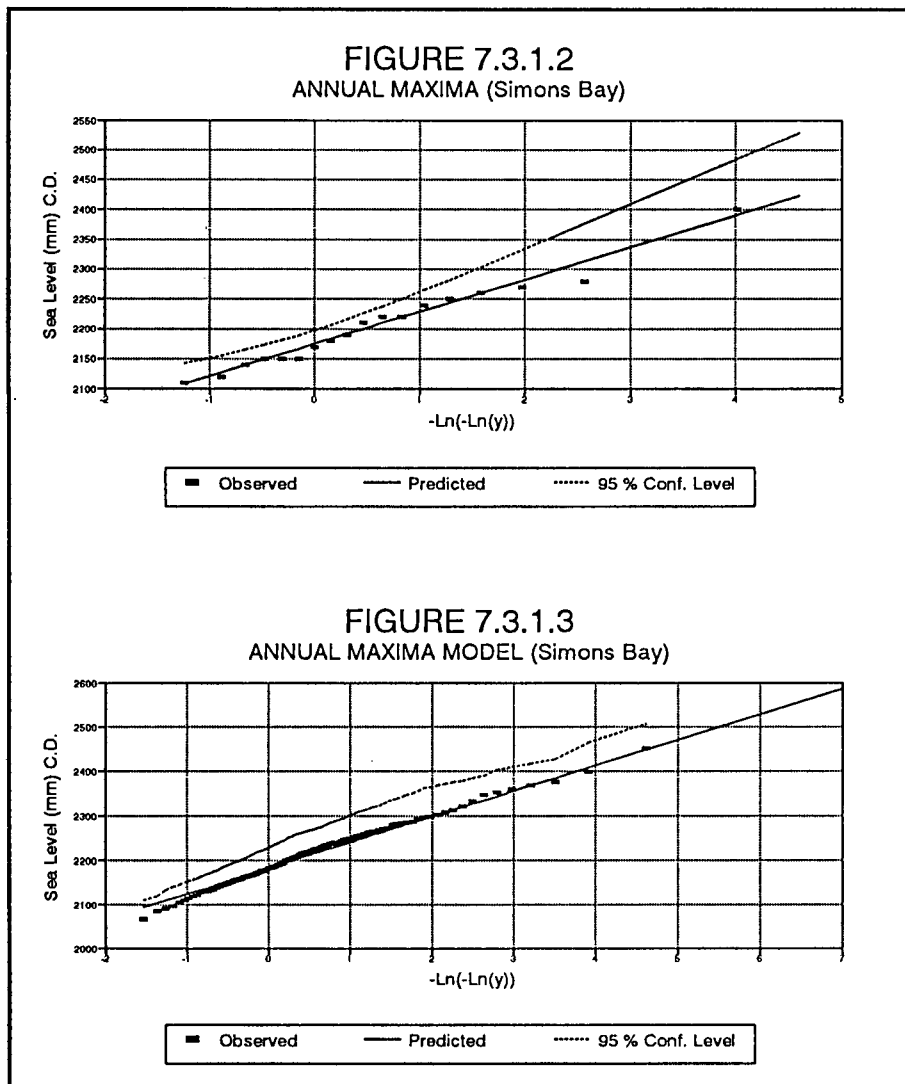


TABLE 7.3.1.1.

GUMBEL vs MODEL PREDICTIONS

RETURN PERIODS	GUMBEL		MODEL	
YEARS	PREDICTIONS(mm)		PREDICTIONS(mm)	
	+(95%)		+(95%)	
5	2256	(42)	2270	(66)
10	2296	(57)	2313	(63)
20	2335	(71)	2355	(55)
25	2348	(76)	2368	(50)
50	2386	(91)	2408	(57)
100	2423	(106)	2448	(58)

A chi square goodness of fit test confirms that the comparison between these results exceeds a level of significance of 5% thus indicating a significant level of fit. The average difference between these results is in the order of 21mm which is within acceptable limits. The variance appears to increase with larger return periods, which is expected. In most engineering applications it is important to associate some form of confidence level with the predicted values. Rossouw (1989) describes a one sided upper limit value method, proposed by Carter and Challenor (1986), where the confidence interval is assumed to have a normal distribution and the data an extreme type I distribution. The level of confidence relates to the size and variance of the the original data set. For the Gumbel method this will relate to N= 18. In the case of the model, N relates to the number of events obtained from the threshold analysis for shelf waves (N = 609). The model is rerun using the 95% upper limit distribution for event magnitude. The resulting combinations are listed in table 7.3.1.1. indicating the 95% upper limit.

The larger number of events used by the model to simulate the extreme events would appear to narrow the 95% one sided confidence limit. It would appear from these

results that the model is capable of estimating extreme values with a smaller degree of variance than the Gumbel method for a combination of shelf waves and astronomical tide.

7.3.2 *Astronomical Tide Plus Shelf Wave Plus Wind Waves*

The combination of tide, shelf waves and wind waves was undertaken using Mossel Bay data for the period 1980–84. The appropriate convolution procedure is illustrated in figure 7.3.2.1. An analysis of joint wave height/sea level recordings from the deep sea waverider and tide gauge at Mossel Bay indicates that these two processes are statistically independent. The model replicates this characteristic by generating independent wave height, shelf wave and tidal values. Figure 7.3.2.2.(a) illustrates the observed relationship between sea level and wind wave height for 1980 to 1984. Sea level values have been sorted in descending order. It is clear that no consistent relationship exists between these two measurements. These values can be compared with the results obtained for the model for several runs for the period 1980 to 1984 (see figure 7.3.2.2.(b)). It can be seen that the general characteristics of the two time series remain the same. The values have been sorted according to descending values of sea level. The wave height measurements can be seen, in both cases to follow a random distribution in relation to sea level.

A further comparison which was carried out considered the combined effect of sea level and deep sea wave height. If an empirical relationship between deep sea wave height and induced shoreline water level (wave setup) is accepted as $0.1 \times H_{mo}$ (deep water) then it is possible to compare the observed shoreline water level and predicted shoreline water levels for events during the period 1980 to 1984. Figure 7.3.2.3. (a) and (b) illustrate the observed and predicted values respectively. It can be seen that the two graphs have similar overall characteristics. It should be noted that the model values are representative of only one run. It has been shown in section 7.2. that it is possible to generate hourly tidal, shelf wave and wind wave values. From the data presented, in this section, it can be seen that it is possible to combine statistically independent tides, shelf waves and wind waves which realistically simulate event scale observations.

FIGURE 7.3.2.1.

FLOW DIAGRAM

ASTRONOMICAL TIDE PLUS SHELF WAVE PLUS WIND WAVES

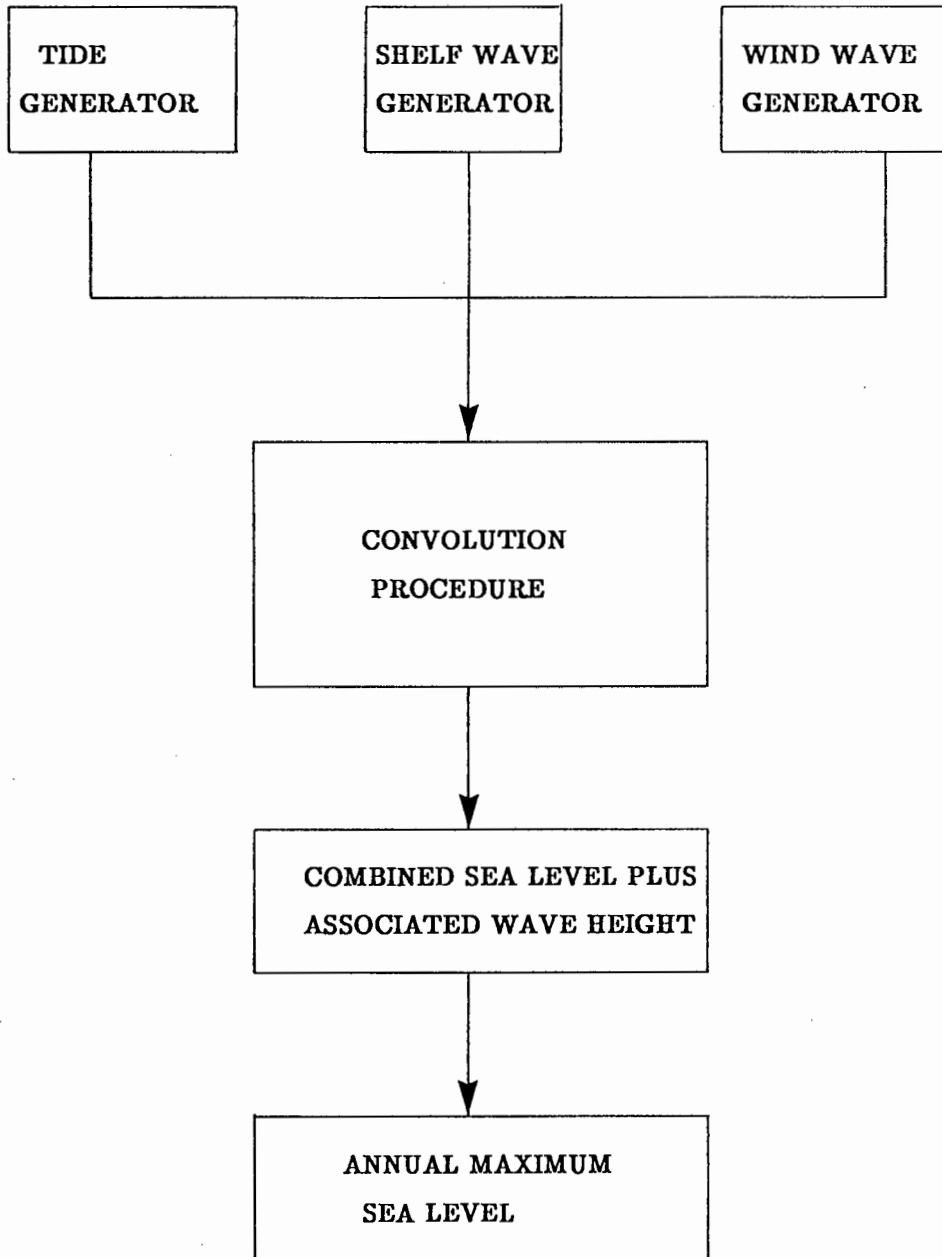


FIGURE 7.3.2.2 (a)
OBS Sea Level vs Hmo (MB 1980-84)

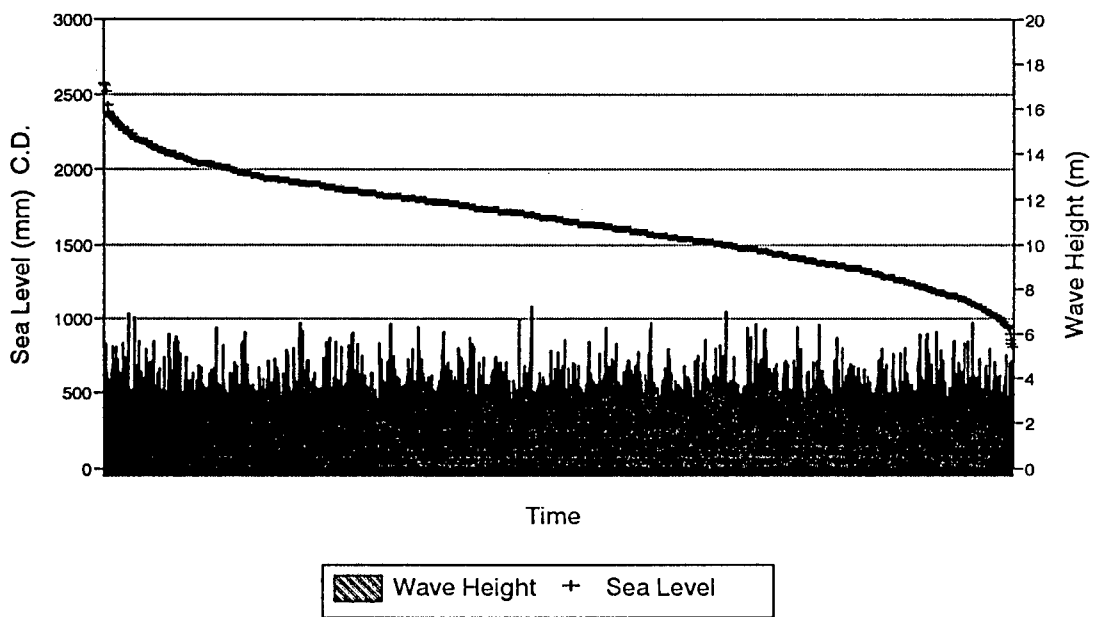


FIGURE 7.3.2.2 (b)
MODEL Sea Level vs Hmo MB

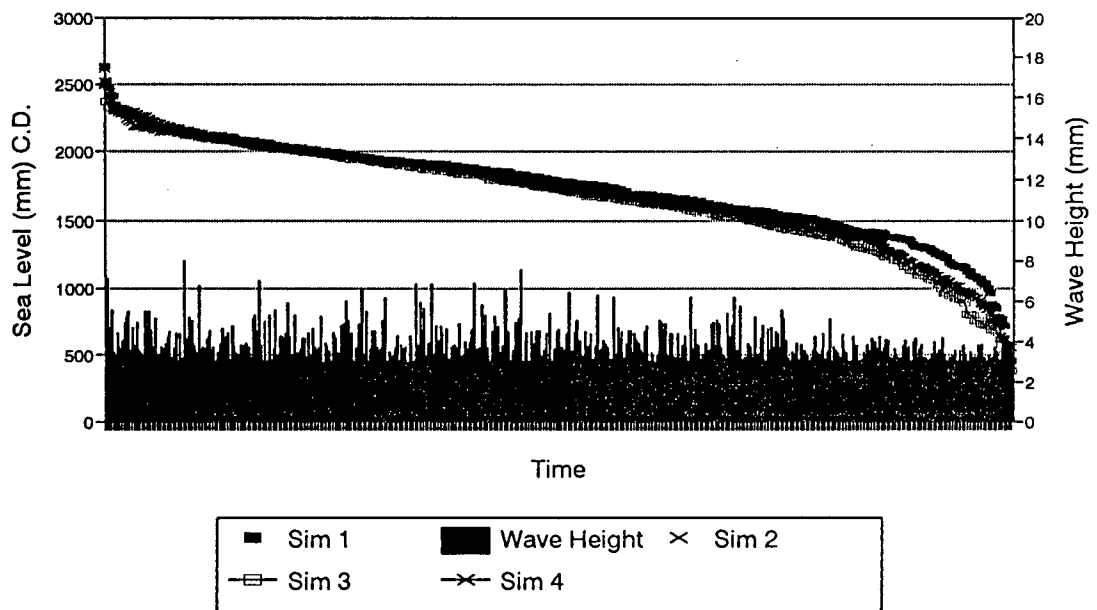


FIGURE 7.3.2.3 (a)
OBS Sea Level + 0.1Hmo (MB 1980-84)

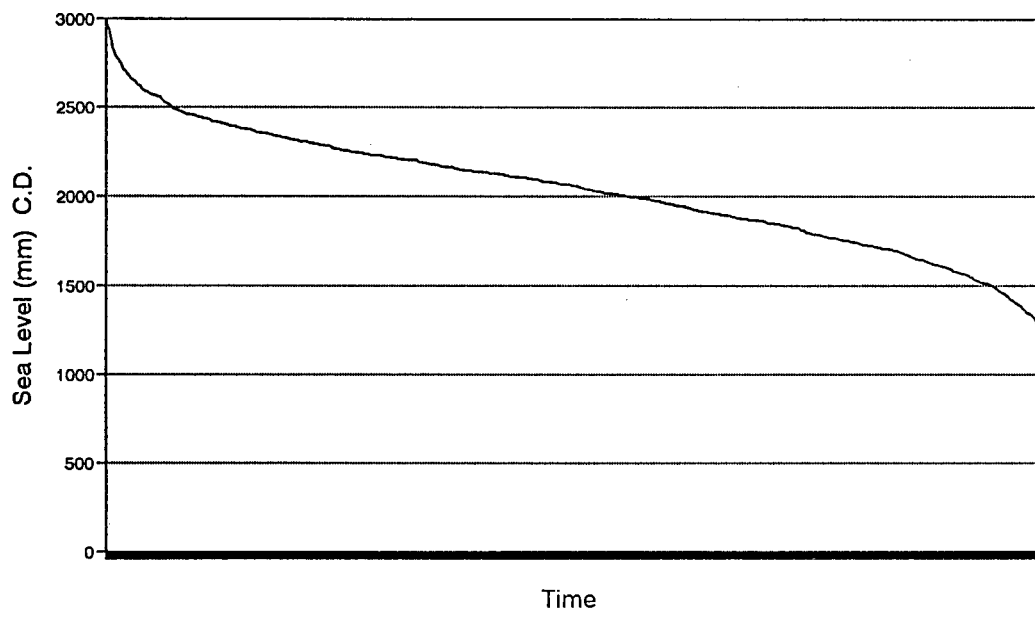
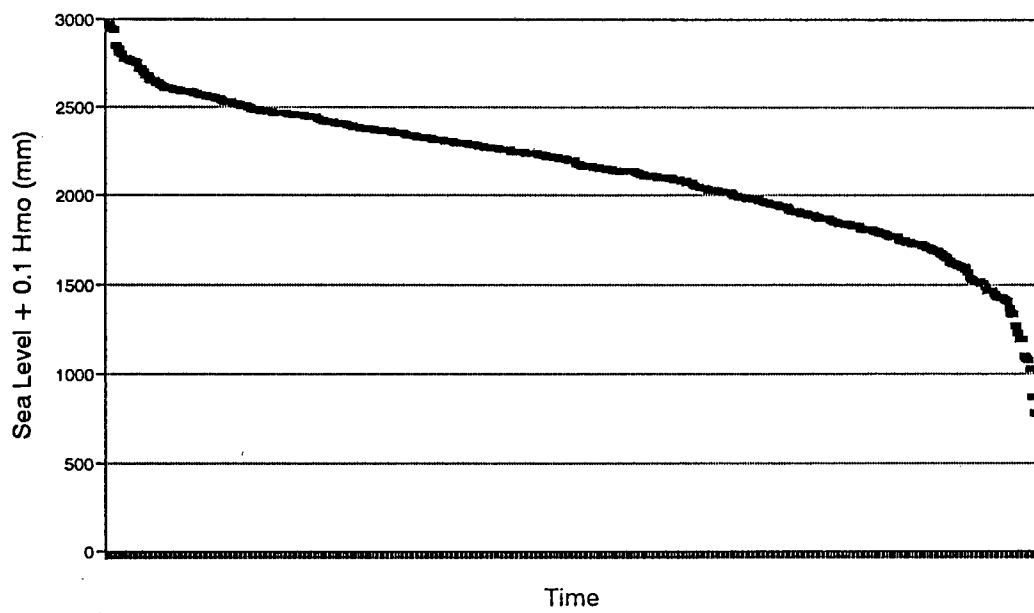


FIGURE 7.3.2.3 (b)
MODEL Sea Level + 0.1Hmo



7.3.3. *Tide Plus Shelf Waves Plus Edge Waves*

The combination of shelf waves, tide and edge waves was undertaken using Mossel Bay and Port Nolloth data. The different processes were combined on an hourly basis for a simulation run of 500 years. Figure 7.3.3.1. illustrates the basic model flow chart. The results of the run are summarized in figure 7.3.3.2. It would appear from these results that the inclusion of edge waves, as understood and modelled at this point in time, has a relatively small impact on the overall design sea level. One may intuitively reason that the nature of edge waves, notably fairly infrequent short duration events of moderate magnitude, implies that they will only have minor influence on design sea level. Table 7.3.3.1. illustrates the difference between the model run including and excluding edge waves. The results differ most notably in the tail values. This can be attributed to the fact that the combination of moderate size events define the extreme values. Hence relatively small edge waves superimposed on a moderate shelf wave event and spring tide could represent the extreme condition. It can be seen from this section that the model is capable of simulating the effects of edge waves. Although it is not possible to compare the results with field observations for Mossel Bay it can be shown that there is a distinct difference between the model output with and without the edge wave subroutine.

TABLE 7.3.3.1.

MODEL PREDICTIONS

RETURN PERIODS (yrs)	MODEL PREDICTIONS including edge waves (mm)	MODEL PREDICTIONS excluding edge waves (mm)	RESIDUAL (mm)
5	2631	2635	- 4
10	2696	2687	9
25	2777	2753	24
50	2860	2788	72
100	2915	2847	68

FIGURE 7.3.3.1.

FLOW DIAGRAM
TIDE PLUS SHELF WAVE PLUS EDGE WAVES

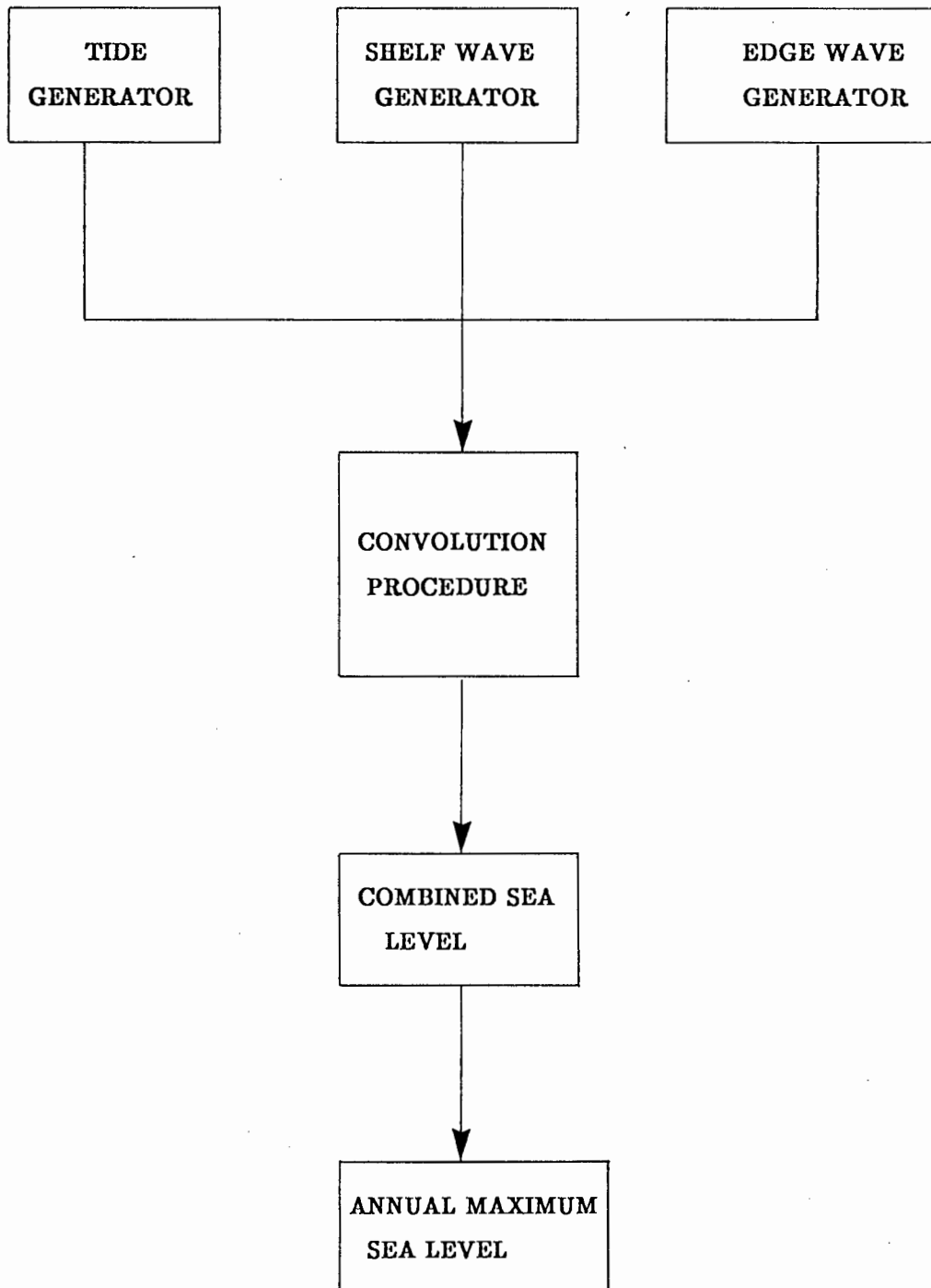
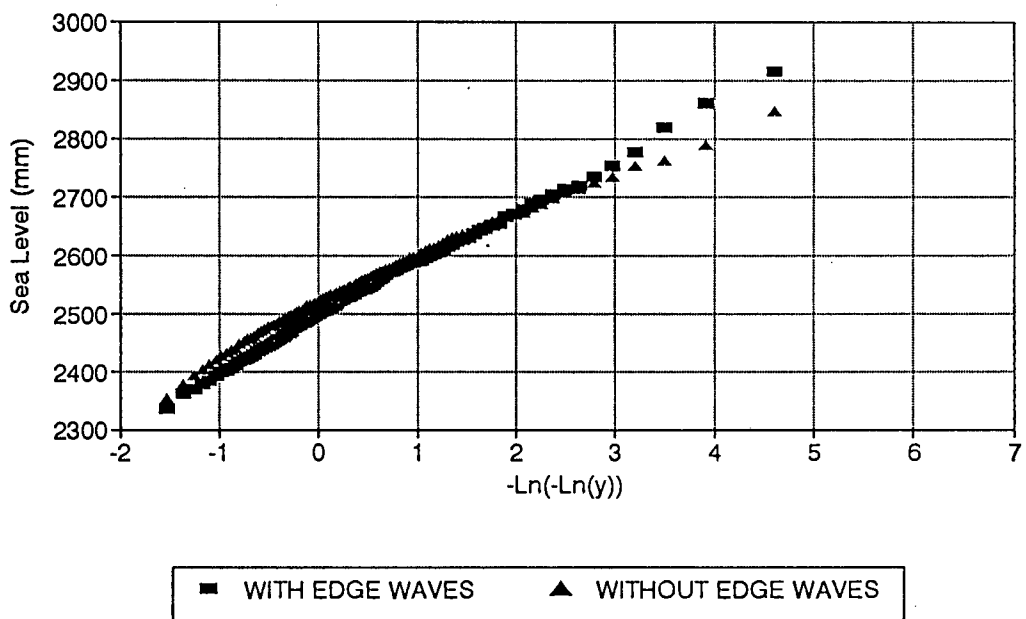


FIGURE 7.3.3.2
INFLUENCE OF EDGE WAVES ON MODEL



Having established that the wave subroutine does indeed influence the model results, it was possible to compare thirty three years of observed annual maximum sea level values for Port Nolloth, with the model prediction. The assumption was made that the Port Nolloth record includes shelf waves and edge waves. The observed annual maximum values were fitted to an extreme type I distribution and extrapolated to estimate the extreme design levels. The model was run over a 1000 year period for the data analyzed in chapter 6 for Port Nolloth. The results for both runs are summarized in Table 7.3.3.2.

TABLE 7.3.3.2.

MODEL vs GUMBEL PREDICTIONS

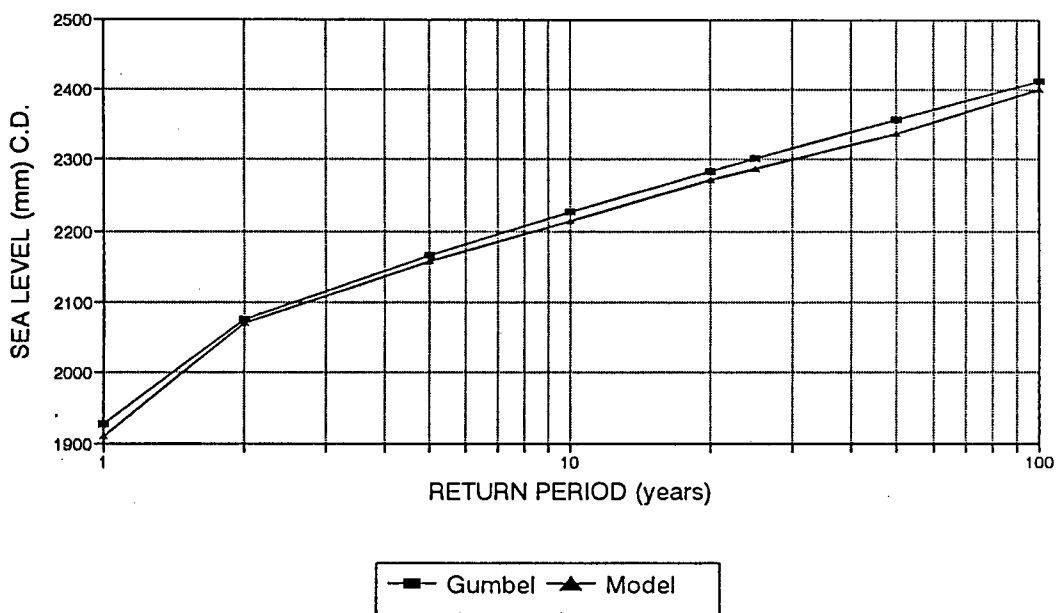
RETURN PERIODS	GUMBEL	MODEL	RESIDUAL
(yrs)	PREDICTIONS	PREDICTIONS	
	(mm)	(mm)	(mm)
5	2166	2155	11
10	2226	2213	13
25	2301	2287	14
50	2357	2336	21
100	2412	2400	12

A chi-square fit of the distributions exceeds a level of 0.05, indicating a significant level of correspondence. The largest difference between the Gumbel and Model results is 21 mm which is within acceptable limits. A graphical illustration of these results is given in figure 7.3.3.3. From this assessment it can be seen that the model performs well in comparison with the Gumbel approach.

7.3.4. Discussion

It has been shown that the model is capable of realistically combining statistically independent processes in order to obtain a new joint probability distribution for sea

FIGURE 7.3.3.3
GUMBEL vs MODEL PREDICTIONS



levels. Comparisons have been made between available observed data for sea level and predicted information. In all three cases, the model would appear to produce satisfactory results. In the next section a comparison is made between the conventional approach to combining sea level information and the results given by the model.

7.4. MODEL VERSUS CONVENTIONAL APPROACH

It is deemed useful to compare the model results with accepted methods currently used in engineering practice. Numerous approaches are used to evaluate the combined probability of exceedence of processes influencing sea level. The Gumbel method is most commonly used to assess annual maximum values. However, in this instance a significant number of years of data are required to obtain a reasonable level of confidence in the results. Furthermore, the extrapolation of multiple process statistics, in itself, introduces more uncertainty. An alternative approach has been to assess each individual component affecting sea level independently, then using various extreme value distributions, determine their respective return periods and finally combine these values to estimate the extreme results. If any form of dependency exists between the processes then the equivalent exceedence levels are combined (ie : 1/100 event = 1/100 edge wave + 1/100 shelf wave). If the events are considered as statistically independent then the probability of exceedence of the levels is multiplied to determine the extreme event (ie: 1/100 event = 1/10 edge wave + 1/10 shelf wave). Other approaches which include the effect of event duration and rate occurrence have also been developed but will not be discussed here. The former procedure can be illustrated by the following example for Mossel Bay for statistically independent processes. The method illustrated is presented as an example of engineering practice and not on the basis that it is considered as being correct. Using standard methods it is possible to calculate the extreme return period magnitudes for tides and shelf waves such that:

TABLE 7.4.1.

CONVENTIONAL APPROACH VERSUS MODEL RESULTS

RETURN PERIOD (years)	SHELF WAVE (mm)	TIDE (mm)	SUM (mm)	MODEL VALUE (mm)
1:5	590	2380	2970	2634
1:10	600	2381	2981	2687
1:25	640	2404	3044	2753
1:50	680	2406	3086	2788
1:100	701	2415	3116	2847

It should be noted that the shelf wave and tidal values are considered as being statistically independent, therefore the 1/100 return period is calculated as :

$$\begin{aligned}
 & P_{1/10}[\text{Shelf wave}] + P_{1/10}[\text{Tide}] = P_{1/100}[\text{Sea Level}] \\
 : & P_{1/10}[\text{Shelf wave}] = 701\text{mm} \\
 & P_{1/10}[\text{Tide}] = 2415\text{mm} \\
 : & P_{1/100}[\text{Sea Level}] = 3116\text{mm}.
 \end{aligned}$$

Similarly, an assessment of the combination of three processes such as tides, shelf waves and wind waves would be assessed as :

$$P_{1/4.6}[\text{Tide}] + P_{1/4.6}[\text{Shelf wave}] + P_{1/4.6}[\text{Wind wave}] = P_{1/100}[\text{Sea Level}]$$

An additional assumption is made that wave setup is 10% of the deep sea wave height. The results are given in Table 7.4.2.

TABLE 7.4.2.

TIDES, SHELF WAVE AND WAVE SETUP

RETURN PERIOD (years)	SHELF WAVE (mm)	TIDE (mm)	WAVE SETUP (mm)	SUM (mm)	MODEL (mm)
1:100	630	2404	850	3884	3234
1:1000	701	2415	910	4026	3475

If the combination of shelf waves, tides and edge waves are considered the difference between the two approaches become more dramatic.

TABLE 7.4.3.

TIDES, SHELF WAVES AND EDGE WAVES

RETURN PERIOD (years)	SHELF WAVE (mm)	TIDE (mm)	EDGE WAVE (mm)	SUM (mm)	MODEL (mm)
1:100	630	2404	447	3481	2915
1:1000	701	2415	490	3606	3066

It can be seen that there is a significant difference between the conventional approach and the full probabilistic approach. Coastal structures which are cost sensitive to sea level will therefore be subject to over design if conventional assessment approaches for independent processes are used and therefore deemed uneconomic in instances when they could indeed be viable.

It would appear that the conventional approach over predicts the value of 1/50 and 1/100 year events. This can be attributed to the fact that this approach does not consider the time series characteristics of the process such as duration and recurrence interval. Both these parameters play an important role in determining the probability of the combined occurrence of events.

7.5. THE INFLUENCE OF RECORD DURATION

The shortage of data required for the assessment of extreme sea level conditions is a problem common to most projects. In order to assess the influence of record duration on results obtained using the model and conventional approaches, Simons Bay observations from 1970-1988 were used. The data were divided into three overlapping sets comprising a 2, 6 and 18 year record. (namely 1970-71, 1970-75, and 1970-88). The individual records were analyzed and the probability distributions inputted into the model. The model was run over a period of 100 years. This was repeated six times and the ranked results averaged. The conventional approach described in 7.4 was used to calculate the associated 1/100 year events. Table 7.5.1. summarizes the results of this exercise.

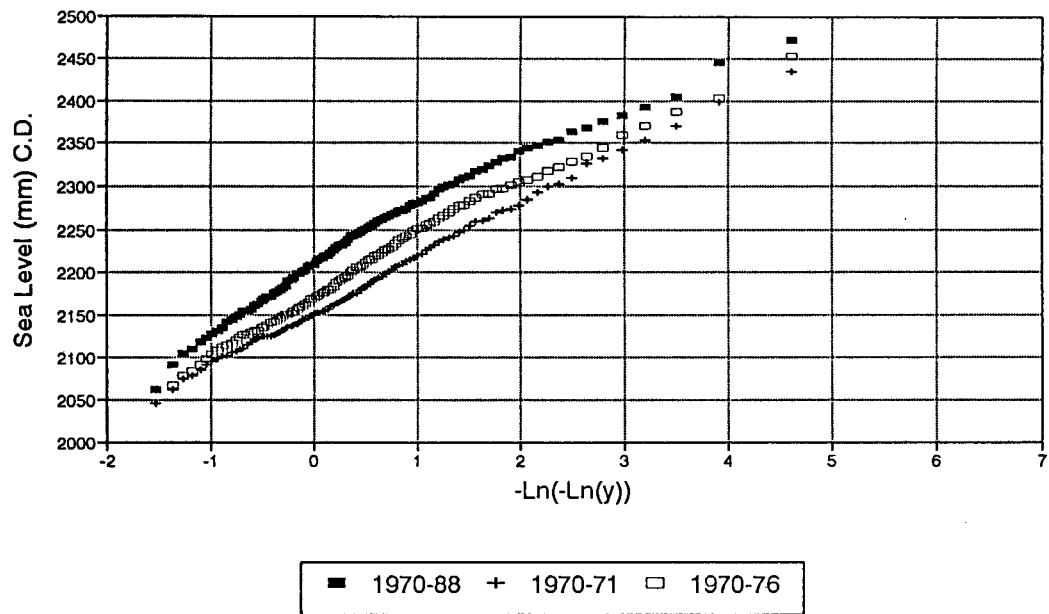
TABLE 7.5.1.

RECORD DURATION vs PREDICTED EXTREMES

RETURN PERIOD (yrs)	RECORD DURATION	MODEL VALUE (mm)	CONVENTIONAL APPROACH (mm)
1:50	2	2398	2465
1:50	6	2403	2476
1:50	18	2445	2574
1:100	2	2434	2494
1:100	6	2453	2510
1:100	18	2472	2607

It can be seen that record duration does influence the magnitude of the extreme value. Figure 7.5.1. illustrates a general downward shift in the values obtained using 1970-75 and 1970-71 data. This would appear to relate to the fact that at Simons Bay fewer events/per year were measured from 1970-71 than 1970-75 and from 1970-75 than 1970-88. The interval analysis for these periods indicates that the number of events measured for the respective records was 22 events per year, 24 events per year and 34 events per year.

FIGURE 7.5.1
Variable Record Duration



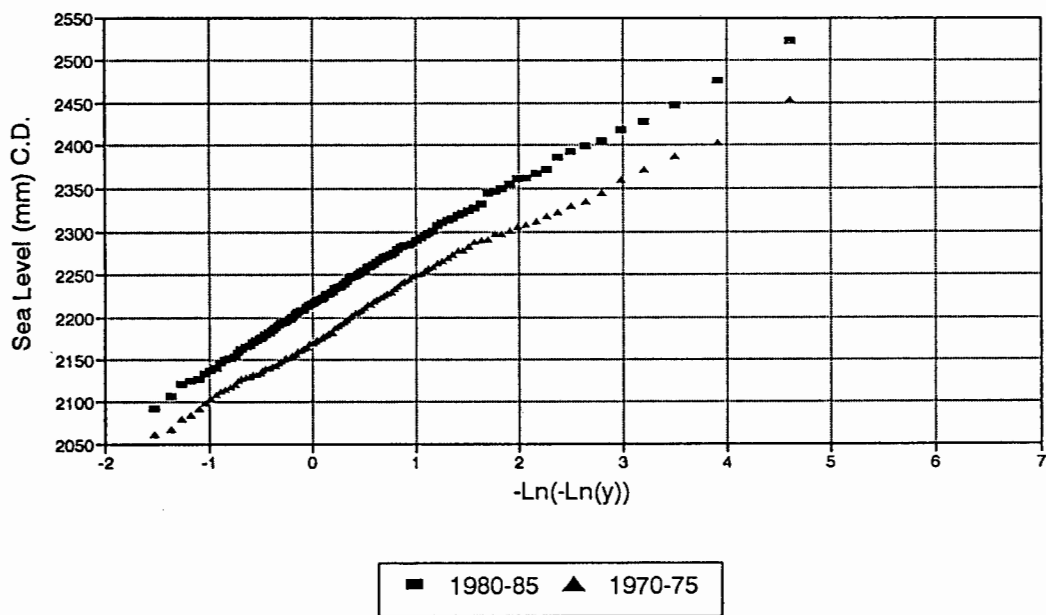
A further comparison between the conventional approach and model results indicates that whilst the difference between the 2 and 18 year record for the model is 38mm that the associated difference for the conventional method is 113mm. This would appear to confirm that the model is less sensitive to record duration than conventional extreme value predictive approaches. This can be attributed to the fact that more information is used by the model to determine annual maximum values. (ie: the actual duration, magnitude and interval distributions.)

If the records for 1970-75 and 1980-85 are compared, it can be seen (fig 7.5.2.) that the later period displayed higher extreme value predictions. The average number of events per year for that period was 45 events per year, almost double that of the former. Two possible explanations may be advanced, namely that the data set possesses a datum shift which results in more events being identified in the threshold analysis or that the occurrence of shelf waves, being related to climate, does not represent a stationary process and thus changes with time. This would still however appear to be less sensitive to the difference in the observed data sets than conventional extremes value methods.

SUMMARY

The simulation model structure has been described, the various components of the model tested and the convolution model itself verified against observed measurements. The results indicate that the model is capable of realistically simulating hourly sea levels based on various combinations of processes. The resulting sea level probability distribution can be used to assess different design conditions. A comparison between the conventional method of hazard calculations and the model indicated a tendency of the latter to over predict design levels. An assessment of the influence of record duration points towards the model being less sensitive to small data sets than the conventional approach.

FIGURE 7.5.2
Different 5 year periods



CHAPTER 8

THE APPLICATION OF THE MODEL

The stochastic simulation model developed and validated in chapter 7 is applied in this chapter with a view to assessing regional and site specific design conditions. The primary objective is to demonstrate the use of the model as a practical design tool. The purpose of the regional assessment is to gain insight into characteristics and trends with regard to design sea levels for the sub-continent. The generalized nature of these results preclude their direct application, however, they do provide useful guidelines for the conceptual or preliminary design phases. The site specific assessment considers a case study for a breakwater design. The problem relates to the recent design for the reconstruction of the Mossel Bay harbour breakwater. The design waves for the structure are depth limited, thus a direct relationship exists between wave height and the prevailing sea level. The ability of the model to provide a quantitative design condition under these circumstances is demonstrated. Each section is concluded with a general discussion of the results.

8.1. Design Sea Levels for Southern Africa

The regional assessment of design sea levels in southern Africa was aimed at evaluating sea level characteristics for different coastlines. The ports of Port Nolloth, Mossel Bay and Richards Bay on the west, south and east coasts respectively, were considered as representative of these regional areas. The model was run for all three ports with regard to the combination of astronomical tides, shelf waves, wind waves and edge waves. The results are expressed in terms of sea level relative to chart datum (CD), highest astronomical tide (HAT) and lowest astronomical tide (LAT). A number of assumptions were made with regard to the application of the model. These are listed below :

1. Design sea levels were evaluated for open coastline conditions.
2. Wind wave information related to deep water unrefracted waverider data. Shallow water wave transformation was not included.
3. Edge waves were not included for the Richards Bay model simulation.

4. Wave setup was assumed as being 10% of the unrefracted deep sea wave height.

The results given in this section should be seen within a comparative context and should not therefore be used quantitatively. A detailed study of a particular structure, as in section 8.2, would need to consider local effects, such as shoaling, refraction, diffraction, reflection and wave breaking.

The data used to undertake this study are summarized in chapter 6. This information will not be repeated in this section. Each port was assessed for 10 (100 year) simulations. These runs were sorted and averaged with only the annual maximum events being considered.

Table 8.1.1. presents the results for all three ports for typical open coastline conditions based on the assumptions made earlier. Under these assumptions it can be seen that Mossel Bay has the highest design sea levels. If these values are related to sea level above highest astronomical tide (HAT), a common design condition, it can be seen that Mossel Bay, Richards Bay and Port Nolloth vary by 800 mm, 481 mm and 647 mm respectively, for the 1/100 year condition. Figures 8.1.1.(a) and (b) illustrate the relative differences between the ports. These results should be seen as representative of a uniform idealized coastal form. Within this context the south coast should be expected to be subject to higher maximum levels than the west or east coasts. It should be noted that although the sea levels relative to chart datum, are higher for Richards Bay than Port Nolloth, the values relative to HAT indicate that the west coast would experience larger variations in sea level than the east coast.

DESIGN SEA LEVELS

S.A. PORTS

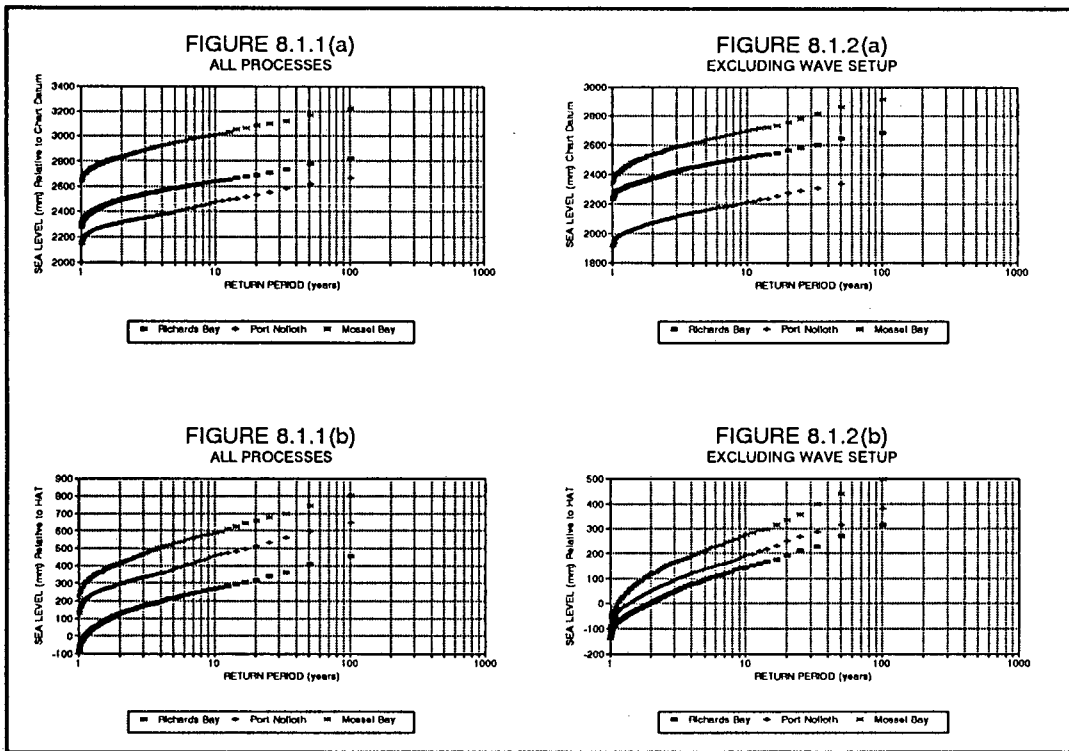


TABLE 8.1.1.

**DESIGN SEA LEVEL FOR SOUTHERN AFRICAN PORTS
COMPARATIVE ANALYSIS: ALL PROCESSES**

RETURN PERIOD (years)	RICHARDS BAY (mm)	MOSSEL BAY (mm)	PORT NOLLOTH (mm)
1	2281	2630	2144
5	2584	2945	2390
10	2641	3010	2479
25	2708	3100	2553
50	2781	3160	2618
100	2824	3220	2667

(Relative to chart datum)

If the influence of wind waves (wave setup) are removed and only tide, shelf waves and edge waves considered, then it can be seen from table 8.1.2. and figures 8.1.2(a) and (b) that Mossel Bay remains the most responsive location to the variation in sea level. The removal of wave setup means that the results can be applied more generally as wave transformation is less important for the other processes. The 1/100 year design level relative to HAT for the south, west and east coasts are 495 mm, 380 mm and 342 mm respectively. In many instances HAT is considered as an extreme design level. It is noteworthy that for the 1/100 condition the variation may be up to 500 mm larger than HAT. Possibly of more practical significance is the fact that HAT is exceeded on average every $2^{1/2}$ -3 years around the entire coastline of southern Africa.

TABLE 8.1.2.

DESIGN CONDITIONS FOR SOUTHERN AFRICAN PORTS
SEA LEVEL EXCLUDING WAVE SETUP

RETURN PERIOD (years)	RICHARDS BAY (mm)	MOSSEL BAY (mm)	PORT NOLLOTH (mm)
1	2234	2336	1910
5	2462	2631	2155
10	2515	2696	2213
25	2581	2777	2287
50	2642	2860	2336
100	2685	2915	2400

(Relative to chart datum)

A further point of interest is to assess the sea level associated with the expected maximum wave height (H_{m_0}). Table 8.1.3. and figure 8.1.3. indicate that annual maximum H_{m_0} vs associated sea level values follow no particular trend with regard to the H_{m_0} . This is to be expected as the model assumes statistical independence between H_{m_0} and all other processes. A frequency plot of these sea level values appears normally distributed (see figure 8.1.4.). From a design point of view it would appear appropriate to use an average sea level stand with the maximum H_{m_0} for deep water conditions. It should be noted that the average values for sea level correspond closely with the mean sea levels at each of the respective ports.

DESIGN WAVEHEIGHTS

S.A. PORTS

FIGURE 8.1.3
DEEP SEA SIGNIFICANT WAVEHEIGHT

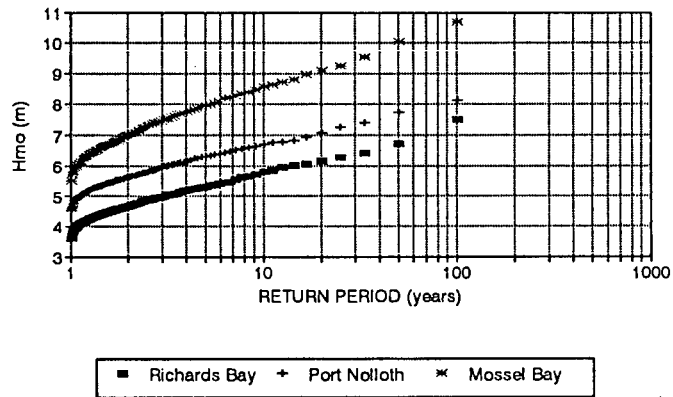


FIGURE 8.1.4
Sea Levels associated with Hmo(max)

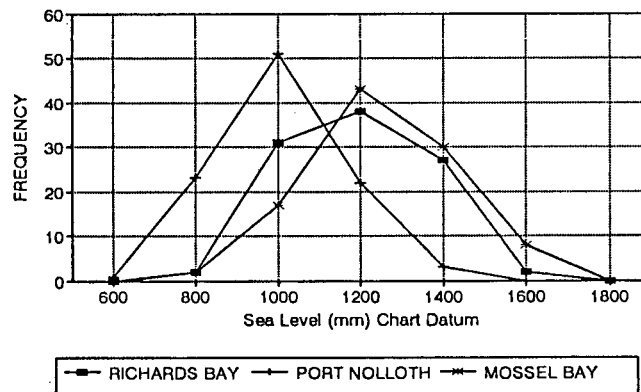


TABLE 8.1.3.

**DESIGN CONDITIONS FOR SOUTHERN AFRICAN PORTS
DESIGN SIGNIFICANT WAVE HEIGHT AND ASSOCIATED SEA LEVELS**

RETURN PERIODS (years)	RICHARDS BAY		MOSSEL BAY		PORT NOLLOTH	
	Hmo (m)	Sea level (mm)	Hmo (m)	Sea level (mm)	Hmo (m)	Sea level (mm)
1	3.62	1015	5.50	1228	4.52	1084
5	5.28	982	7.92	950	6.27	721
10	5.80	984	8.57	1189	6.71	840
25	6.25	887	9.24	1274	7.25	804
50	6.72	1289	10.04	1290	7.71	828
100	7.50	1265	10.68	1274	8.13	1058
Average :		1085		1150		904
Standard Deviation:		157		156		149

(Values relative to chart datum)

If one considers the minimum sea levels around the coastline it can be seen that the same trend as maximum design levels is obtained. The results are summarized in table 8.1.4. and figure 8.1.5 (a) and (b). The south coast is clearly subject to larger variations in minimum sea level than Port Nolloth and Richards Bay. Similarly, if these levels are related to LAT then the 1/100 design sea levels are -632 mm, -368 mm and -339mm for the south, west and east coasts, respectively. As is the case with HAT, LAT is often used as an extreme design level. These results suggest that these values may be inappropriate particularly for the south coast. An aspect of some concern, perhaps, is that water depths which relate to chart datum, will be at times up to 650 mm shallower than indicated on navigational charts. Water depths should be expected to be shallower than LAT several times a year in Mossel Bay and at least every year in Port Nolloth and Richards Bay.

DESIGN SEA LEVELS

S.A. PORTS

FIGURE 8.1.5 (a)
Minimum Sea Levels

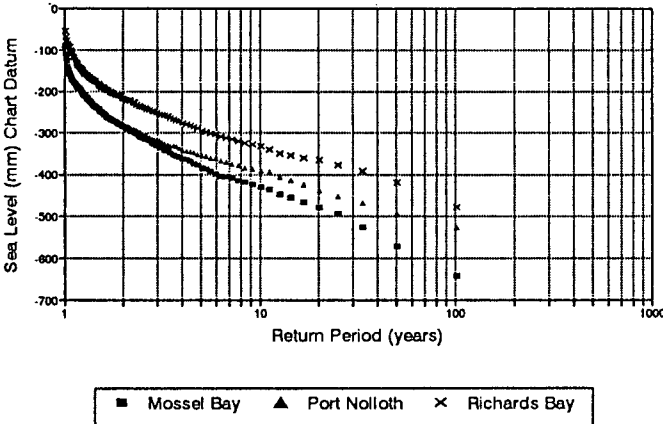


FIGURE 8.1.5 (b)
Minimum Sea Levels

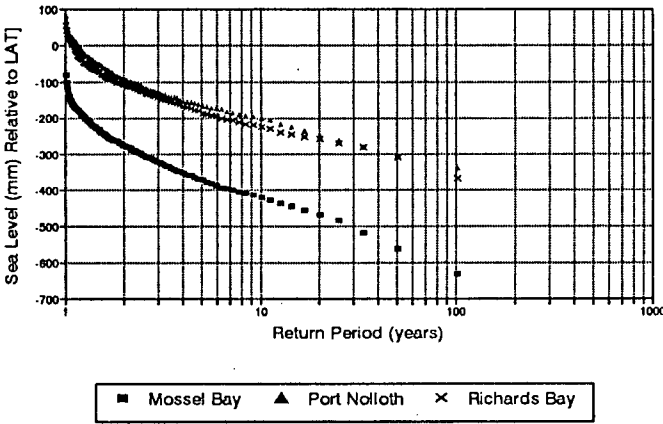


TABLE 8.1.4.

DESIGN CONDITIONS FOR SOUTHERN AFRICAN PORTS
MINIMUM SEA LEVELS

RETURN PERIOD (yrs)	RICHARDS BAY (mm)	MOSSEL BAY (mm)	PORT NOLLOTH (mm)
1	-54	-91	-111
5	-296	-383	-357
10	-334	-431	-392
25	-379	-494	-454
50	-419	-572	-495
100	-478	-642	-529

Relative to chart datum

If the ports of Mossel Bay, Port Nolloth and Richards Bay can be considered as representative of the south, west and east coasts respectively, then it can be seen, from a generalized application of the model, that the south coast is subject to the largest sea level variations. Furthermore, it would appear that the west coast is more responsive than the east coast. The reason for this would appear to relate to the driving processes and their response along the respective coastlines. It has been shown in chapter 6 that the south coast is most responsive, followed by the west and east coasts, to wind wave and shelf wave events. Maximum tidal range, on the other hand, is largest at Richards Bay (2.48 m), followed by Mossel Bay (2.43 m) and Port Nolloth (2.21 m). These differences would appear to be relatively insignificant. If the effect of edge waves is considered as being small, it can be seen that the shelf waves and wind waves represent the dominant processes affecting the regional sea level characteristics of the southern African coastline.

The differences between the regional characteristics would appear large enough to suggest the design guideline that these areas be treated independently when assessing sea levels. Spatial transfer of data between these regions is therefore not recommended.

The current practice of using LAT and HAT as representing extreme sea level in southern Africa should be discontinued. It is clear that these conditions will be exceeded on many occasions during the life of a structure. This is particularly applicable to the practice of using LAT as equivalent to chart datum. It should be expected that variations of up to 600 mm below predicted still water level will be measured at least once a year.

8.2. MOSSEL BAY HARBOUR BREAKWATER DESIGN

The existing breakwater at Mossel Bay has suffered considerable damage since its construction during the 1970's (see plate 8.2.) The CSIR has undertaken a number of studies in recent years with regard to the reconstruction of this breakwater. In this section results from the wave condition analysis, CSIR (1988), are used to compare the conventional approach to design conditions with the methodology proposed in this thesis. This exercise should be viewed as a comparative assessment of two approaches and not as a detailed assessment of the design condition. The evaluation of the Mossel Bay breakwater afforded an opportunity to demonstrate the significance of design sea level for structures subject to depth limited wave conditions as described in chapter 3. The assessment of the design conditions were undertaken for a range of situations. These may be listed as follows :

1. Design wave height and associated sea level 50 m in front of the breakwater in 4.9 m (M.S.L.) water depth. This assessment was undertaken to compare the results of the model with the design conditions presented in CSIR (1988).
2. Design wave height and associated sea level at the breakwater in 3.7 m (M.S.L.) water depth.
3. Design runoff on the structure.
4. Design setup on the structure.
5. Design drawdown and expected wave height.
6. Design drawdown at the toe of the structure.

The Hudson formula was used to compare the influence of the model results on the size primary armour units for the purpose of comparison. In order to limit the scope of the

PLATE 8.2.1.

MOSSEL BAY BREAKWATER



PLATE 8.2.2.

MOSSEL BAY BREAKWATER



PLATE 8.2.3.

MOSSEL BAY BREAKWATER



work certain simplifying assumptions were made. The information contained in CSIR (1988) was considered as representative of the true design conditions. These assumptions can be summarized as follows :

1. The design wave condition is limited by the depth in front of the breakwater.
2. One design condition was represented by the wave height 50 metres seaward of the breakwater in 6 m water depth at mean high water spring (MHWS), where MHWS is equal to 1.1 m above mean sea level (M.S.L.).
3. Refraction and shoaling coefficients were calculated using the Wale Refraction Model.
4. The sectors considered in terms of the detailed refraction study, namely, east and south south west were representative of the design condition.
5. Easterly waves were considered as being 65% of the magnitude of SSW wave heights.
6. The maximum possible combined refraction/shoaling coefficient used was 1.7.
7. A typical breakwater section is given in figure 8.2.

The results of the CSIR (1988) study are summarized in table 8.2.1.

TABLE 8.2.1.

DESIGN WAVE CONDITIONS

DEEP SEA DIRECTION	T _p	MOST OBLIQUE ANGLE OF INCIDENCE	H _{m0} FOR VARIOUS RETURN PERIODS			
			1	10	50	100
E	9.0	70	5.1	5.1	5.1	5.1
E	15.5	104	5.4	5.4	5.4	5.4
SSW	13.5	110	2.5	3.1	3.5	3.7
SSW	15.5	108	1.9	2.3	2.6	2.8

5,4 t Dolosse used for damage calculation (238/25m)



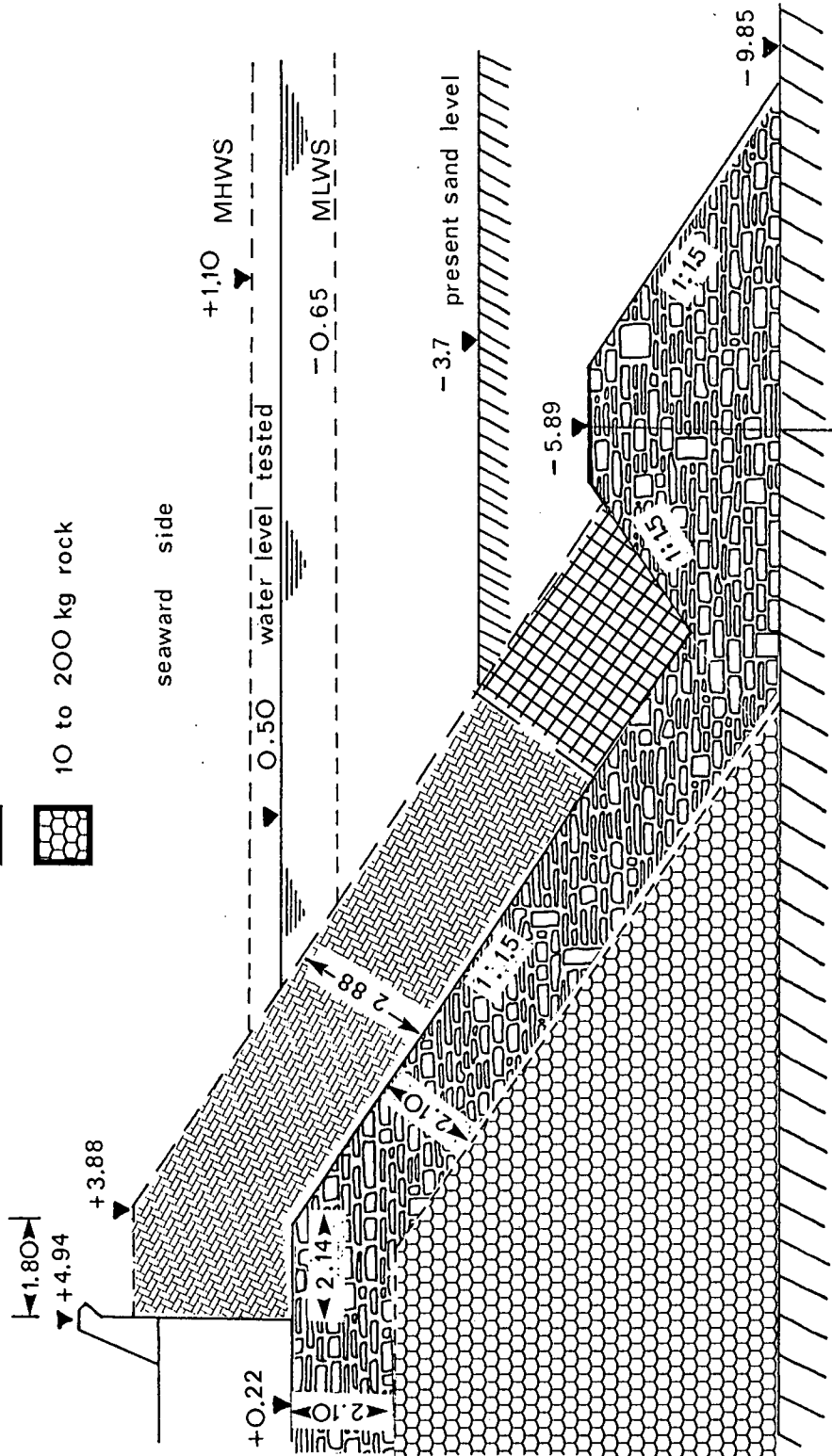
5,4 t Dolosse below sand level



1000 to 3000 kg rock



10 to 200 kg rock



SCALE 1:150

CSIR
EMATEK

REPAIR OF MOSSEL BAY BREAKWATER
CROSS-SECTION USED FOR
CALIBRATION TEST
EXISTING 5,4 t DOLOSSE

FIGURE 8.2.

These results were seen as the primary input conditions to the detailed model study. The application of the model was carried out using the following input parameters :

1. Sea level at Mossel Bay was determined by tide, shelf waves and edge waves using the data analyzed earlier in this study.
2. Due to the lack of refraction study information, only two directional sectors were used. These were divided into NNE to SSE and S to NNW. The seasonal division, for deep sea conditions, was obtained from VOS data for sector 41 as summarized by Rossouw (1984).
3. The seasonal occurrence for summer and winter were as follows :

(i)	Summer	-	easterly	31.23%
		-	southerly	68.77%
(ii)	Winter	-	easterly	23.71%
		-	southerly	76.29%

4. Wave height magnitude was modified as follows :

easterly waves	-	$0.65 * H_{mo} * 1.70$
southerly waves	-	$1.00 * H_{mo} * 0.31$

5. An average wave period of 13 seconds was assumed to be applicable.
6. The relationship between $H_b/d_b = 0.90$ was used throughout.

The programme was designed to select the largest annual combination of nearshore wave height and sea level values. The model was run for 10 simulations of 100 years each. These results were sorted according to the maximum combined response and analyzed. It should be noted that in cases where the wave height exceeded the water depth at a point 50 metres from the wall, it was assumed that the design wave height would be H_b at this position. The assumption here is that during a particular storm, several waves smaller than H_{mo} would be experienced in the area of interest. These would then represent the design condition.

The results from the simulation model are summarized in table 8.2.2. and figure 8.2.1. as follows :

TABLE 8.2.2.

DESIGN WAVE AND SEA LEVEL CONDITIONS

RETURN PERIOD	NEARSHORE WAVE HEIGHT (mm)	ASSOCIATED SEA LEVEL		WATER DEPTH M.S.L. (mm)
		chart datum (mm)	M.S.L (mm)	
1	4.36	1623	723	5.62
5	5.35	2003	1103	6.00
10	5.45	2095	1195	6.10
25	5.58	2208	1308	6.21
50	5.64	2272	1372	6.27
100	5.68	2317	1417	6.32

Water depth at = 4.9 m mean sea level.

The significance of these results becomes apparent when one considers the sizing of armour units. If the Hudson formula is used to assess the stability of typical concrete units then :

$$W = \frac{g \rho_s (\gamma_b d_b)^3}{K_d \left(\frac{\rho_s}{\rho_w} - 1 \right)^3 \cot \alpha}$$

(as defined in chapter 3)

If the 1/100 return period is used as the design condition then from tables 8.2.1. and 8.2.2. it can be seen that unit sizes will vary between 5.7 tonnes for CSIR (1988) and 6.6 tonnes for the model results. The difference between these results can be attributed to the fact that sea level is considered as a stochastic variable in the model whilst the

MOSSEL BAY BREAKWATER DESIGN CONDITIONS

FIGURE 8.2.1
DESIGN WAVE HEIGHT

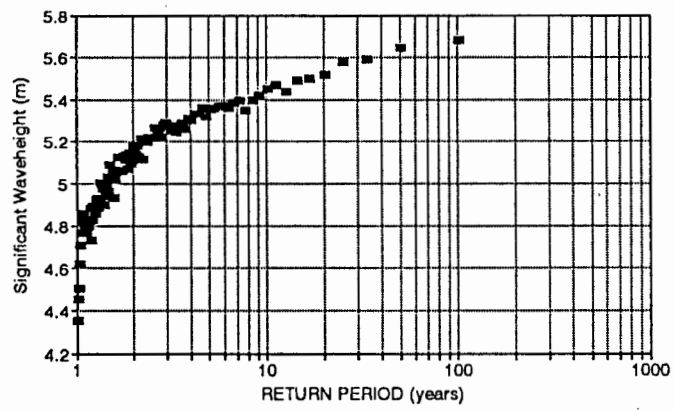
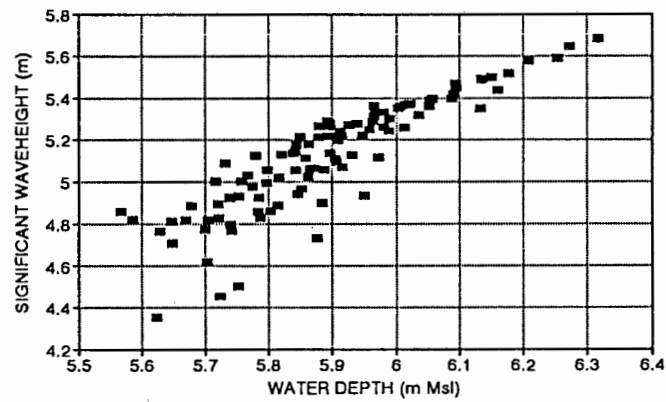


FIGURE 8.2.2
WAVEHEIGHT versus WATERDEPTH



Water Depth = 4.9 m MSL

previous work considered it to be constant. Of some interest perhaps is that the 1/1 year condition for the model is less than that given in table 8.2.1. This difference is most likely due to the model using the extreme type I distribution as opposed to the log normal distribution used in CSIR (1988). If wave height is plotted against associated sea level (see figure 8.2.2.) then it can be seen that for the more frequent condition (smaller return period) the relationship is almost independent. As the condition becomes less frequent so the dependency between H_{m_0} and associated sea level becomes more important. This can be attributed to the depth limited conditions at this point.

If the former example is modified, it is possible to assess the design conditions immediately in front of the breakwater. The assumption made here, using linear wave theory, is that the waves increase in height, due to shoaling, by 4% from 4.9 metres m.s.l. to 3.7 metres m.s.l. The results of a 1000 year simulation are summarized in table 8.2.3. and illustrated in figures 8.2.3. and 8.2.3.(a).

TABLE 8.2.3.

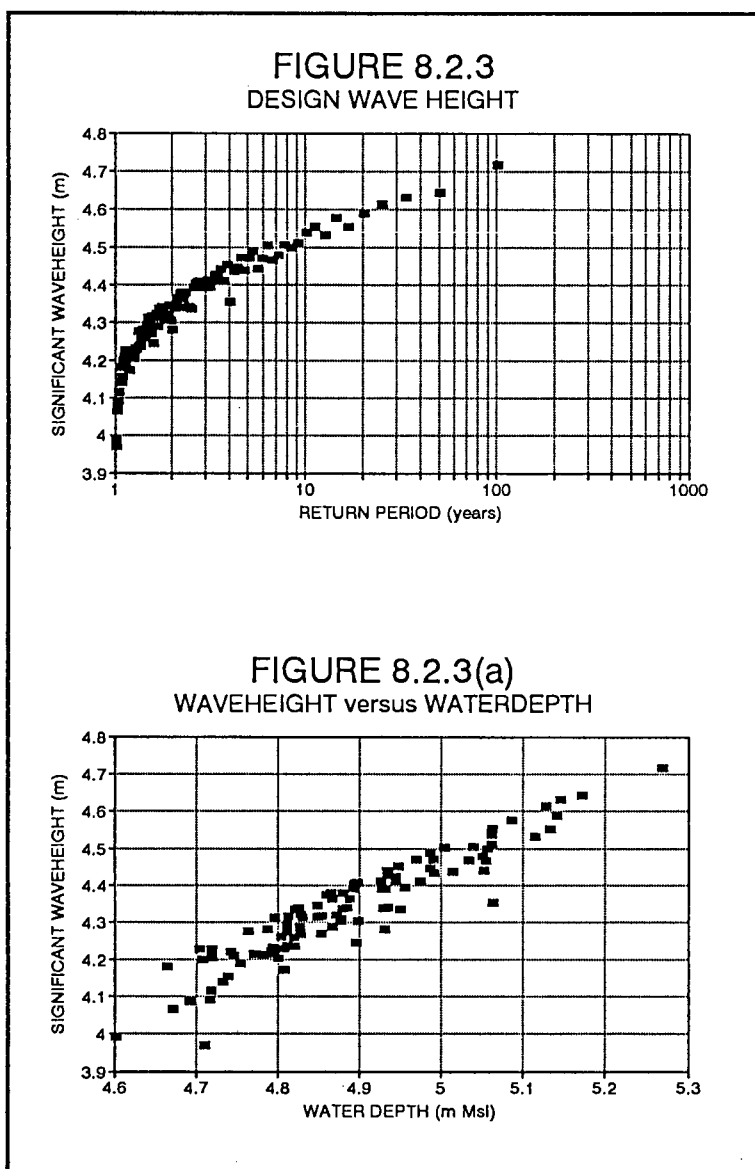
DESIGN WAVE AND SEA LEVEL CONDITIONS

RETURN PERIOD	NEARSHORE WAVE HEIGHT (m)	ASSOCIATED SEA LEVEL		WATER DEPTH M.S.L. (mm)
		C.D. (mm)	M.S.L. (mm)	
1	3.99	1802	902	4.60
5	4.45	2200	1300	5.00
10	4.54	2262	1362	5.06
25	4.61	2328	1428	5.13
50	4.64	2372	1472	5.17
100	4.72	2470	1570	5.27

Water depth at breakwater = 3.7 metres m.s.l.

If these results are compared with those in table 8.2.2., it can be seen that whilst the wave height decreases due to the depth limited conditions, the 1/100 year associated

MOSSEL BAY BREAKWATER DESIGN CONDITIONS



Water Depth = 3.7 m MSL

sea level condition increases by approximately 150 mm. This results in a proportionally larger wave height at the breakwater as compared with 4.9 metres water depth. The explanation for this is that at shallow water depths there exist a greater number of event combinations which increases the probability of larger events occurring concurrently.

If the armour unit size is compared, as in the previous example, then using the conventional approach, the design wave height would be determined by the water depth at mean high water spring ($3.7 + 1.1 = 4.8$ m). If $H_b/d_b = 0.90$, then $H_b = 4.32$ m. If the Hudson formula is used to differentiate between these two design conditions then :

$$\begin{aligned} \text{Difference} &= \frac{(H_b \text{ model})^3}{(H_b \text{ conventional})^3} \\ &= \frac{105.15}{80.62} = 1.30 \end{aligned}$$

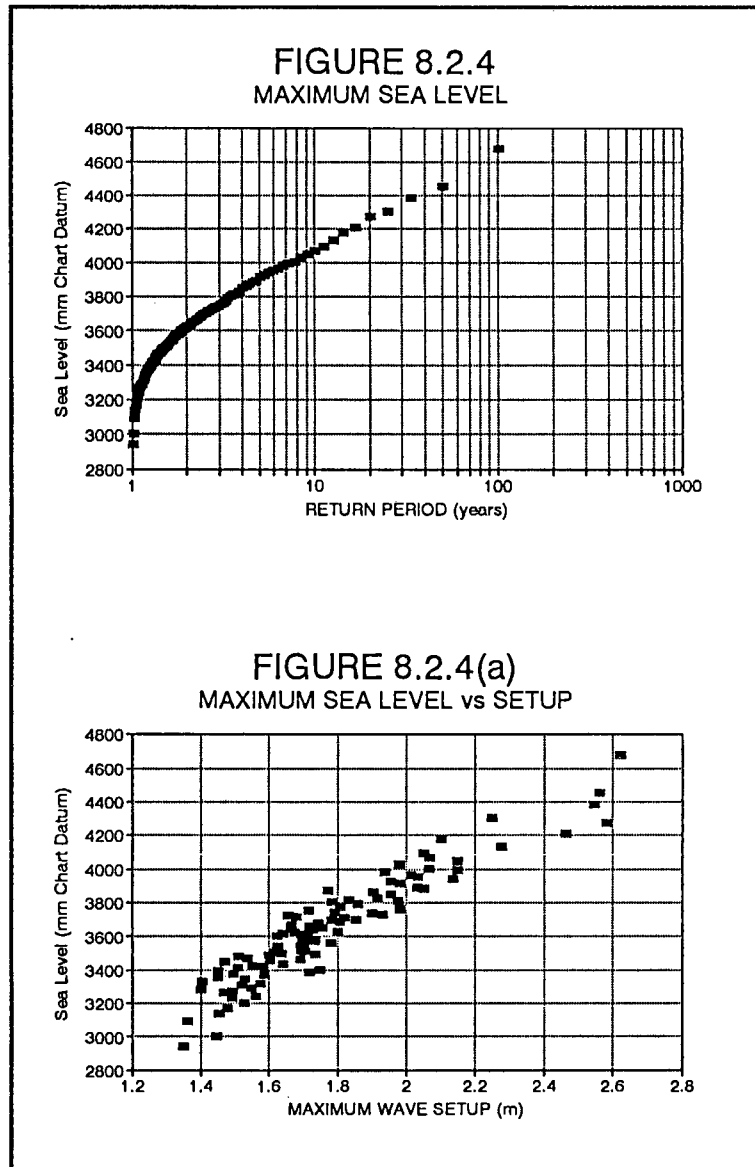
This represents a 30% increase in the unit size which could be considered as significant, thus illustrating the importance of considering sea level as a stochastic variable.

In order to assess the maximum sea level at the breakwater as a result of wave setup it was assumed from Bruun (1985) that $S = 0.3 * H_b$

where S = maximum wave setup
 H_b = breaking wave height (m)

This represents the maximum setup level. The results are summarized in table 8.2.4. and figure 8.2.4.

MOSSEL BAY BREAKWATER DESIGN CONDITIONS



Water Depth = 3.7 m MSL

TABLE 8.2.4.
MAXIMUM SEA LEVEL AT BREAKWATER

RETURN PERIOD (yrs)	MAXIMUM SEA LEVEL		WATER DEPTH (m)	ASSOCIATED MAX SETUP (m)
	C.D. (mm)	M.S.L. (mm)		
1	2943	2043	5.74	1.350
5	3000	2100	5.80	2.000
10	4068	3168	6.87	2.068
25	4306	3406	7.11	2.248
50	4452	3552	7.25	2.562
100	4679	3779	7.48	2.621

Water depth at breakwater = 3.7 m m.s.l.

Using the results obtained from table 8.2.3. and making the simplifying assumption that wave runup on steep slopes can be represented by $R = Hm_0$ it is possible to get some indication of the probability of occurrence of specific runup levels. The results are given in table 8.2.4.(a).

TABLE 8.2.4.(a)
MAXIMUM RUNUP LEVELS ON BREAKWATER

RETURN PERIOD (YEARS)	MAXIMUM RUNUP	
	CHART DATUM (mm)	MEAN SEA LEVEL (mm)
1	5792	4.892
5	6650	5.750
10	6802	5.902
25	6938	6.038
50	7012	6.290

The crest elevation of the breakwater wave wall is located at +4.98 metres m.s.l. It can be seen from table 8.2.4. that the Mossel Bay breakwater can be expected to be overtopped at frequent intervals and that significant overtopping will occur under the 1/50 and 1/100 design condition. These results could be improved by providing a more comprehensive formulation for wave runup for the purposes of a quantitative assessment.

The assessment of minimum design sea levels, or maximum drawdown, is required in order to determine foundation stability and the extent of the primary armour protection at the toe. The primary failure modes, in this instance, are scouring and subsequent undermining of the breakwater slope. The model was run using minimum shelf wave and tide combinations over a period of 1000 simulated years. The results are listed in table 8.2.5. and illustrated in figure 8.2.5.

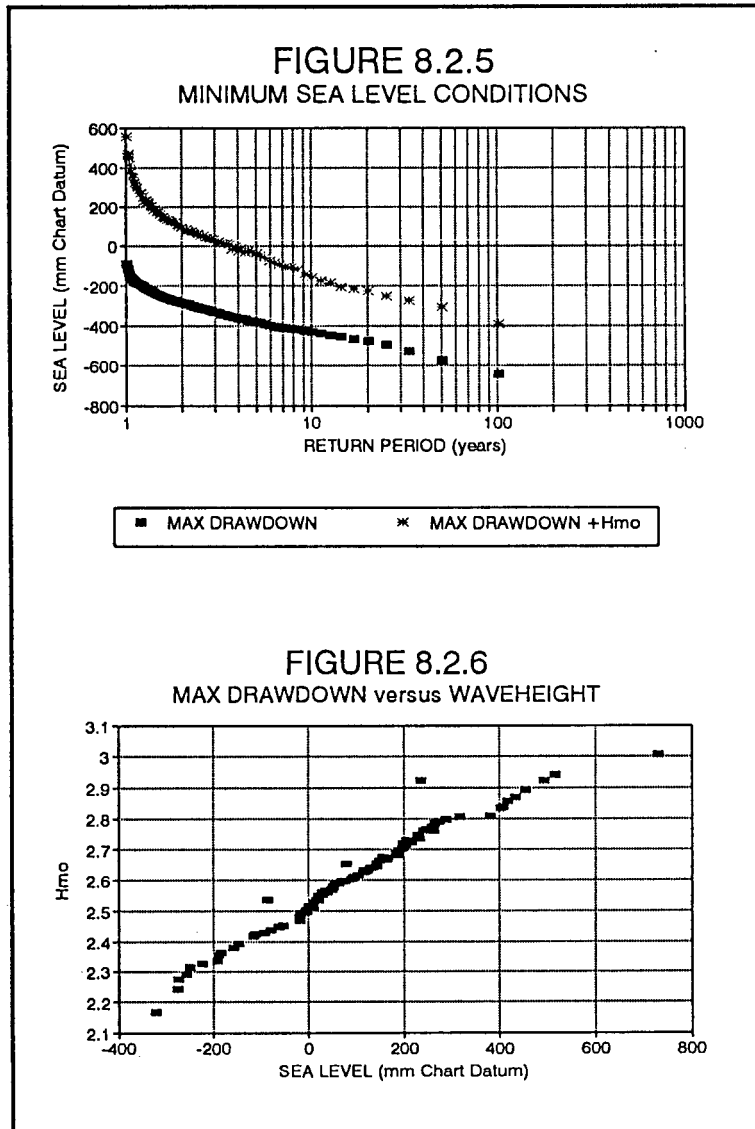
TABLE 8.2.5.

MINIMUM SEA LEVEL AT BREAKWATER

RETURN PERIOD (yrs)	MINIMUM SEA LEVEL		WATER DEPTH (m)
	C.D. (mm)	M.S.L. (mm)	
1	-91	-991	2.71
5	-380	-1280	2.41
10	-431	-1371	2.37
25	-494	-1394	2.31
50	-572	-1472	2.23
100	-642	-1542	2.16

The maximum drawdown versus wave height combinations are given in table 8.2.6. and illustrated in figure 8.2.6.

MOSSEL BAY BREAKWATER DESIGN CONDITIONS



Water Depth = 3.7 m MSL

TABLE 8.2.6.

MINIMUM SEA LEVEL/MAXIMUM WAVE HEIGHT AT BREAKWATER

RETURN PERIOD (YEARS)	NEARSHORE WAVE HEIGHT (m)	ASSOCIATED SEA LEVEL		WATER DEPTH (m)
		C.D. (mm)	M.S.L. (mm)	
1	3.01	+559	-341	3.36
5	2.50	-281	-928	2.77
10	2.38	-156	-1056	2.64
25	2.29	-254	-1154	2.55
50	2.24	-306	-1206	2.49
100	2.17	-392	-1292	2.41

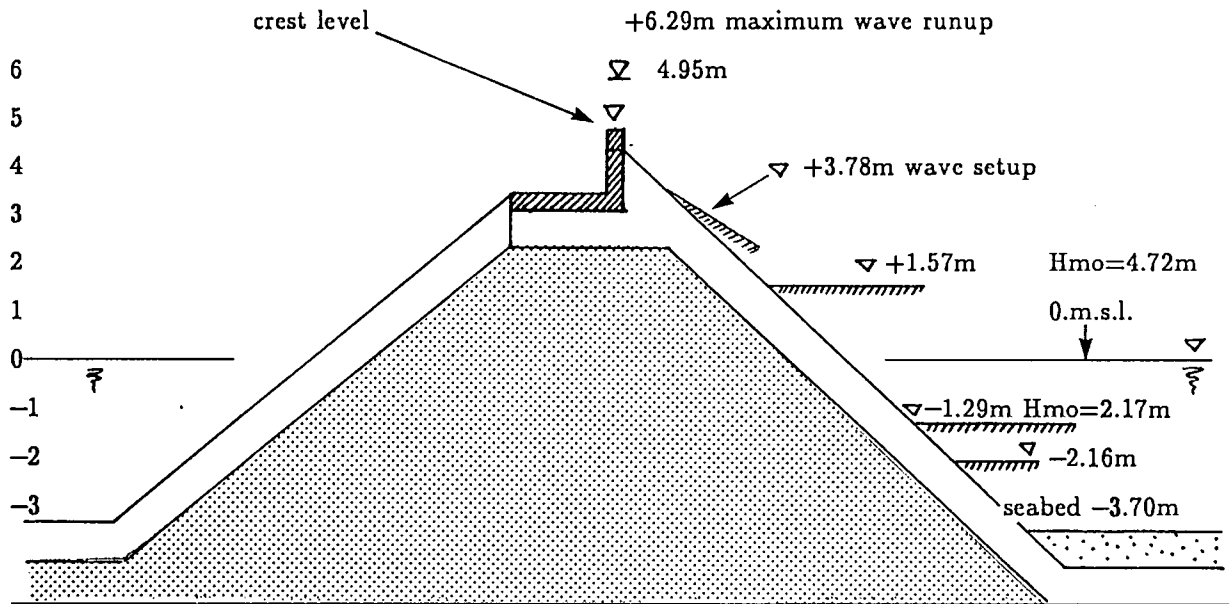
It can be seen from table 8.2.6. that the wave conditions are depth limited. Although the minimum sea level stand will expose the lower slope and toe of the breakwater to wave action, the wave energy will be limited. Bruun (1985) considers it necessary to extend the primary armour units down to an elevation of $-2 * H_{mo}$. This would imply having to extend the units to -5.63 metres m.s.l. Although this is a severe condition it is not likely to constitute the worst for the toe. This would be represented by the maximum combination of the required primary unit extent and the associated sea level.

A number of design conditions have been assessed using the model. These results may be compared with the conventional approach as presented in CSIR (1988) and other publications. A summary of the 1/100 year design conditions are given in figures 8.2.7.(a) and (b) respectively. The results given by the model emphasize the importance of evaluating the combined design conditions of sea level and associated wave height for depth limited structures. The stochastic nature of both parameters introduces new variability into the assessment which is reiterated by the dependency of wave height on water depth. It has been shown, for the case study, that the armour unit size increases by 30% when this relationship is taken into consideration. It can be shown furthermore that the relative importance of variable sea level increases as one progresses from deep to shallow water. This may be attributed to the larger number of

FIGURE 8.2.7.

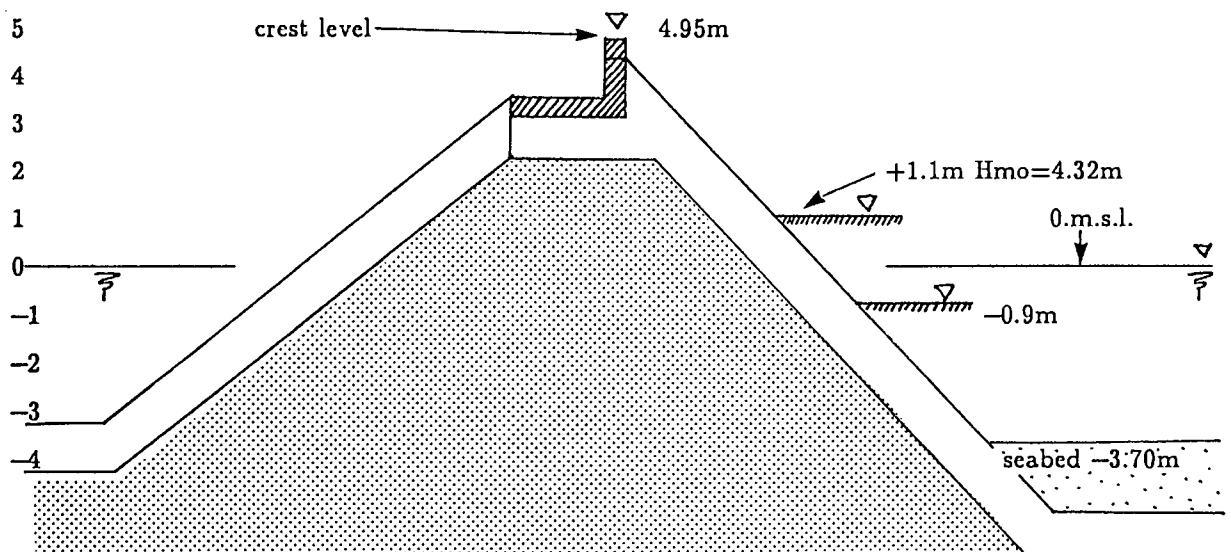
SUMMARY

DESIGN CONDITIONS MODEL



1/100 YEAR CONDITION

DESIGN CONDITIONS : CONVENTIONAL APPROACH



MAXIMUM CONDITION

small waves in the distribution which are depth limited. In practical terms this implies that the defined level (MHWS) will deviate more in shallow than deep water which will result in the underestimation of the final design wave using the former approach.

The breakwater assessment confirms the observation that extensive overtopping will occur during south easterly storm conditions, possibly several times a year. On the other hand, minimum sea level has been shown to be substantially lower than would be expected in conventional design practice. This should have an implication with regard to the foundation stability and extent of primary armour units. It should be noted that the sand bed can be expected to vary. This will affect the design depth which affects the design wave. This parameter should ultimately be included in any detailed assessment as a stochastic variable.

The case study presented in this section can be seen as representative of the information necessary for the quantification of the load component in the level II or level III probabilistic assessment. These results can be used to assess the probability of the failure of the breakwater under various design scenarios once the actual resistance or strength of the structure has been evaluated.

CHAPTER 9

CONCLUSIONS AND RECOMMENDATIONS

The thesis has dealt with a broad spectrum of topics relating to design sea level in southern Africa in an endeavour to develop a more rational quantitative basis to current coastal engineering practice. The processes affecting sea level have been identified and described. The underlying theoretical basis to the work undertaken in the thesis has been set out. All available data relating to the individual processes have been analyzed and described. A convolution model has been developed, validated and applied to a particular case study as an illustration of the model. In this chapter an attempt is made to summarize the most important findings of the work, and define possible areas of future research. In order to structure this chapter, conclusions and recommendations will be discussed under the headings :- processes affecting sea level in southern Africa, data analysis, the model development and verification, the model application and general comments.

PROCESSES AFFECTING SEA LEVEL IN SOUTH AFRICA

1. It would appear from the literature that the assessment of design sea levels in southern Africa should not be undertaken using techniques developed for application in the other parts of the world. The fundamental reasoning behind this proposition is that southern Africa is subject to a unique combination of responses which differ in time and space scales with processes normally considered in coastal engineering practice.
2. The dominant processes affecting sea level in Southern Africa are astronomical tides, shelf waves, wind waves and edge waves. Other phenomena such as wind setup, tropical cyclones and tsunamis are only considered as being of secondary importance.
3. Wind setup and tropical cyclones would only appear to affect highly localized coastal areas and are therefore not the subject of detailed assessment. Mechanisms capable of generating tsunamis in southern Africa do exist, however, these events are considered extremely rare and are therefore not considered as part of conventional coastal engineering design.

DATA ANALYSIS

1. Southern Africa is subject to moderate tidal variations which are relatively uniform along the entire coastline. Tidal range varies between 0.56 m and 2.48 m with the east coast being marginally more responsive than the west coast. The **TOGA** tidal prediction model, with some modification, was found to be suitable for predicting long term (18.6 year) hourly tidal levels for ports in southern Africa.
2. Shelf wave and wind wave events can be characterized in terms of their peak magnitude, duration and recurrence interval using a three parameter threshold analysis technique. This technique would appear to be an effective method of isolating independent identically distributed events from existing data sets.
3. The analysis of the available data from Mossel Bay, Port Nolloth and Richards Bay could not identify any coherent relationship between the magnitude of shelf waves, wind waves or wind speed at these locations. Whilst some form of relationship would appear to exist with regard to the joint occurrence of shelf wave and wind wave events, nothing but a correlation is evident between the associated event magnitudes. For the ports under consideration, the assumption of statistical independence between processes would appear to be justified.
4. The response of a particular location to shelf waves around the southern African coast is variable. The port of Mossel Bay would appear most sensitive to shelf wave activity in both magnitude and duration. Approximately twice as many events, larger than 100 mm, can be expected at this point than at Richards Bay or Port Nolloth. This may in some way be related to the significantly wider continental shelf at Mossel Bay.
5. Current literature on shelf waves proposes that events are generated on the West coast and propagate eastwards. This would appear to be reflected in the statistics for Port Nolloth and Richards Bay. Port Nolloth is characterized by medium size short duration events which are expected in a

generating area. This may be compared with the smaller long duration events measured at Richards Bay, which appear to have lost some of their energy, which is characteristic of areas some distance from the source. The data set, therefore, remains consistent with our physical understanding of the shelf waves in southern Africa.

6. The evaluation of negative shelf waves smaller than -100 mm reflect the same trends as the positive wave heights. The total expected duration of events less than -100 mm would appear to be larger for negative than positive waves. This asymmetry may be attributed to the forced nature of these waves.
7. The trends identified by Rossouw (1989) with regard to wind waves in Southern Africa were confirmed in this study. Significant differences were found between the waverider stations at Mossel Bay, Port Nolloth and Richards Bay. Mossel Bay registered the highest waves for the longest period of time (above a three metre threshold). Almost twice as long as Port Nolloth and ten times as long as Richards Bay. These differences have a significant bearing on the probability of occurrence of combined events.
8. The quantitative evaluation of edge waves in southern Africa is restricted by the limited extent of the data available. Overall characteristics were defined, based on published literature. It would appear that long period edge waves are essentially restricted to the west and south coast as far as Port Elizabeth.

MODEL DEVELOPMENT AND VERIFICATION

1. It has been shown in chapter 7 that it is possible to produce synthetic hourly sea level data possessing the same statistical characteristics as the observed data set for the purpose of assessing design sea level at particular locations.
2. For the ports investigated in this study, it is possible to combine astronomical tide, shelf waves, wind waves and edge waves to obtain a

convoluted sea level distribution of annual maximum values. These values may be used to evaluate the design sea level of interest for a specific location. A comparison between the Gumbel method for estimating extreme values and the model, for 18 years of Simons Bay and 33 years of Port Nolloth data, would appear to confirm that the model is capable of generating realistic results.

3. The model when compared with conventional methods of estimating extreme design events, possesses less variability as measured by the 95% confidence level. The reduction in variance can be attributed to the larger number of events used to characterize the underlying processes. This is particularly relevant in a region where the extreme events are defined by a combination of a number of moderate magnitude events rather than one single process.
4. The model was found to be less sensitive to the impact of short duration data sets than conventional methods. The model would appear most sensitive to event magnitude and the rate of occurrence of events when estimating extreme levels.
5. The inclusion of the dependency structure of tides and event information with regard to the stochastic processes affecting sea level would appear to be the major contributing factors towards a more accurate estimation of extreme design levels.

APPLICATION OF THE MODEL

1. A regional assessment of design sea levels reflects the same overall trend identified in the underlying processes. Shelf wave and wind waves appear to represent the dominant forcing components. The largest variations in sea level (highest and lowest) occur on the south coast. It is notable that HAT and LAT should be expected to be exceeded at least every $2\frac{1}{2}$ - 3 years for all the ports around the coast. In the case of the south coast LAT will be exceeded several times per year. The practice of using HAT or LAT as extreme design sea levels should therefore be discontinued. Similarly port

authorities should be made aware that water depths indicated on navigational charts can, under certain conditions, be up to 600 mm shallower than indicated.

2. The regional differences in design sea level can be attributed primarily to the shelf waves and wind waves. The characteristics of these processes on the west, south and east coasts are essentially different. Thus, although design levels on the west and east coast may seem similar, spatial transfer of data should not be considered between regions.
3. The assessment of the Mossel Bay breakwater indicates quite clearly that the prototype structure was under designed. The results indicate furthermore that an appropriate 1/100 year significant wave height/sea level combination would be 4.72 m and +1.57 m m.s.l. respectively. The conventional practice of using mean high water spring (MHWS) +1.1m m.s.l. as a design level would result in the under estimation of the design wave height, runup and overtopping for the structure.
4. Mean low water spring (-0.65 m m.s.l.) would appear to considerably underestimate the maximum drawdown for the structure. The model estimates the 1/100 drawdown to be -1.54 m m.s.l.
5. The design range for the structure would be expected under normal design conditions to be MHWS (+1.1) - MLWS (-0.65) = 1.75m. The expected range for the model for the 1/100 combined wave height/sea level and wave height/drawdown would be (+1.57m) - (-1.292) = 2.86m.
6. The model illustrates the sensitivity of depth limited structures to design sea level. The relative importance of sea level increases with decreasing water depth. Structures designed in the past using depth limited conditions assuming a design sea level at MHWS are likely to have been under designed.

GENERAL

1. Design sea levels in southern Africa represent the resultant combination of a number of moderate magnitude events. Conventional extreme value techniques are not suited to estimating extreme values for combinations of events. Individual processes should be combined taking into account their particular characteristics of serial dependency and event structure. (magnitude, duration and rate of occurrence).
2. Any estimation of extreme levels must take into consideration the expected event duration as well as the magnitude and annual rate of occurrence. Events with longer expected durations and higher rate of occurrences will have a larger probability of occurring in conjunction with large tidal or wind wave events.
3. For shallow water structures, sea level should be treated as a stochastic rather than deterministic variable. These structures should be assessed for the maximum combined design condition (sea level + associated wave height).
4. The relative importance of variable sea level, in the evaluation of design waves, increases from deep to shallow water. The greater number of smaller wave events, increases the probability that high sea level and large wave events will occur concurrently.

RECOMMENDATIONS

The methodology developed in this thesis provides the basis for the assessment of design sea level for the Southern African region. Whilst the framework has been developed a number of recommendations may be made with regard to areas of further research.

1. More data should be made available and analyzed so that the model input distributions may be updated which will improve overall model accuracy. Furthermore assessments should be undertaken for other ports around the coastline in order to improve our current understanding of specific areas.

2. A comprehensive statistical analysis of all the available data on edge waves would greatly improve on the assumptions made, in this regard in the thesis.
3. More work should be undertaken with regard to the possible interrelationships between the driving processes. More comprehensive field data will be required in order to investigate this problem.
4. Some interesting results were obtained in this study with regard to the statistics of storms (shelf waves and wind waves) around Southern Africa. Important insight could be obtained from a more detailed assessment of storm characteristics and their subsequent incorporation into the model.
5. Whilst the characteristics of the loading component in the design procedure have been dealt with in this study, a parallel effort should be made to assess the stochastic nature of structural response for coastal engineering structures. This will facilitate the application of a full probabilistic design approach in future.
6. The stochastic model developed in this study is not region or process specific. The model could therefore be applied to other parts of the world and for other processes. A particularly useful addition would be to incorporate local wind effects with a view to small craft harbour design.
7. The generation of a synthetic hourly sea level and wave record displaying the same statistical characteristics as the original observed records, implies that the approach developed in this thesis could have widespread application as a more realistic input to existing design models for assessing beach morphology, structure stability and construction and operational conditions.

CHAPTER 10

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