DESIGN WAVES FOR THE SOUTH AFRICAN COASTLINE

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Declaration

I the undersigned declare that the work contained in thesis is my own original work and has not previously in its entirety or in part been submitted at any University for a degree.

Signature [signature] Date: 5/12/88

DESIGN WAVE CONDITIONS FOR THE SOUTH AFRICAN COASTLINE

SYNOPSIS

Several aspects related to the estimation and selection of design wave conditions were investigated.

An analysis program which includes strict quality control routines was developed for digital Waverider data. All available Waverider data from deepsea records were analysed with this program.

A remarkable similarity in simultaneously recorded wave heights between Cape Town and Port Elizabeth was found. This similarity was used to compile a near continuous wave record over an eight year period for the Southern Cape coast.

The 10537 values of significant wave height (H_{mo}) which made up the record for the Southern Cape were found to give a good visual fit to the Extreme I and Log-normal distributions over the entire range of H_{mo} values.

Design wave heights derived from the Extreme I distribution were found to be insensitive to assumptions regarding the independence and identical distribution of the wave height samples and the method used for parameter estimation.

Design wave heights for the coastline between Oranjemund and Port Elizabeth were found to be strongly correlated to the latitude of the recording site. High waves along these coasts are invariably caused by the passage of cold fronts past the southern tip of the continent. Wave heights reduce as the distance from the west to east route of these cold fronts increase, thus the reason for the abovementioned correlation.

No deepwater wave records are available east of Port Elizabeth. Shallow water records indicate that a reduction in wave height can be expected between Port Elizabeth and East London.

Cyclones affect the extreme wave heights along the Natal North coast and standard methods of design wave height estimation are not applicable in this area.

Wave period distributions between Oranjemund and Port Elizabeth are virtually identical with an 80 percent range of peak energy period (T_p) of 9s to 16s and a median T_p of 12,5s. East of Port Elizabeth wave periods reduce gradually all the way up to Richards Bay.

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LIST OF SYMBOLS

The following symbols are used in this paper.

Notation	Units	Description
a, b	m	Parameters of the Exponential distribution
E		Percentage exceedance of wave height
f	Hz	Frequency
f_N	Hz	Nyquist frequency
f_p	Hz	Spectral peak frequency, i.e. frequency at which S(f) is a maximum
f(x)		Probability of occurrence of x
F(x)		Cumulative probability of x i.e. $F(\hat{H}) = \int_0^{\hat{H}} f(H)\,dH$ = probability of non-exceedance of $\hat{H}$
h	years	Design horizon, i.e. design life of the structure
H	m	Wave height
H_1^{10}	m	Highest crest and lowest trough in 10 minutes
$H_{1/3,u}$	m	Zero upcrossing significant wave height. Average of the highest one-third zero upcrossing wave heights
H_{max}	m	Maximum individual wave height
H_{mo}	m	Estimate of significant wave height, $4\sqrt{mo}$

H_{mo_R}	m	Most probable value of H_{mo} with a return period of R years, eg. $H_{mo_{100}}$
H_s	m	Significant wave height defined as the average of the highest one-third of the wave heights calculated as $H_{1/3,u}$ or estimated as (a) H_{mo} (b) H_σ
H_{sinc}	m	Incorrect estimate of H_s which underestimates H_s by 5 to 7 percent
H_σ	m	Estimate of significant wave height, $4\sigma_\eta$
$H_{mo}(95)$	m	Estimate of H_{mo} at 95 percent confidence
m_n	m^2Hz^n	nth moment of spectral density, i.e. $m_n = \int f^n S(f)\, df$
p		Probability of non exceedance, i.e. equivalent to F(x)
r		Risk of exceedance
S(f)	$m^2.s$	Spectral density
t	s	Variable of time
T_G	s	Average period of wave group
$T_{H_{max}}$	s	Period of H_{max}
T_{mo2}	s	Average period defined by $\sqrt{mo/m2}$; $T_{o,2}$ m IAHR list, frequency domain estimate of T_z
T_p	s	Spectral peak period, $1/f_p$

T_r	s	Record length
T_R	years	Return period of recurrence interval
T_z	s	Average zero upcrossing period (Tu in IAHR list)
$W(t)$		Windowing function
$x(t)$	m	Value of the instantaneous water surface relative to a given datum
α, β	m	Parameters of the Extreme I distribution
$\tilde{\alpha}$, $\tilde{\beta}$	m	Estimates of the parameters of the Extreme I distribution
$\eta(t)$	m	Water surface elevation referred to mean water level
σ		Standard deviation
σ_η	m	Standard deviation of $\eta(t)$
ν		Degrees of freedom of the χ^2 distribution
χ^2		Chi-square distribution
γ		Peak enhancement parameter (Jonswap spectrum)

ACRONYMS

CERC	Coastal Engineering Research council (US)
CSIR	Council for Scientific and Industrial Research
FFT	Fast Fourier Transform
GMT	Greenwich Mean Time
IAHR	International Association of Hydraulic Research
NIO	National Institute of Oceanography (UK)
NMERI	National Mechanical Engineering Research Institute of CSIR (RSA)
NRIO	National Research Institute for Oceanology (RSA)
POT	Peak over Threshold
SANCOR	South African National Committee for Oceanographic Research
SOEKOR	Southern Oil Exploration Corporation
VISKOR	Fisheries Development Corporation of S.A. Ltd.
VOS	Voluntary Observing Ships

DESIGN WAVE CONDITIONS FOR THE SOUTH AFRICAN COASTLINE

1. INTRODUCTION

This study originated out of a need to consolidate all the available wave information along the South African coast to enable the proper selection of deepsea wave conditions for the design of various structures along the coastline. At the start of the study in early 1978 a number of sources of wave data were available. These included visual estimates by Voluntary Observing Ships (VOS), instrument aided visual estimates (clinometer) and recorded data from a number of different types of wave recorder such as the NIO shipborne recorder, pressure transducers, inverted echo sounders and accelerometer buoys (Waveriders). Design wave heights and periods derived from these data sources varied in a rather haphazard fashion around the coast and no clear understanding of the wave climate could be derived from these data.

The original aim of this study was therefore to do a systematic check of all the available data and to evaluate the accuracy and reliability of the various data sources. The more reliable sources were then to be used to develop a general understanding and description of the wave patterns along the South African coastline.

As the study progressed it soon became apparent that by far the most reliable source of wave height and period information were the data collected by the accelerometer buoys. Much of the original Waverider data were only available in analogue form (strip chart records) and large quantities were still un-analysed. Where digital data were available various shortcomings in the analysis programs and more specifically the quality control of the data were discovered. This resulted in many invalid records finding their way into the data banks and distorting the wave patterns emerging from the data.

To improve the data coverage all the raw analogue records were subjected to a simple analysis to provide a wave height parameter for each record. A computer program which included extensive quality control routines was developed and all available digital records were re-analysed with this program. Only data passing strict quality control routines were used in the further analysis.

In March 1984 a report summarising the results of the study and containing all available data up to July 1981 was published (Rossouw, 1984). The main purpose of that report was to make the data available to designers. No effort was made to provide theoretical background or to explain the methodology used in wave analysis or design wave determination.

This PhD dissertation attempts to give more of the theoretical background and to explain methodologies used in the analysis and presentation of the data. It describes in detail certain aspects of wave measurement and analysis pertinent to the study and the logic used in the quality classification of wave records by computer. The weather patterns responsible for generating large waves along the coast are described and examples of a number of these storms are given.

The spatial variation in wave height along the coast is investigated and this knowledge is applied to compile a virtually continuous wave record over a period of eight years for the Southern Cape coast. This data set is used to investigate certain aspects of the long term wave height distributions and to make recommendations about the methodology to be used in arriving at values for design wave heights. The recommended methodology is then applied to all available deep sea wave data and a design wave height distribution for the South African coastline is obtained.

The short term statistics obtained during a number of the largest recorded storms are also investigated and this information is used to make recommendations about the periodicity, spectral form and groupiness of the design waves.

Finally a few examples are given of how the data contained in this dissertation could be applied for design purposes. Recommendations for improvement of wave recording, wave analysis and the design wave methodology are also made.

A fold-out map of South Africa showing all the recording sites and place names used in this dissertation is provided at the back of this document.

2. WAVE INFORMATION REQUIRED BY THE DESIGN ENGINEER

2.1 GENERAL

When designing structures in the sea the wave information required by the design engineer can vary widely depending on the type of structure he is designing and the design procedure he is following. In the simplest case of deterministic design such as calculating the maximum force on a submerged cylinder, all that is required is the maximum individual wave height that can reasonably be expected to occur during the lifetime of the structure and the likely range of periods and directions associated with that wave. If a structure is to be model tested using irregular waves or if it is subjected to some form of dynamic analysis, the significant wave height and energy density spectrum of the design storm as well as some short term statistics such as the height distribution and occurrence of wave groups during the storm are required. If resonance of the structure is expected to be a problem, information on maximum energy levels at the resonance frequencies are also of importance. In fatigue analysis the distribution of individual wave heights and periods during the lifetime of the structure is required. In addition to the above, information on the persistence of calms or storms are required for estimating downtime during construction.

In this study emphasis is placed on the estimation of wave height and wave period parameters during extreme events.

2.2 WAVE HEIGHT PARAMETERS

2.2.1 Design Significant Wave Height

The emphasis in this study is on the determination of the significant wave height with a given risk of being exceeded during the design lifetime of the structure. Of all the design parameters the extreme values of significant wave height have received by far the most attention and hundreds of articles and reports dealing with this subject have been published. Notwithstanding all the attention, standard procedures for estimating this parameter have not yet emerged. Chapters 7 to 9 of this dissertation deal with this subject.

2.2.2 Maximum Individual Wave Height

The maximum individual wave height that may occur during the design storm is normally derived from the design significant wave height discussed in Section 2.2.1 above by making assumptions about the duration of the storm and the distribution of individual wave heights in this storm. The same approach is followed in this study and the validity of the assumptions is investigated.

2.2.3 Occurrence of Groups of High Waves

The occurrence of groups of high waves during the design storm have recently been recognised as an important design parameter It is of particular importance where some form of resonance can occur at the structure. Procedures for describing the groupiness of waves and incorporating groupiness parameters in the design process are at present the subject of active research and standard procedures for this purpose have not yet found general acceptance. In this study a simple description of the occurrence of groups during a few of the more severe storms is given.

2.3 WAVE PERIOD PARAMETERS

2.3.1 Energy Density Spectrum Associated with the Design Storm

The best description of the wave period characteristics of the design storm is in terms of its spectral density. The shape of the spectrum associated with extreme values of significant wave height is an important design parameter and is discussed in Section 15.4 of this report.

2.3.2 Peak Energy Period (Tp) and Zero Crossing Period (Tz) Associated with the Design Storm

Since it is cumbersome to describe the periodicity of waves in terms of the energy at each frequency, a single wave period parameter is mostly employed for this purpose. The two most frequently used parameters are Tp and Tz. None of these parameters describe the periodicity of the waves adequately. They are however used in this dissertation since especially Tp is found to correspond to the higher waves in the wave train.

2.3.3 Period associated with the Maximum Individual Wave and Groups of High Waves

For many designs it is necessary to estimate the period of the maximum individual design wave height discussed in Section 2.2.2 above. Similarly the period associated with groups of high waves in the design storm needs to be estimated. Procedures for estimating these periods are discussed in Section 15.3.

3. WAVE RECORDING

3.1 GENERAL

This section contains a short summary of the history of wave recording in South Africa. Most of the data used to describe the wave height and period distributions around the South African coast were obtained from digital Waverider records. Some pertinent aspects of digital recording by Waverider which have a bearing on the results of this study are discussed.

3.2 HISTORY OF WAVE RECORDING IN SOUTH AFRICA

In this section only wave recordings are considered which have supplied data that may be of use in determining design wave conditions on the South African coast. The first recordings of this type were the visual observations of wave height, wave period and wave direction made from ships. Voluntary observing ships (VOS) record wave conditions at six-hourly intervals and the data of interest to South African conditions are stored in computer-compatible form by the Dutch and German weather bureaux for the areas East and West of the 20°E meridian respectively. Data for the period from 1944 onwards are available and form an extensive data base.

Wave data for use in coastal engineering projects in South Africa were first collected by means of a wave clinometer off the Bluff in Durban in April 1961 (CSIR, 1970). The wave clinometer consists of a telescope having graduations on one lens through which nearshore wave directions are observed from a high vantage point on the shore. Visual estimates of wave height and wave period can also be made by observing the movements of a moored buoy through the graduated lens of the telescope. A more complete description of the clinometer has been given elsewhere (CSIR, 1968).

During the period 1964 to 1969 the research ships Africana II, Thomas B Davie, Meiring Naudé and Benguela, as well as the survey ship SAS Natal were fitted with NIO shipborne wave recorders. These recorders, in which use is made of a combination of accelerometers and pressure sensors, were fitted to the ship's hull and gave as output an analogue trace of the water surface from which wave height and wave period data could be extracted. Wave directions were estimated visually from the bridge of the ship with reference to the ship's compass. The NIO recorders are more fully described in CSIR (1970). South Africa also operated a weather ship, the F H Hughes during the period September 1969 to March 1974. The weather ship was usually stationed at 40°S and 10°E. It was fitted with a Boersma recorder which was later replaced by a NIO recorder.

In February 1967 a wave research group was established within the National Mechanical Engineering Research Institute (NMERI) of the CSIR with the main aim of recording and statistically analysing wave conditions along the coastlines of South Africa and South West Africa. This so-called "Ocean Wave Research Project" was initiated by the South African National Committee for Oceanographic Research (SANCOR) with financial support and guidance from SANCOR, VISKOR, SOEKOR and NMERI.

The main wave measuring instrument used during the period 1961 to 1970 was the wave clinometer. The number of wave clinometer stations was increased from one in 1965 to twelve in 1970. The wave clinometer stations continued in service until late in 1974 when most of the wave measurement by means of the clinometer was stopped.

In an attempt to improve on the accuracy of the visually measured wave data obtained by the clinometer, a number of wave-recording instruments were experimented with in the period 1966 to 1969. First an inverted echo sounder (Kelvin Hughes), connected by cable to the shore was tried in Durban and Cape Town, but it was eventually abandoned mainly because of difficulties with the laying and maintenance of the cable through the surf.

A self-contained inverted echo sounder (INES) was developed and used for a while but it suffered from many internal defects and from leakages and eventually fell into disuse without producing many useful results.

A few pressure recorders (OSPOS) were bought from Van Essen in Holland by NMERI and VISKOR and these self-contained units proved to be very reliable. The records were, however, difficult to analyse and doubts were expressed regarding the transfer functions used to convert the pressure-record to a water-surface record.

In July 1969 an accelerometer buoy (Datawell Waverider) was installed in 100m water depth off Mossel Bay. The moored buoy transmitted to shore where the data were recorded. Initially a paper tape recorder was used but it never functioned satisfactorily and later all recordings were made on strip charts. The Waverider soon proved to be superior to any of the wave-recorders tried previously and in the period 1971 to 1973 the number of Waverider stations was increased from one to seven. All these stations recorded waves in analogue form on paper rolls usually for 20 minutes every six hours. All analysis had to be done by hand and by late 1974 the backlog in analysis became so large that it was decided to reduce the number of Waverider stations to those required for urgent coastal engineering studies until instrumentation and software could be developed for recording in computer-compatible form for analysis by computer. The first digital recorder was installed at Slangkop in February 1976.

Although the Waverider has become virtually the standard wave-recording instrument in South Africa, it does not measure wave direction. In an effort to obtain regular wave direction information, VISKOR developed the DOSO. This instrument senses the direction of the orbital motions of the wave and is normally placed on the sea bottom in a water depth of less than 20m. A more complete description of the DOSO was given by Retief (1974). Useful data have been obtained from the DOSO at a few sites such as Koeberg and Gansbaai.

Radar was also used to record nearshore wave direction at Koeberg and useful, although intermittent, information was obtained by analysis of photographs of the image on the radar screen. Since the DOSO and Radar were used to measure waves in relatively shallow water, refraction techniques were required to obtain the more generally applicable deepsea wave direction.

3.3 THE WAVERIDER RECORDING SYSTEM

3.3.1 The System

The Datawell Waverider system consists of a 600mm diameter floating buoy on a flexible anchorage system. The vertical acceleration of the buoy is measured as it follows the water surface. This acceleration is integrated twice to provide the vertical displacement of the water surface. The analogue signal of vertical displacement is continuously transmitted by a radio transmitter contained in the buoy and recorded at given intervals at a shore station. The recording medium is normally magnetic tape (for computer analysis) backed up by a strip chart recorder. Before computer analysis the record is converted from analogue to digital form.

In the design of the recording system basic decisions are required, regarding the sampling interval, duration of the record and recording interval.

3.3.2 Sampling Interval

When converting the analogue signal to a digital record, it is necessary to decide on the sampling interval, i.e. the time interval between individual samples of the water surface. In South Africa and in fact in most countries where waves are recorded, this time interval (Δt) is normally 0,5s. This sampling interval has a direct bearing on the highest frequencies that can be recorded by the system. If there is energy present in the record at frequencies higher than the so-called Nyquist frequency

$$(f_N = \frac{1}{2\Delta t}),$$

this energy will be erroneously recorded at lower frequencies. The errors due to Nyquist folding cannot be corrected after the record is digitised. Filtering the analogue record before it is digitised to get rid of all energy at frequencies higher than 1Hz (periods less than 1s) is the only way to ensure that Nyquist folding does not occur. Since the wave energy at these high frequencies is very low, filtering of the analogue record is not done in South Africa.

Using unfiltered records also provides the opportunity to test for the quality of the record (see Section 4.6.2).

3.3.3 Duration of the Record

For many of the spectral analysis routines used in the analysis of waves, the number of samples used in the analysis should be 2^n where n is an integer. Typical values used for n are 10, 11 and 12 which for Δt = 0,5s leads to record lengths of 512s, 1024s or 2048s. In South Africa, to date a record length of 1024s (17m 4s) has been used. This record length has a direct bearing on the frequency resolution and the accuracy of the energy density spectrum derived from the sample, both improving with increasing record length. The record length also has a bearing on the accuracy of the estimate of other wave parameters such as for example, the significant wave height. The duration of the sample is however limited by practical considerations such as storage space on the recording tape, economy of computer analysis and the stationarity of the wave condition. These aspects are further explained in Section 4.4 where wave analysis is discussed.

3.3.4 Recording Interval

In South Africa wave records are obtained at 6 hour intervals generally at 0h00, 06h00, 12h00 and 18h00 GMT. These records are considered to be representative of a six hour period, i.e. for the 3 hour period before and after the recording. In most of the analyses wave conditions are assumed to be stationary during this 6 hour period. In many countries a recording interval of 3 hours has become the standard. Advantages of a shorter recording interval are a reduced risk of missing the peak of the storm, a shorter gap in the record if a recording is lost and an improved description of the variation in wave conditions with time. Obvious disadvantages are increased storage requirements for raw and analysed data and increased analysis costs. Recommendations about appropriate recording intervals are made in Section 17.1.

3.3.5 Frequency Response of the System

The response curve of a typical Waverider system is shown in Figure 3.1. The amplitude response of the Waverider starts falling off at periods exceeding 14s. For periods between 1s and 2s resonance occurs and amplitudes can be overestimated by up to 200 per cent.

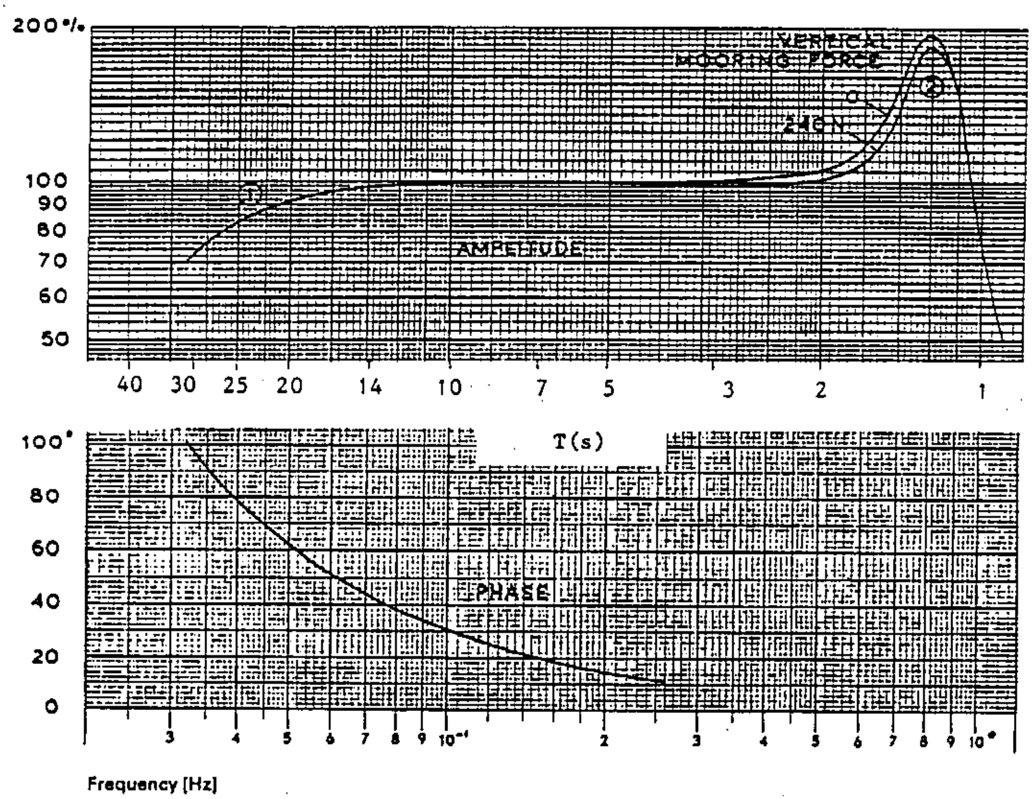


FIGURE 3.1 : WAVERIDER RESPONSE

(Datawell, 1979)

To date no response corrections have been made to South African Waverider records and slight under-estimates of wave height could result especially for the longer period waves. The influence thereof on the accuracy of wave height recording is briefly considered in Section 12.5

3.3.6 Calibration

Calibration tests on the recording system are performed on a regular basis by rotating the buoy in a specially constructed calibrating device. The accuracy of the recorded amplitudes and periods are normally within 2 per cent.

Comparisons between Waveriders and other instruments such as Wavestaffs were also undertaken. In general these studies confirmed the confidence in the Waverider system and where differences occurred these were mainly attributed to the alternative recording instrument (CSIR 1983 and CSIR 1986).

4. WAVE ANALYSIS

4.1 GENERAL

The history of wave analysis has been described elsewhere (Rossouw 1984) and is not repeated here. In this dissertation only aspects of wave analysis that have a direct bearing on the results, conclusions and recommendations of this study are discussed. Only the analysis of analogue and digital Waverider records are considered and only wave parameters relevant to this study are defined. Where possible the notations and definitions of the International Association for Hydraulic Research (IAHR) are adhered to (IAHR 1986).

4.2 TIME DOMAIN ANALYSIS OF ANALOGUE RECORDS

In the early days of wave recording in South Africa, most of the Waveriders recorded only on paper chart recorders. An example of such a record is shown in Figure 4.1. These records had to be hand analysed and the technique mostly used was that proposed by Draper (1966).

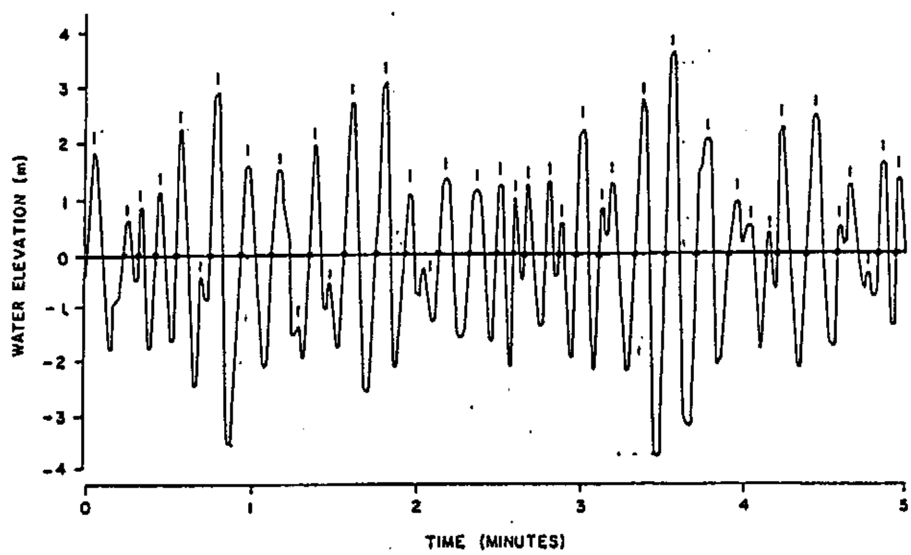


FIGURE 4.1 : EXAMPLE OF WAVERIDER RECORD

The following parameters of relevance to this study were produced by this analysis: H_1, H_{max}, H_s and T_z. Since H_1 and H_{max} are functions of the record length the notation H_1^{10} and H_{max}^{10} are used to indicate that a ten minute record was used in the analysis. Due to errors made in the application of the Draper analysis to some of the earlier records (Van Wyk, 1978) an incorrect H_s was derived which under-estimated the true H_s by approximately 5% to 7% (Rossouw 1984). This incorrect estimate is termed $H_{s_{inc}}$ in this study.

4.3 TIME DOMAIN ANALYSIS OF DIGITAL WAVE RECORDS

In the time domain analysis of digital records the parameters H_1, H_{max} and T_z can be calculated in an identical fashion to those from analogue records. In general the duration of the records used in the case of digital records was however either 512s or 1024s which will have an influence on H_1 and H_{max}.

To calculate H_s from a digital record it is more convenient to base the estimate of H_s on the standard deviation (σ_η) of the instantaneous water surface, i.e.

$$H_s = 4.\sigma_\eta = H_\sigma$$

4.4 FREQUENCY DOMAIN ANALYSIS OF DIGITAL RECORDS

In the past a number of different programs were used for the analysis of digital records. Most of these programs were poorly documented and will not be discussed here. In the early stages of this study the need for a well documented analysis program was realised and such a program was developed in cooperation with Dr C J Visser (Visser (1980)) who provided most of the statistical and programming input. Most of the data contained in this report were analysed with this program. The main features of the analysis process are :

(i) Windowing

Due to the finite length of a wave record it is necessary to taper the ends of the record to limit energy leakage to adjacent frequencies. This is done before the record is transferred from the time domain to the frequency domain. An extended cosine bell window was applied to the first and last 10 per cent of the record as follows :

$$W(t) = 0{,}5(1 - \cos(2\pi\,5t/T_r)) \quad \text{for } 0 < t < 0{,}1T_r \text{ and } 0{,}9T_r < t < T_r$$
$$= 1 \quad \text{for } 0{,}1T_r < t < 0{,}9T_r$$

where t is elapsed time, and T_r total duration of record.

(ii) Transformation to the Frequency Domain

Many algorithms are available whereby the windowed time domain record can be transferred to the frequency domain. The Fast Fourier Transform (FFT) routine described by Ralston et al (1977) was used for this purpose. The algorithm requires 2^n data points with n an integer. For purposes of data qualification (see Section 4.5) the 1024s record was split in half. This provided a record length (T_r) of 512s containing 1024 data points. The raw energy density spectrum produced in the process has a frequency resolution $\Delta f = 1/T_r = 1/512 = 0.00195$Hz with each spectral estimate containing 2 degrees of freedom ($\nu = 2$). These estimates are rescaled to compensate for the window used by dividing each estimate by 0,9.

(iii) Smoothing of the Spectrum

The raw spectral estimates are χ^2 distributed with 2 degrees of freedom. To improve the accuracy of the estimate the raw spectrum is smoothed by averaging over 5 lines, thereby increasing the degrees of freedom for the smoothed estimate to 10. This results in a frequency resolution of 5x0,00195 = 0,00975 Hz. By further averaging the spectral densities at each frequency of the two halves of the record, the degrees of freedom of each smoothed estimate are increased to 20.

It should be realised that with a frequency resolution of 0,00975Hz, the resolution of period at low frequencies (or high period) becomes rather poor. Estimates of energy density are only made at the following periods in the range 10 to 30s.

f(Hz)	,094	,084	,074	,064	,055	,045	,035
T(s)	10,67	11,91	13,47	15,52	18,29	22,28	28,47

This resolution can only be improved by less smoothing which will result in less accurate energy density estimates, or by increasing the recording length.

(iv) Frequency Domain Parameters

All the frequency domain parameters are calculated via the moments of the spectrum defined as

$$m_n = \int f^n S(f).df$$

where m_n = nth moment of the spectrum
f = frequency
n = integer
$S(f)$ = energy density at frequency f
df = frequency interval

As such the 0'th moment m_0 represents the area under the spectrum (total energy), the first moment m_1 the moment of inertia, etc. The following parameters calculated from the frequency domain are used in this study :

$$H_{mo} = 4\sqrt{m_0}$$
$$T_{mo2} = \sqrt{m_0/m_2}$$
$$T_p = 1/f_p$$

4.5 SELECTION OF PARAMETERS FOR USE IN THIS STUDY

For the purpose of a simple description of a wave record a single height and period parameter for each record is required. For a Gaussian stochastic process it can be shown that $H_s = 4\sigma_\eta = H_{mo} = 4\sqrt{mo}$ and that $T_z = T_{mo2}$

Simple linear regression between H_s and H_{mo} as well as between T_z and T_{mo2}, using the frequency domain parameters as independent variables, were undertaken for a number of deepsea wave records. The correlations found are summarised in Table 4.1 below :

Table 4.1 Correlation between Time Domain and Frequency Domain Parameters

Station	Number of Records	Relationship	Correlation Coefficient (r)
Slangkop	4412	$Hs = 0,02 + 1,007\ H_{mo}$	0,998
Sedco K	4707	$Hs = 0,01 + 1,010\ H_{mo}$	0,998
Slangkop	4412	$Tz = 0,045 + 0,9986\ T_{mo2}$	0,961
Sedco K	4707	$Tz = 0,24 + 1,0096\ T_{mo2}$	0,955

As can be seen from these correlations the difference between the time domain and frequency domain estimates of significant wave height is less than 2 per cent for heights above 2m.

Similarly the difference between T_z and T_{mo2} varies from 3 to 6 per cent for periods in the range 6 to 12s.

In all cases the frequency domain parameters lead to slightly higher values than the time domain parameters. These differences are small enough to be ignored.

Where possible in this study H_{mo} will be used as the wave height parameter. Where other height parameters are used this is clearly stated.

The wave period parameters T_z and T_{mo2} are both influenced by either recording or analysis techniques. T_z is very dependent on the response of the recording instrument and will decrease with increasing sensitivity of the instrument. Similarly T_{mo2} is very dependent on the high frequency cut-off of the spectral estimates since it is calculated using the second moment of the spectrum. Both these period parameters are unsatisfactory and often have little physical meaning.

The preferred wave period parameter is T_p and where possible this parameter is used to describe the periodicity of the waves. Unfortunately the resolution of this period is poor for the longer period waves (see Section 4.4 (iii)).

4.6 QUALITY CONTROL

4.6.1 General

Erroneous data contained in a data bank can seriously distort the statistical properties of the data such as the wave height distribution, especially if fictitious high waves are included in the data bank. Inspection of all the high wave events contained in the data sets at the start of this study revealed the presence of many such fictitious storms. The only quality check that was done on the early data was to check for outliers where an outlier was defined as any data point which deviated more than four standard deviations from the mean water level. Artificially high values of the standard deviation, caused by factors such as zero drift in the recorder or a multitude of outliers due most probably to radio interference, made this test inadequate and many erroneous records resulted. It was further noted that many spectra obtained from these data banks contained a high level of energy at very low frequencies, i.e. outside the frequency bands for which the Waverider is sensitive.

Since visual inspection of all the digital records was impractical, means were sought which would enable the elimination of erroneous data during the analysis process on the computer.

More detailed accounts of the quality control procedures used in this study can be found elsewhere (Rossouw et al (1982) and Visser et al (1980). Some aspects of these procedures which have a direct bearing on the results of this study are repeated here.

4.6.2 Data Qualification Procedures

The approach used in developing a data qualification procedure was to devise a large number of tests varying from crude empirical models to standard statistical tests. The outcome of these tests was compared with the judgement of experienced observers and adjustments were made until satisfactory agreement occurred.

The procedure used in the analysis of all digital data was to split the record in half, using a record length of 512s or 1024 data points for each half. Tests for stationarity based on the comparison of the results from the two halves were performed. These tests were however not used for data classification. Six tests on time domain and frequency domain parameters were performed on each half of the record to classify the record as good ("GOOD"), suspect ("BAD?") or bad ("BAD") (See Appendix A). On the basis of the outcome of each test a severity count was assigned to the test, the severity codes were then used to determine a category ("GOOD", "BAD?" or "BAD") for each half.

For the analysis used in this report all data categorised as "GOOD" or "BAD?" were considered acceptable and where both halves of the record fell into this category the mean value of the parameters from the two halves was used to represent the record. In cases where only one half of the record fell into the "GOOD" or "BAD?" category only that half of the record was used.

The following tests were used to categorise the records - see example of printout of analysis in Appendix A :

(i) Flatheads ("P2", "P3", "P4" and ">P4" in Appendix A)

A flathead is identified when consecutive $x(t_i)$ have the same numerical value (to the nearest 10mm, that is, the resolution of the digitiser). Thus the value of "P2" denotes the number of times two (and no more) consecutive $x(t_i)$ with the same value are detected. Similarly for the "P3", "P4" and ">P4" parameters. (For example, if ">P4" = 2, then two occurrences of flatheads with more than four consecutive identical $x(t_i)$ values occurred.)

(ii) Rate of Change of Wave Profile ("ERR" in Appendix A)

If the slope of two consecutive data points is greater than the theoretical maximum slope that the water surface can attain, an erratic point is identified. The value of "ERR" denotes the number of erratic points in the record.

(iii) Consecutive Erratic Points ("CON" in Appendix A)

The value of "CON" denotes the number of times two erratic points next to each other are detected.

(iv) The Sample Correlation Coefficient ("CORR" in Appendix A)

A linear trend in the data, which can be caused, for example, by zero-drift in recording instrumentation, is detected when the correlation between $x(t_i)$ and t is statistically significant.

(v) Normality of the $x(t_i)$ ("SKEWNESS" and "KURTOSIS" in Appendix A

Experience shows that the instantaneous water elevation $x(t_i)$ of sea waves follows approximately the normal distribution. Tests for normality are done using slightly adjusted distributions for skewness and kurtosis. (Relaxing slightly on the theoretical distributions.)

(vi) Low-Frequency Detector ("LF.DET" in Appendix A)

Most Waverider buoys in South Africa will respond only to waves with periods in the range 1 to 33 seconds (band widths of 0,03 to 1,0 Hz). This covers the domain for most applications. The buoy is "blind" to waves with periods outside the mentioned range (for example, the tide). Therefore, if the power spectrum contains significant spectral information outside the mentioned band width, invalid data that could not be measured with the buoy are indicated. The sum of the first four power spectral density values 0,006 to 0,035 Hz is denoted by "LF.DET".

The test criteria used to assign severity counts to the six parameters are shown in Appendix B.

4.6.3 Rejection Rate

The program "Waves", features of which have been described in Sections 4.3 to 4.6.2 above, was developed during the early stages of this study and has since become the standard program for analysis of digital prototype wave data in South Africa. Some users were critical of the quality classification procedures claiming that the program rejects too many records. Coetzee in Rossouw et al (1982) investigated the rejection rate and the reasons for rejection at two stations where large numbers of records were available. The results of this exercise are shown in Figure 4.2.

It is interesting to note that at Koeberg where the transmission distance from buoy to receiver was small and where the station was far removed from the major shipping lanes, the rejection rate was as low as 1,2 per cent. At Slangkop where the transmission distance was 70km and where a major shipping route passes between the buoy and the receiver, the rejection rate was 12,6 per cent. This seems to indicate that the quality of the radio link between the buoy and receiver had a direct bearing on the quality of the records. The deterioration of this link in stormy conditions with rough seas (see

Section 7.2) is most probably also the reason for the tendency to lose records during such conditions.

It is further of interest to note from Figure 4.2 that the low frequency detector is the major source of data rejection. The occurrence of apparent energy at frequencies lower than the recording range of the Waverider can be caused by a number of reasons including Nyquist folding of high frequency interference of the radio signal, zero drift in the instrument or the frequent occurrence of erratics. The low frequency detector therefore includes a number of the other quality tests.

Coetzee (pers.com.) also reports that a major reduction in the rejection rate was experienced at stations where a direct link to computer was established. He attributes this improvement to the elimination of the data cassettes used for the recording of the raw data. It therefore seems to be advisable to first investigate the data handling procedures before considering relaxing the quality control routines contained in "Waves".

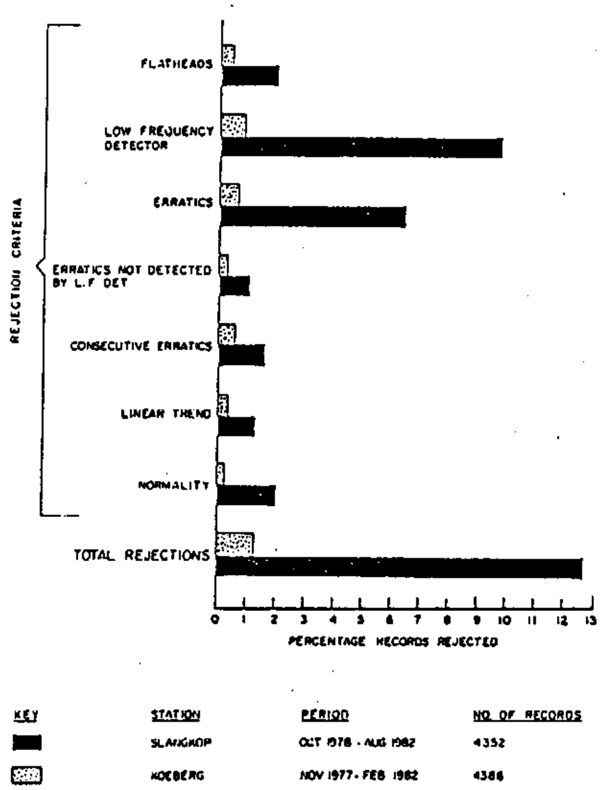


FIGURE 4.2 : WAVE DATA QUALITY ANALYSIS FOR SLANGKOP AND KOEBERG

5. AVAILABLE WAVERIDER RECORDS

A summary of the available Waverider records is given in Table 5.1 and the positions of the recording sites are shown in Figure 5.1. Only sites along relatively open stretches of coast are included. All stations where the water depth equalled or exceeded 50m can be considered reasonably representative of deepsea conditions. At the shallower stations refraction and other shallow water affects will influence the data. The emphasis in this study is on waves recorded in waterdepths exceeding 50m. Where such data are not available (eg. East of Port Elizabeth) or where gaps occur in the deepwater records, some use is made of data from shallow water recorders.

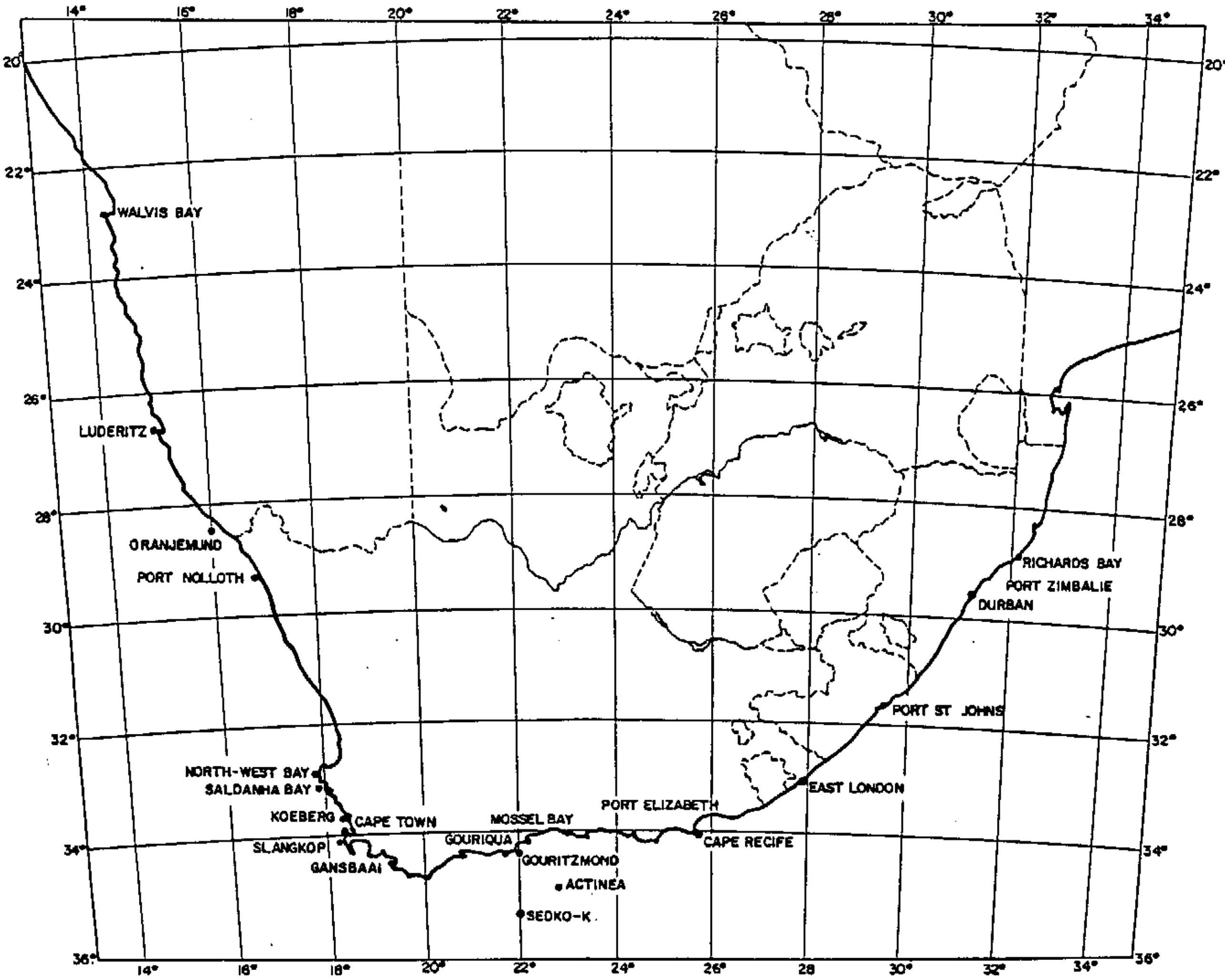


FIGURE 5.1 : LOCATION OF WAVE RECORDING STATIONS

Wave data collected prior to 1978 were mainly hand analysed to produce the parameters H_1^{10}, H_s or $H_{s_{inc}}$. Whenever digital data were available analyses by "Waves" were performed to produce H_{mo} as height parameter. Only the digital data passing the quality tests described in Section 4.6 are included in Table 5.1.

TABLE 5.1 : AVAILABLE WAVE RECORDS

Station	Position		Distance Offshore (km)	Water Depth (m)	Recording Period	Number of of Records	Percentage Coverage	Height Parameter
	E	S						
Walvis Bay	14° 23,9'	22° 53.5'	3,3	50	6/72 - 6/74	2194	75	H_1^{10}
Luderitz	14° 25,9'	24° 40.3'	8,1	108	3/73 - 10/74	2281	93	H_1^{10}
Oranjemund (old)	16° 15,6'	28° 30.4'	1,8	20	2/72 - 4/80	1429	12	H_s
Oranjemund	15° 59,5'	28° 26.1'	-	106	11/81 - 5/88	5495	58	H_{mo}
Port Nolloth	16° 49,8'	29° 17.5'	3,3	100	4/87 - 5/88	1815	89	H_{mo}
North-West Bay	17° 50,8'	32° 25.2'	3,0	35	2/83 - 12/83	1139	95	H_{mo}
Saldanha (old)	17° 52,3'	33° 07.6'	8,0	80	3/72 - 5/73	1523	85	H_s
Koeberg[*1]	18° 24'	33° 41'	1 to 3	11 to 23	6/74 - 11/80	8327	88	H_{mo}
Slangkop	18° 10,6'	34° 07.6'	14	170	3/76 - 9/88	11424	63	H_{mo}
Weathership	10°	40°	1000	Deep	4/72 - 3/74	1617	55	$H_{s_{inc}}$
Sedco K[*2]	22°	35° 30'	100	100	5/78 - 3/84	4924	58	H_{mo}
Actinea[*2]	22°	35°	70	100	5/84 - 3/86	1105	41	H_{mo}
Gouritzmond	22° 00'	34° 25.0'	8	76	8/85 - 9/88	3895	89	H_{mo}
Gouriqua	21° 43.8'	34° 23.7'	-	33	3/86 - 5/88	2273	72	H_{mo}
Mossel Bay (Old)	22° 12,9'	34° 13.5'	7	61	2/73 - 6/74	1165	60	H_1^{10}
Cape Recife	25° 42.1'	34° 0.7'	-	85	12/86 - 8/88	2206	90	H_{mo}
East London	27° 54.3'	33° 03.2'	1,2	27	2/84 - 3/85	1457	85	H_{mo}
Port Zimbali	31° 12.8'	29° 33.1'	0,7	16	9/74 - 8/75	813	56	$H_{s_{inc}}$
Richards Bay	32° 06.5'	28° 49.5''	2.6	19	1/79 - 8/88	7521	53	H_{mo}
Richards Bay (old)	32° 06.5'	28° 49.5'	2.6	19	6/73 - 5/79	5490	63	$H_{s_{inc}}$ + H_{mo}

*1 Recording position changed several times (see Rossouw (1984))
*2 Positions varied from Port Elizabeth to Mossel Bay

Data obtained by Boersma and NIO recorders and analysed by hand to produce $H_{s_{inc}}$ as height parameter are also included in Table 5.1. These data were obtained from a weathership stationed well south of the continent.

Data contained in the wave data bank of the CSIR were extracted towards the end of October 1988 and in the case of some stations data up to September 1988 are included.

At Oranjemund and Richards Bay two sets of data are listed at each station. The "old" data refer to data previously analysed and presented in Rossouw (1984). The "new" data at these stations are all analysed via "Waves" and are the sets presently contained in the CSIR data bank.

The number of records at the stations varies from 813 in the case of Port Zimbali (Ballitoville) to 11424 at Slangkop (Cape Town). This represents the equivalent of approximately half a year of data at Ballitoville to eight years of data at Slangkop. At only six stations are more than the equivalent of two years of data available.

6. THE SOUTH AFRICAN WAVE CLIMATE

6.1 INTRODUCTION

The purpose of this section is to give a background description of the weather systems that are responsible for the wave climate around the South African coast. A brief description of wave generation and decay is given whereafter the general patterns in the oceans surrounding South Africa are described. A few examples of high wave events and the spatial distribution of wave heights recorded along the coast are given. This background information is required to explain some of the phenomena that are discussed in later sections of this study where design wave conditions are derived from recorded data.

6.2 WAVE GENERATION AND DECAY

Only wind generated waves are considered here. The height of the generated waves are influenced by three factors, i.e. the wind speed, the duration of the wind over a given wave field and the fetch length. The size of the waves arriving at the coast is further influenced by the decay distance, i.e. the distance between the generating area and the point of interest.

To develop a feel for the relative importance of wind speed, duration, fetch and decay distance on the waves recorded along the coast a few very simple examples are given below. These examples are based on the deepwater wave forecasting equations developed by Sverdrup and Munk (1947) and revised by Bretschneider (1958) and Hasselmann et al (1976) as contained in the Shore Protection Manual (U S Army 1984). These examples are chosen to represent conditions which would provide wave heights consistent with the highest wave heights yet recorded along the South African coast and are needed to illustrate which parameters are limiting the extreme wave events along the coast.

The influence of wind speed on the generated wave height, if there is no limitation on fetch or duration, is illustrated in Figure 6.1(a). To increase the significant wave height (H_s) from 10m to 12m (i.e. by

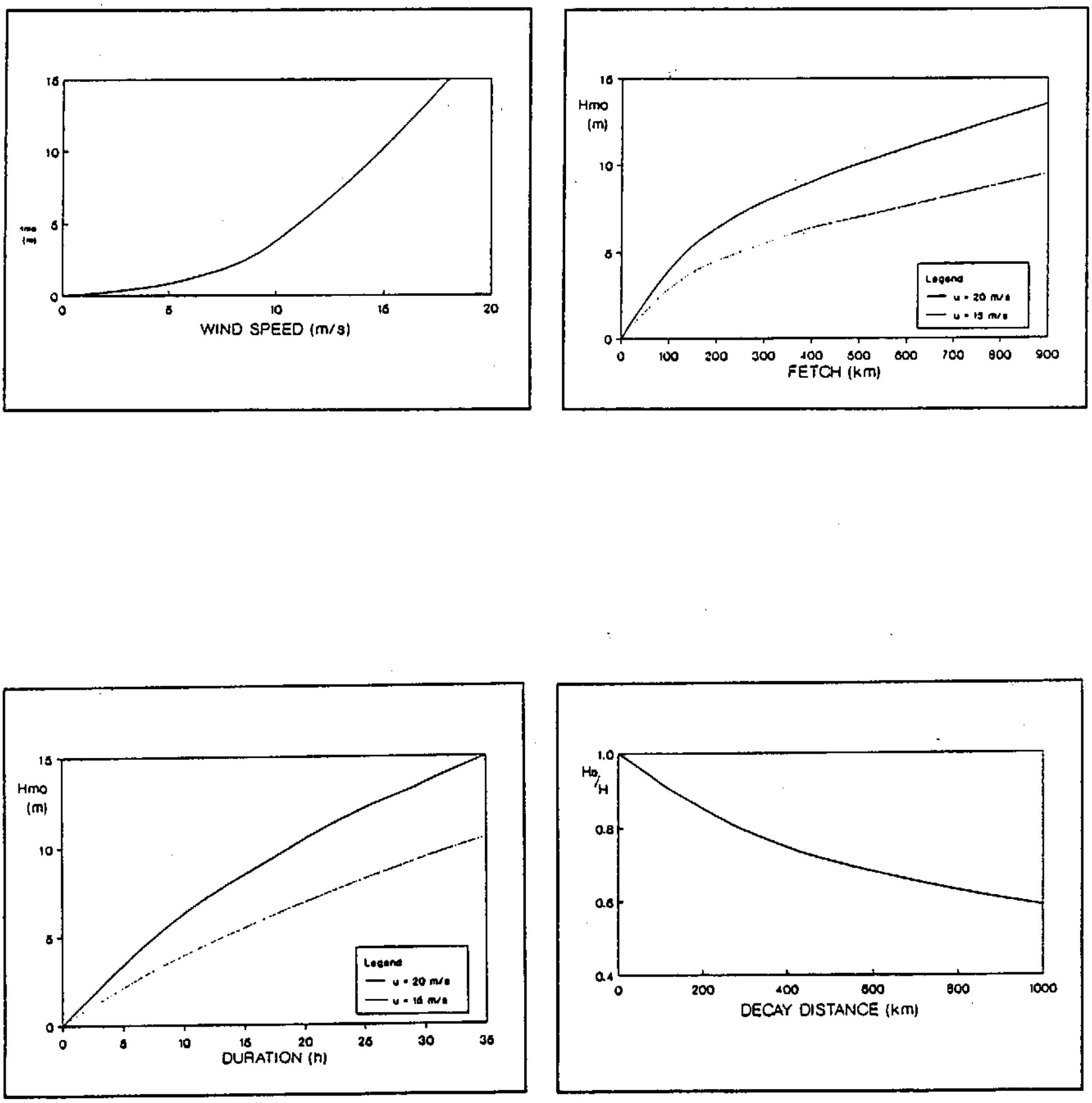


FIGURE 6.1 : INFLUENCE OF WIND SPEED, DURATION, FETCH AND DECAY DISTANCE ON WAVE HEIGHT

20 per cent) an increase in wind speed from 15,2 to 16,5m/s (i.e. of 9 per cent) is required. This sensitivity to wind speed is one of the major sources of inaccuracy in wave forecasting and hindcasting.

In Figure 6.1(b) the influence of fetch on wave height for steady wind speeds (u) of 15m/s and 20m/s and an unlimited duration is given. To increase the significant wave height from 10m to 12m an increase in fetch from 475 to 700km is required if u = 20m/s.

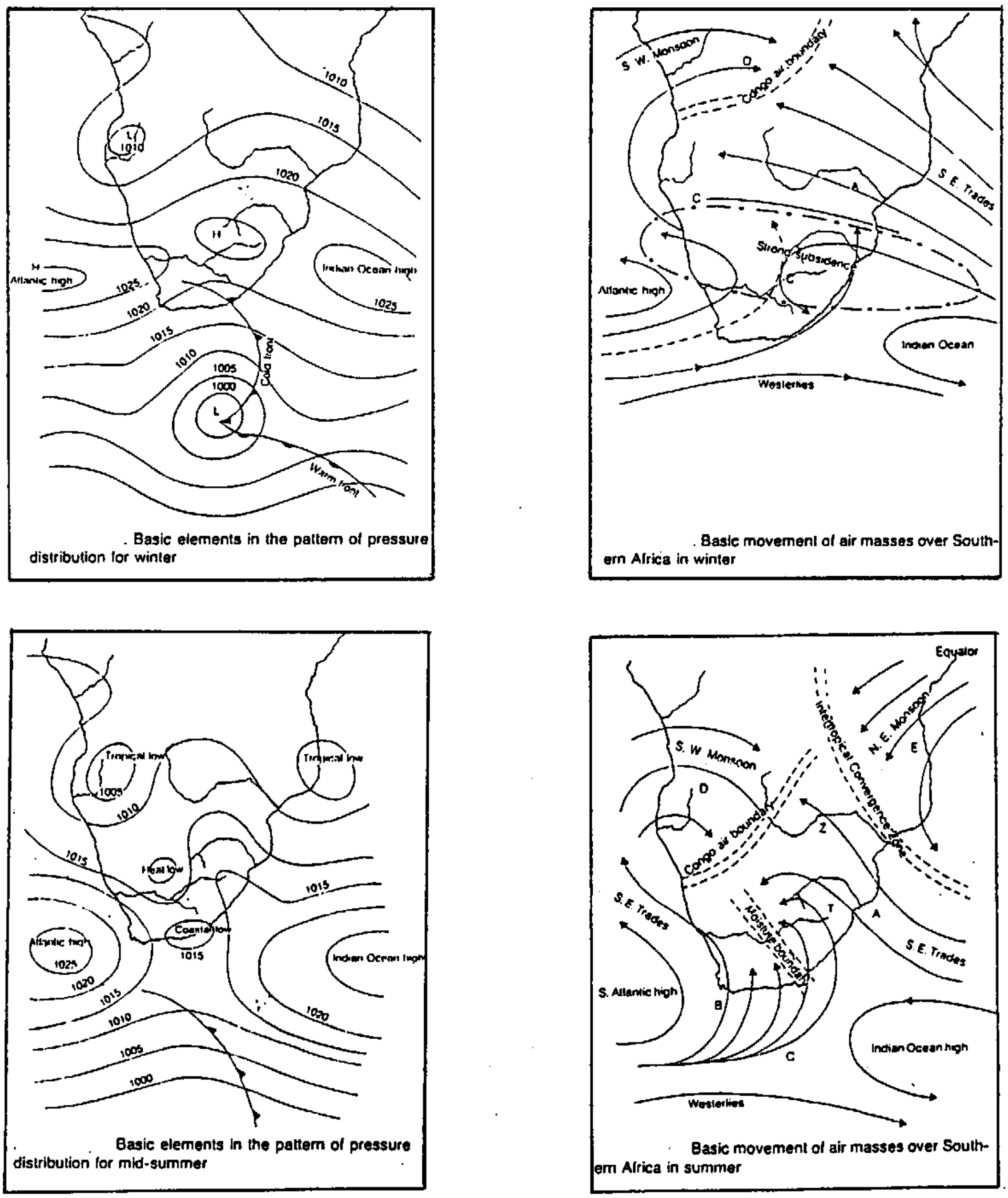
The increase of wave height with increasing duration for steady wind speeds of 15m/s and 20m/s and an unlimited fetch is shown in Figure 6.1(c). To increase H_s from 10m to 12m the duration must be increased from 18 hrs to 24 hrs when u = 20m/s.

It is further important to realise that the initial decay of wave height is rather rapid once the wave leaves the generating area. This is illustrated in Figure 6.1(d) where it can be seen that a decrease of 20 per cent in wave height will occur within the first 250km of the wave leaving the generating area.

6.3 WEATHER PATTERNS

The wind and therefore the wave patterns in the South Atlantic and South Indian oceans are influenced by a number of dominant meteorological features. Heated air which rises in the tropics near the equator moves southward and descends in the vicinity of 30°S to form the so-called Hadley cell. This descending air causes two semi-permanent high pressure systems, the South Atlantic high and the South Indian high, with the air moving in an anti-clockwise rotation around the centre of the high pressure system (see Figure 6.2). South of the Hadley cell the Ferrel westerlies spiral eastwards around the globe. Disturbed air in the Ferrel westerlies creates the low pressure systems of the South Atlantic. Once formed these low pressure systems are moved from west to east within the Ferrel westerly wind system. It is the passage of these depressions with their associated cold fronts that are the main source of large waves affecting the South African coastline.

These cold fronts pass the southern tip of Africa with great regularity at intervals of 3 to 5 days. In winter the path of these depressions is frequently intersected by the southern tip of the African continent. In summer the entire system shifts further south and the depressions mostly pass south of the continent. High waves can therefore be expected to occur more frequently in winter along the southern Cape coast. The occasional northerly excursion of a cold front does however occur in summer resulting in occasional high waves along this coast during this season as well.



. Basic elements in the pattern of pressure distribution for winter

. Basic movement of air masses over Southern Africa in winter

Basic elements in the pattern of pressure distribution for mid-summer

Basic movement of air masses over Southern Africa in summer

FIGURE 6.2 : PRESSURE AND WIND SYSTEMS OVER SOUTHERN AFRICAN OCEANS (Hurry and Van Heerden (1982))

In general these cold fronts move rather rapidly, i.e. at speeds well in excess of the group velocity of the waves. Furthermore the wind direction normally swings from NW through SW to SE during the passage of a cold front past the southwestern and southern Cape coast. These two factors often result in wave conditions which are duration limited and fully arisen seas seldom develop.

Another possible source of high waves along the eastern extremity of the South African coast is the presence of tropical cyclones. In contrast with the cold fronts of the southern Cape, the occurrence and the paths of these cyclones are erratic and unpredictable. The system is invariably much smaller and more intense than those associated with the cold fronts. In Figure 6.3 the most frequently occurring cyclone tracks of the world are shown. From this figure it can be seen that the only part of the South African coast which is influenced by these tropical cyclones is the northern Natal coast. At most only a few of these cyclones influence the wave conditions along this part of the coast each year. These cyclones can however cause very high waves and should not be ignored when design wave conditions for this coast are considered.

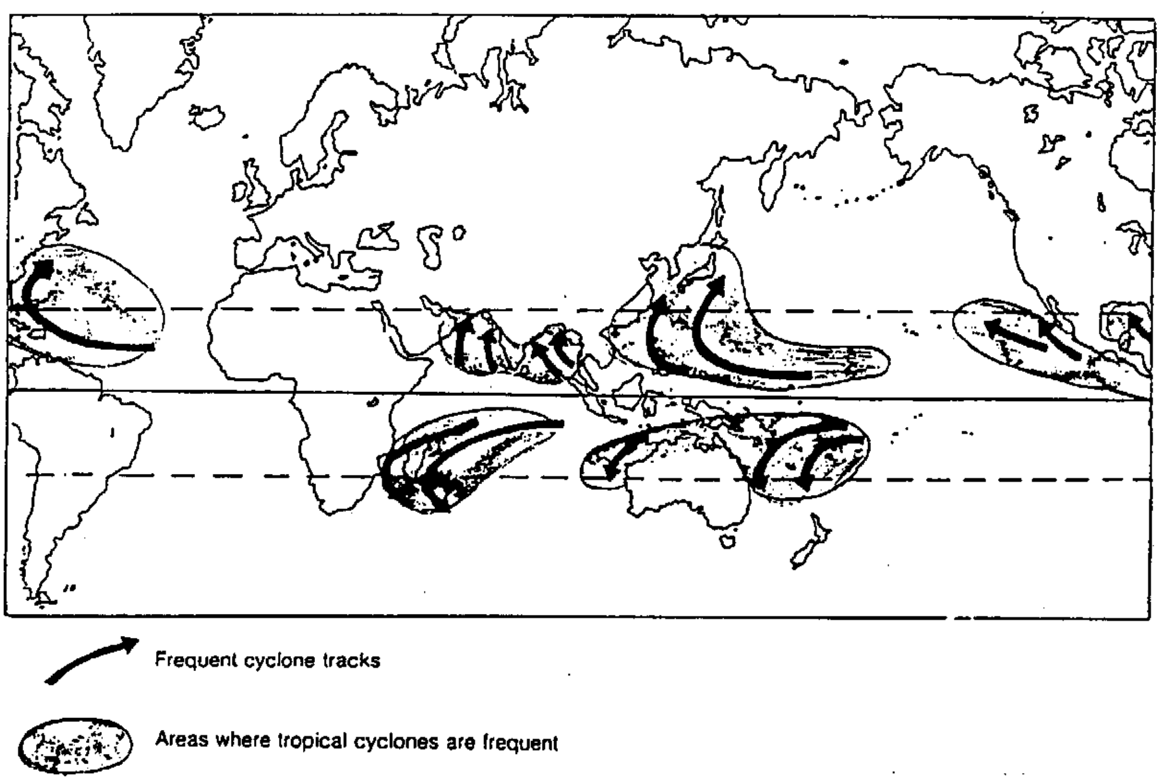


FIGURE 6.3 : FREQUENT CYCLONE TRACKS OF THE WORLD

(Hurry and Van Heerden (1982))

In Figure 6.2 the wind patterns dominating the South African wave climate are summarised. From the doldrums near the equator to approximately 30°S along the West coast, the constant southeasterly trade winds blow away from the coast. Although these southeasterlies are responsible for high waves further offshore (Quale and Elms (1979)), these waves propagate mainly away from the land.

Between the trade winds in the north and the constant westerlies far south of the continent in the so-called "roaring forties", a belt of changing winds occurs. The depressions and their associated cold fronts occurring in this area are the main source of high waves along the entire West and South coast from Oranjemund to Port Elizabeth. The general path of these cold fronts passes just South of the continent and a decrease in wave height can therefore be expected in a northerly direction along this part of the coast. Wave directions associated with the passage of the cold fronts vary from NW to SW on the more southerly locations with a reduction in the NW component in a northerly direction.

The intensity of the cold fronts normally reduces east of Port Elizabeth. The presence of the South Indian high also tends to deflect the cold fronts away from the coast in this area. These two factors combined with a swing in the direction of the coastline in a more SW-NE direction have the affect that the influence of the regular cold fronts is much less pronounced east of Port Elizabeth. Along this part of the coast all the way up to the northern boundary of the South African coast the wave fields are influenced by much smaller systems such as coastal lows. A general reduction in wave heights can therefore be expected east of Port Elizabeth.

6.4 HIGH WAVE EVENTS RECORDED ALONG THE SOUTH AFRICAN COASTLINE

The purpose of this section is to give a few examples of major storms which were recorded along the South African coastline. This will mainly serve to illustrate the influence of the weather patterns described in Section 6.3. Knowledge obtained from these storms will also be used to rationalise the answers obtained from the statistical extrapolation of the data later in this study.

6.4.1 Storm of 16 May 1984

The synoptic charts shown in Figure 6.4 illustrate the typical pattern of an intense winter storm along the Cape SW coast.

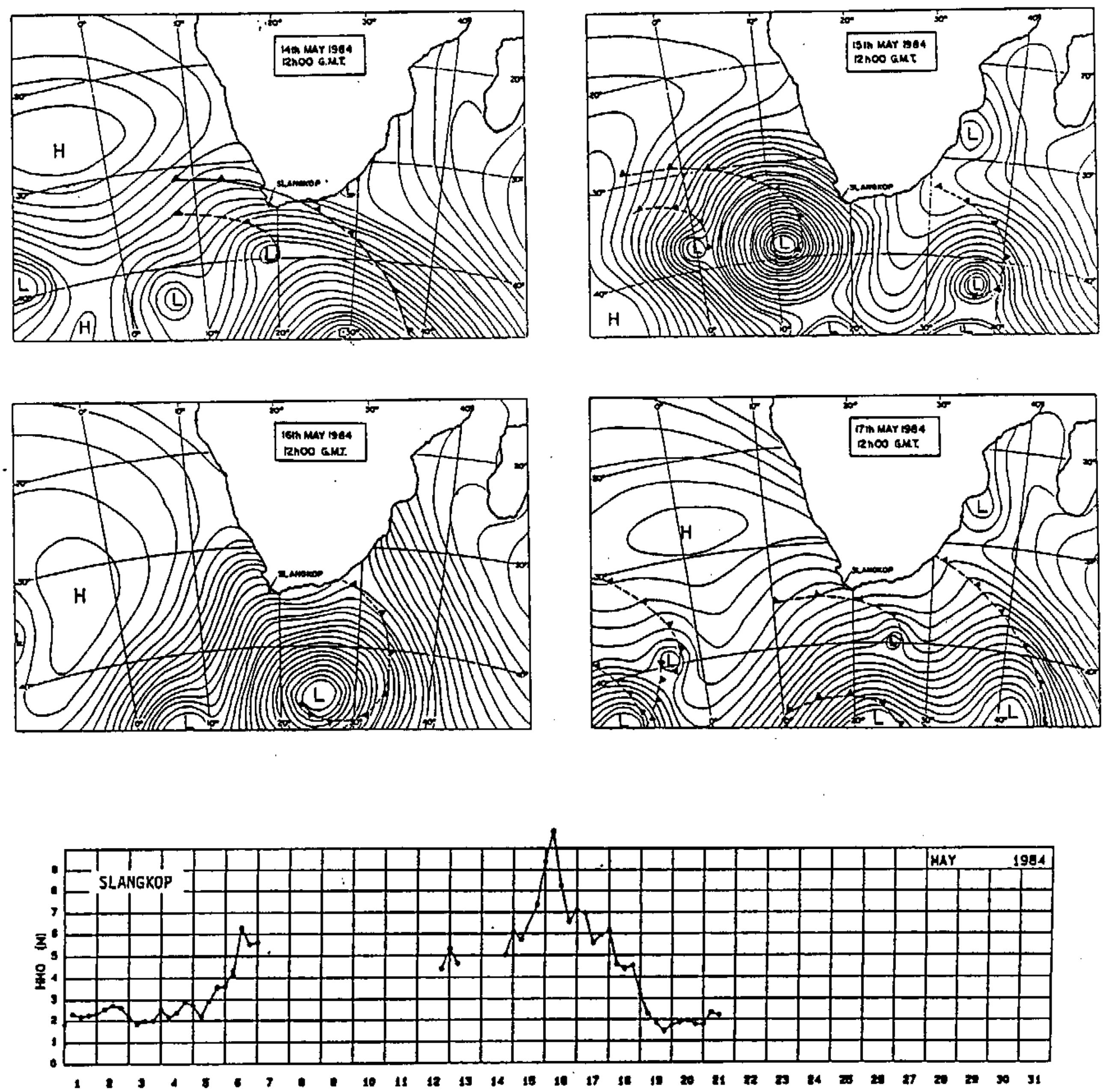


FIGURE 6.4 : STORM OF 16 MAY 1984

At 12h00 GMT on 14 May 1984 the typical winter pattern with a well developed Atlantic and Indian Ocean high and a series of low pressure systems around 40° to 45°S are seen. Twenty-four hours later (15 May) one of these low pressure systems has intensified dramatically with it's centre at 11°E and 38°S. Strong north-westerly winds are generating waves towards the Cape south-western coast. A further twenty-four hours later (16 May), the intense low has moved approximately 1300km towards the SE and is now situated at 25°E and 45°S. During this period the winds generating waves towards the Cape south-west coast have changed from NW to SW. It is important to note the large size of the system and the speed at which the system moved. A very simple analysis indicates that the wave heights in this storm were duration limited. Not only did the cold front move at more than twice the group velocity of the waves but a 90° swing in wind direction severely limited the time for wave generation.

It should be noted that the wave heights recorded during the peak of this storm were the highest yet recorded in South African waters, i.e. H_{mo} = 10,6m was recorded at Slangkop at 06h00 on 16 May 1984. A more complete description of this storm can be found in Pos (1985) and Jury et al (1986).

6.4.2 Storm of 2 September 1978

This particular storm was exceptional in that the duration of the winds generating waves towards the southern Cape coast were much longer than in the more typical storms for this part of the coast. The reason for this was the formation of a blocking high in the south-west Indian Ocean (Shillington et al 1979) which retarded the movement of the low pressure system towards the east. The wave direction during this storm was also exceptional in that it came from the south compared to the more frequent south-westerly direction normally associated with the storms in this area. This is illustrated on the synoptic charts of Figure 6.5.

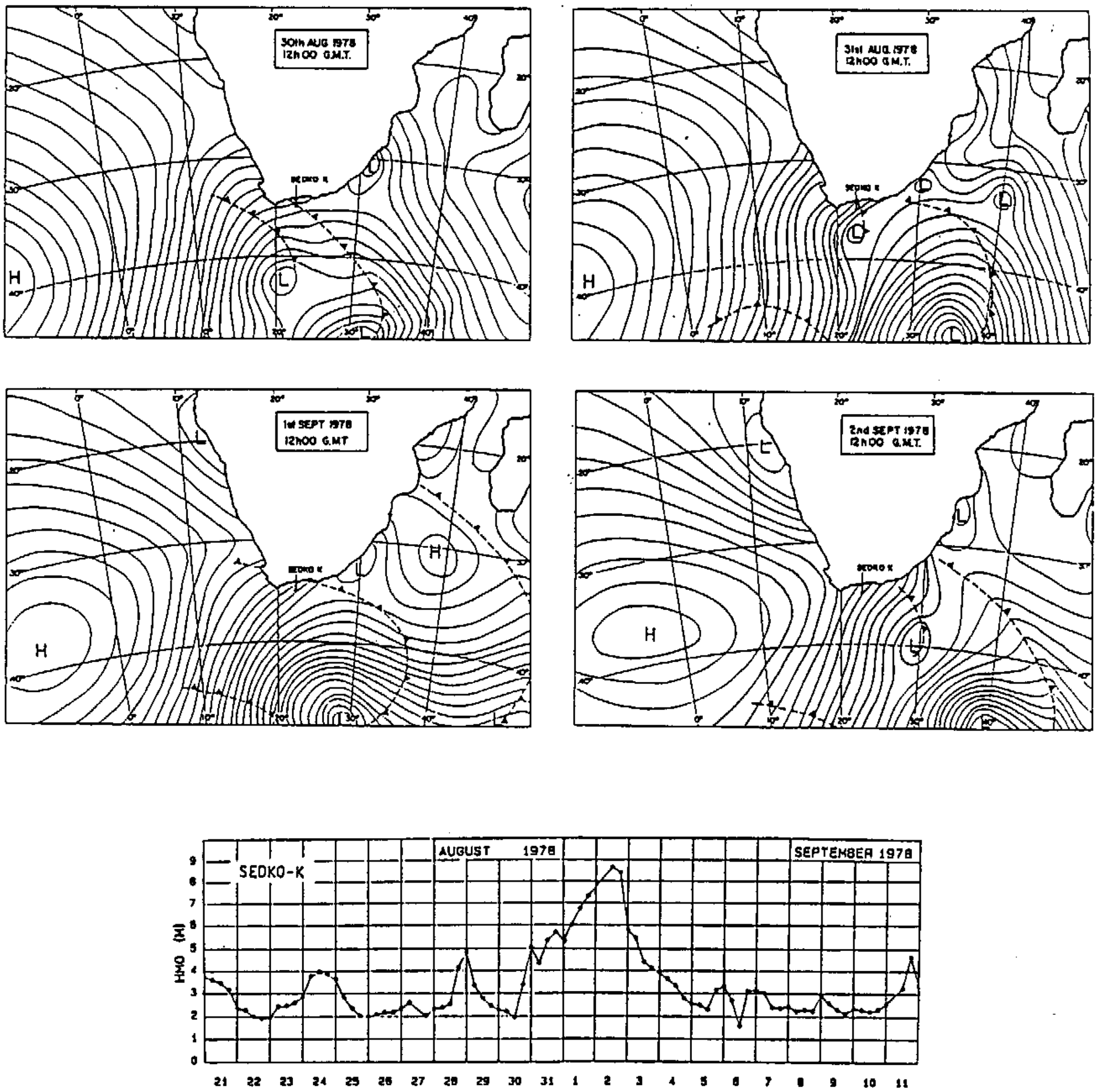


FIGURE 6.5 : STORM OF 2 SEPTEMBER 1978

A wave height ((H_{mo}) of 8,7m was recorded off Port Elizabeth on 2 September 1978 at 12h00 GMT during this storm. The direction of this storm was also much more southerly than normal.

6.4.3 <u>Storm of 16 December 1978</u>

To illustrate that severe storms do also occur in summer, the synoptic charts at 12h00 GMT on 15 and 16 December 1978 are given in Figure 6.6.

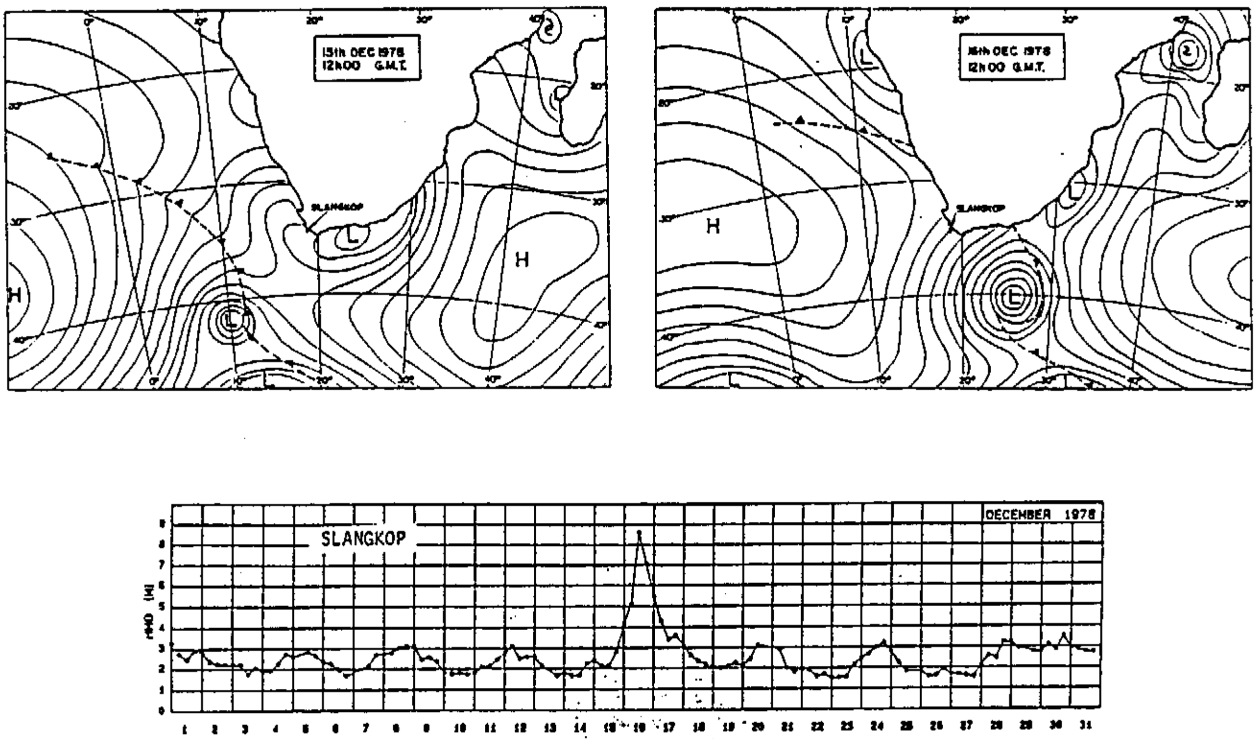


FIGURE 6.6 : STORM OF 16 DECEMBER 1978

An intense low pressure system for this time of the year passed south of the Cape with its centre at approximately 40°S. Strong southwesterly winds associated with this system generated waves towards the Cape and a significant wave height of 8,7m was recorded at Slangkop at 12h00 on December 16 during this storm. This storm was again duration limited with the wave heights increasing from 5m to 8,7m and falling to below 5m again within a 24 hour period.

6.4.4 <u>Storm of 19 February 1984</u>

An example of a tropical cyclone which caused high waves along the Natal north coast is given in Figure 6.7. The tropical cyclone IMBOA formed in the Mozambique channel and slowly moved southwards.

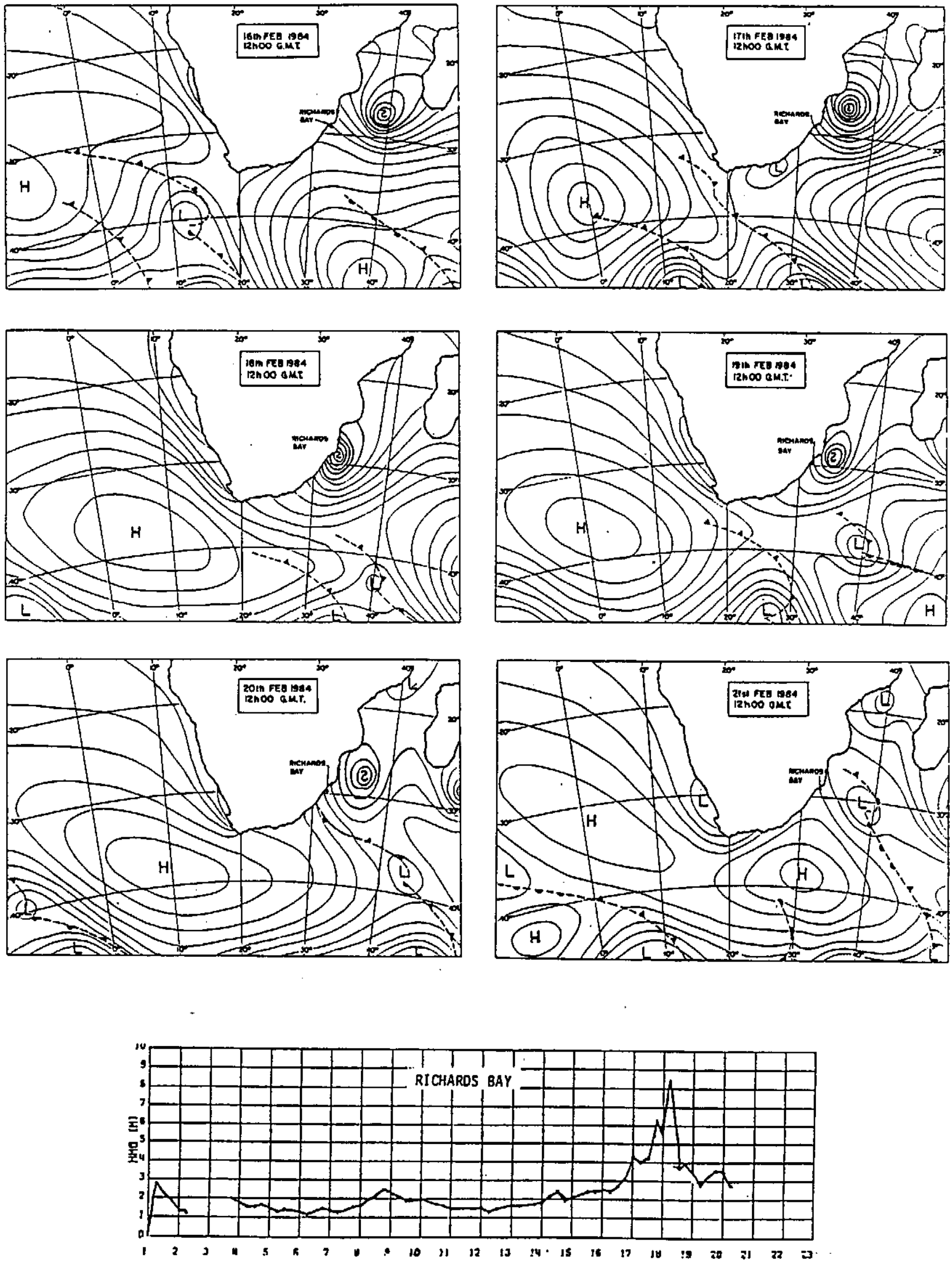


FIGURE 6.7 : STORM OF 19 FEBRUARY 1984

It stayed east of Maputo for a few days but moved further south to approximately 28°S and reached Richards Bay on 18 February 1984. Wave heights (H_{mo}) in the order of 8m were recorded by the Richards Bay Waverider at around 03h00 during the peak of this storm. This is by far the highest wave recorded off the Natal coast to date.

6.5 SPATIAL VARIATION IN WAVE HEIGHT AROUND THE COAST

6.5.1 General

In Section 5 all the available wave height data for the South African coast were summarised. Data coverage at all the sites was intermittent with data being lost due to a variety of reasons such as loss or malfunction of the Waverider system, instruments being removed for service and calibration, etc. To see how the passage of the cold fronts influence the wave conditions around the coast, the spatial variation in wave height was studied by comparing the simultaneously recorded wave heights at the stations where such simultaneous records were available. In doing this study several characteristics of the wave pattern around the coast became apparent:

(i) The passage of major cold fronts influences the wave heights from Oranjemund on the northern extremity of the west coast to approximately Port Elizabeth on the south coast and very similar wave patterns are recorded at all open coast stations along this stretch of coastline.

(ii) East of Port Elizabeth the influence of the cold fronts is much less pronounced and often not even discernible. Along this part of the coast local and often much smaller meteorological systems influence the wave patterns.

The similarity in wave pattern along the entire coast west of Port Elizabeth was of considerable interest to this study since a proper understanding of the relationship between the simultaneously recorded wave heights at these stations would enable the composition of a near continuous wave record along this entire part of the coast from the intermittent records available at the various recording sites. Several factors, however, hampered this study. The most important of these was that invariably during major wave events very few simultaneously recorded wave data were available. The reason for this was partly because of the normally intermittent nature of the recordings but more serious was the tendency of the Waveriders to malfunction during major wave events (see also Section 7.2).

Because of the shortcomings in the data it was not possible to conduct a systematic study of the spatial variation in wave height for all major events. In this section examples are shown of the similarity in wave height patterns at stations between Port Nolloth and Port Elizabeth. This information is then utilised in Section 7 where the methodology of determining design wave heights from recorded data is investigated. The examples chosen are based more on the availability of simultaneous records than on the severity of the events.

6.5.2 Simultaneously Recorded Wave Data for May 1987

The H_{mo} values recorded during May 1987 at Port Nolloth, Gouriqua and Cape Recife are shown in Figure 6.8. This figure illustrates very clearly the similarity in wave height patterns along this entire part of the coast. The synoptic charts for 13, 14 and 15 May are also shown in Figure 6.8 and illustrate a typical response to the passage of a cold front from west to east across the southern part of the continent. In this case the wave heights start increasing virtually simultaneously at all three stations on 14 May but the peak of the storms occur one day later at the two more easterly stations.

6.5.3 Simultaneously Recorded Wave Data for August 1980

Wave heights recorded during August 1980 at Slangkop and Sedco K (off Mossel Bay) are shown in Figure 6.9. This is a very good example of the similarity in wave heights often recorded at these

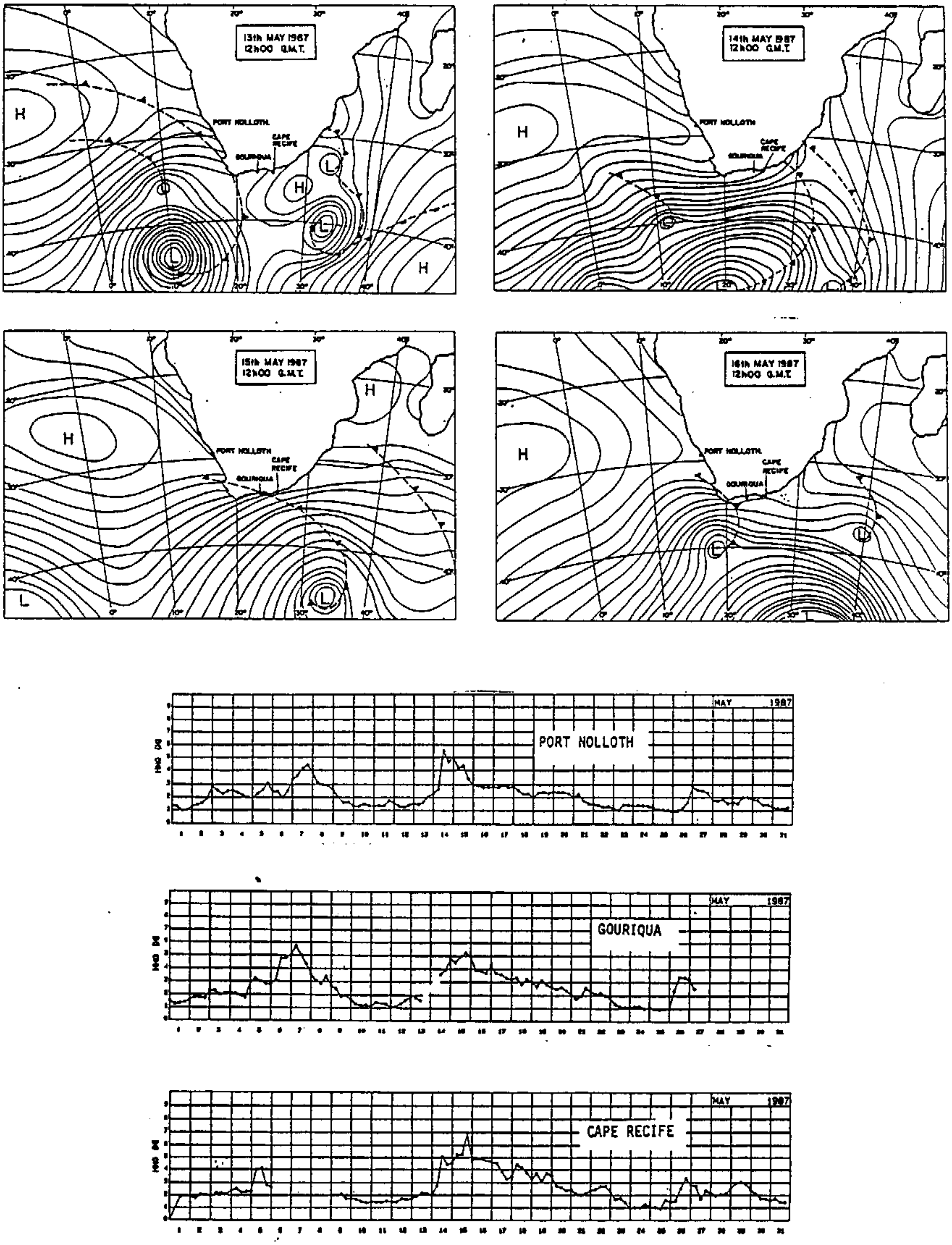


FIGURE 6.8 : WAVES RECORDED DURING MAY 1987

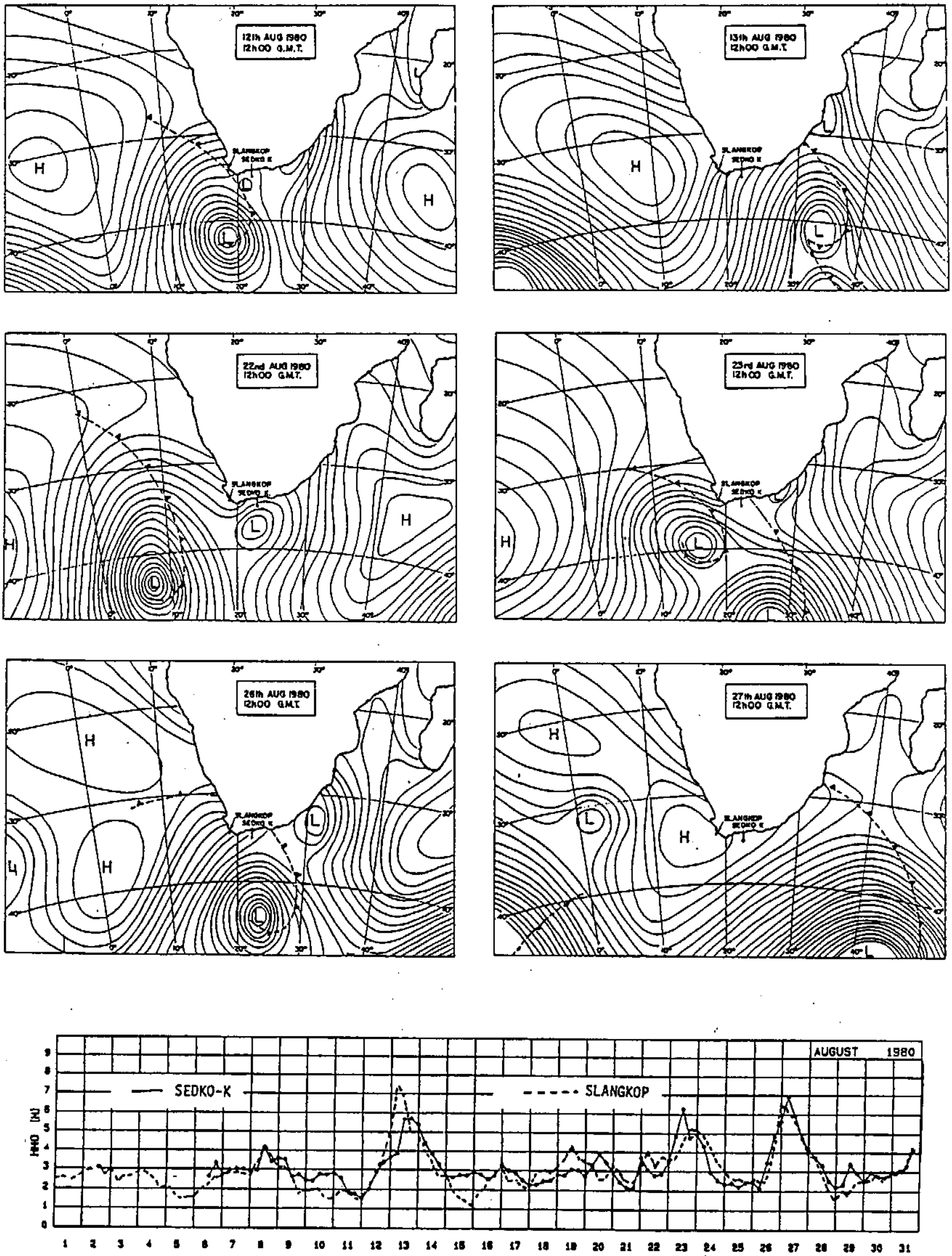


FIGURE 6.9 : WAVES RECORDED DURING AUGUST 1980

two sites. The synoptic charts illustrating the three major wave events during this month are also shown in Figures 6.9. For all three events the low pressure systems and the associated cold fronts moved rapidly on their easterly path and therefore very little delay in the response of the waves at the two stations was experienced. On 13 August the wave heights at the peak of the storm were 1,6m higher at Slangkop than at Sedco K, on 23 August Sedco K peaked at a 1m higher value whereas on 27 August very similar wave heights were recorded at both stations.

6.5.4 Simultaneously Recorded Wave Data for May 1986

The wave heights recorded during May 1986 at Slangkop and Gouritzmond are shown in Figure 6.10.

The similarity of the recorded wave heights is again evident especially for the major event on 12 May. This event again stems from the typical movement pattern of the cold fronts (see Figure 6.10).

To illustrate that the regular pattern does not occur all the time, it is interesting to look at the minor event at Slangkop on 28/29 May which is totally absent from the Gouritzmond record. The synoptic charts for this event (Figure 6.10) show that when the centre of the low pressure system occur further northwards than normal, the waves are generated mainly from NW. These waves are recorded off the west coast (i.e. at Slangkop), but at Gouritzmond the winds are offshore, resulting in calm conditions. The normal pattern where the winds swing from NW to SW during the passage of the front is not present on this occasion with the result that the waves never increased along the south coast during this storm.

6.5.5 Simultaneously Recorded Wave Data for May 1984

The storm of 16 May 1984 has already been described in Section 6.4.1 and the relevant synoptic charts were shown in Figure 6.4. The

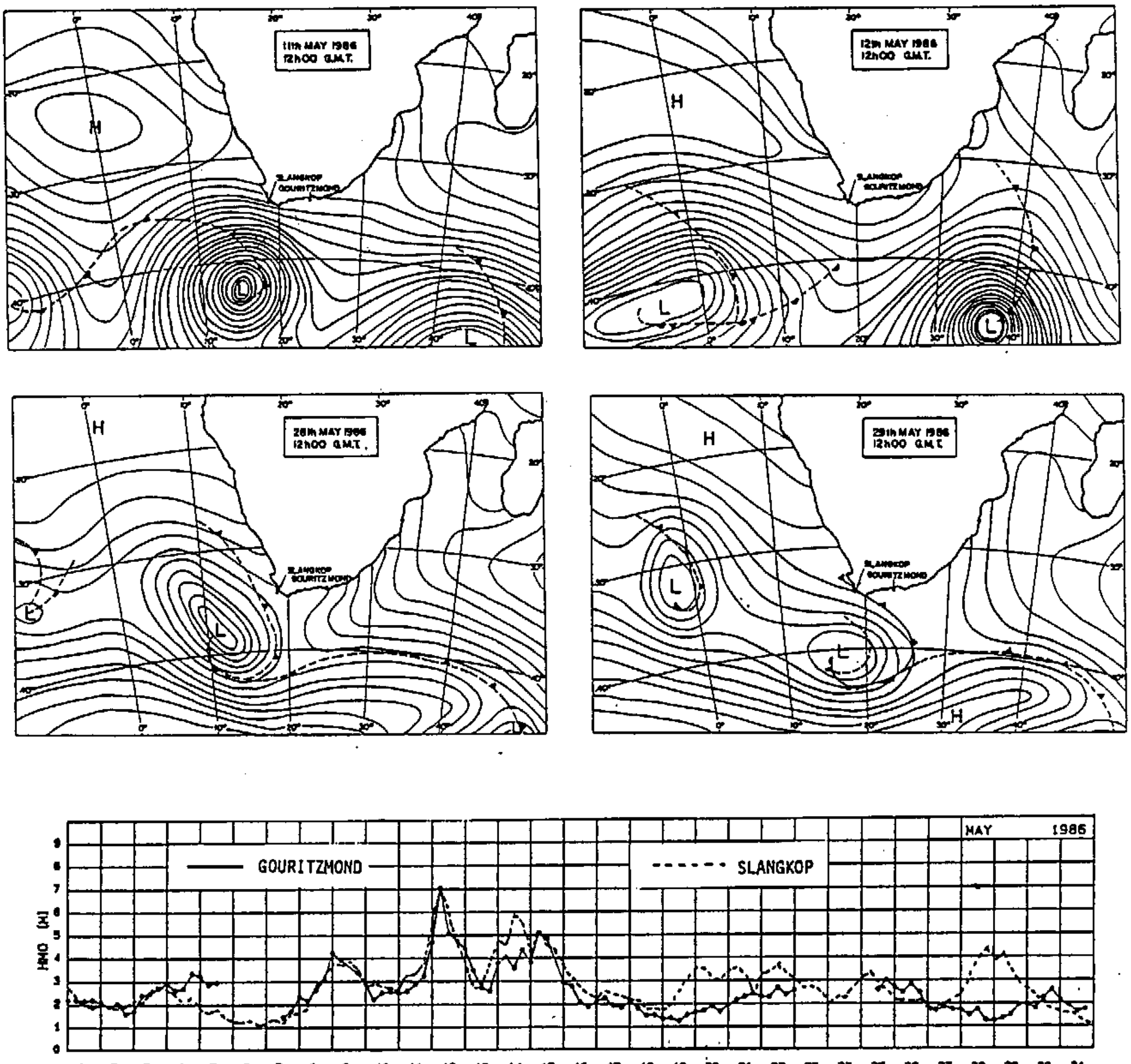


FIGURE 6.10 : WAVES RECORDED DURING MAY 1986

reason for taking another look at this storm is to study the pattern of wave heights recorded around the coastline. Unfortunately, as with most of the more intense storms, not many of the Waveriders were functioning properly. In Figure 6.11 the simultaneously recorded wave heights at Slangkop, Oranjemund and East London are shown.

The Slangkop Waverider only operated intermittently during this month. It is interesting to note that wave heights followed similar patterns at Slangkop and Oranjemund, the height at Oranjemund however being much lower than at Slangkop due to its greater distance from the intense eye of the storm. The peak of the storm occurred twelve hours later at Oranjemund than at Slangkop, i.e. at 18h00 GMT when the wind generating waves towards Oranjemund had already swung to the SW.

It is further very interesting to note that the waves at East London never responded to this storm.

6.5.6 Simultaneously Recorded Wave Data for September 1984

The purpose of looking at this month is to illustrate the lack of similarity in wave height pattern between East London and the other more westerly stations. Wave heights recorded in September 1984 at Oranjemund, Slangkop, Actinea (off Mossel Bay) and at East London are shown in Figure 6.12.
Again the similarity between the Slangkop and Mossel Bay areas is evident as well as the total lack of correspondence between East London and Mossel Bay. The biggest event at East London during this month occurred on 26 September due to a local coastal low (see Figure 6.12). This event was totally absent at Slangkop.

6.6 CONCLUSION

The wave conditions along the South African west and south coasts between Oranjemund and Port Elizabeth are totally dominated by the regular passage of cold fronts from west to east, past the southern tip of the continent.

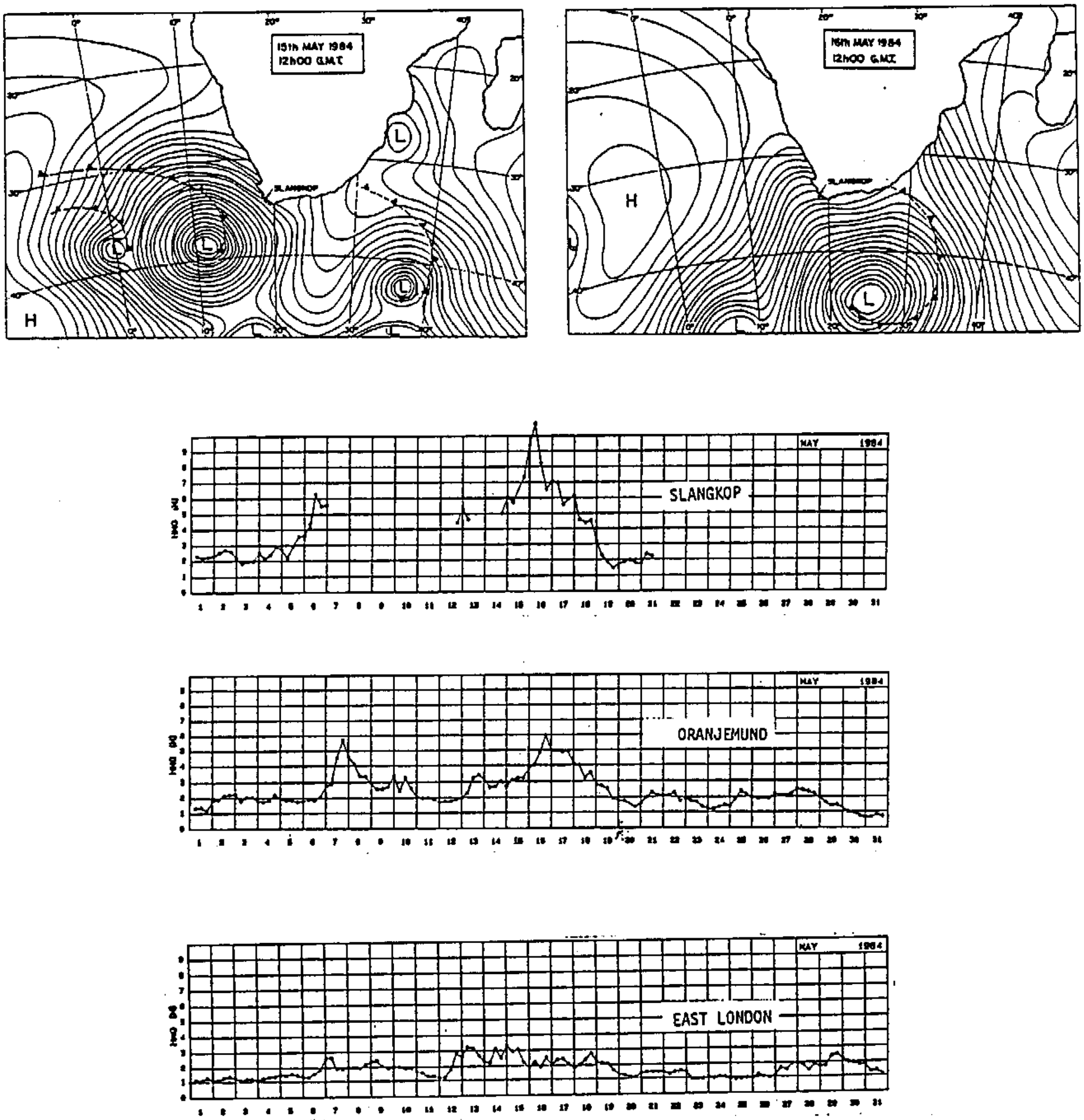


FIGURE 6.11 : WAVES RECORDED DURING MAY 1984

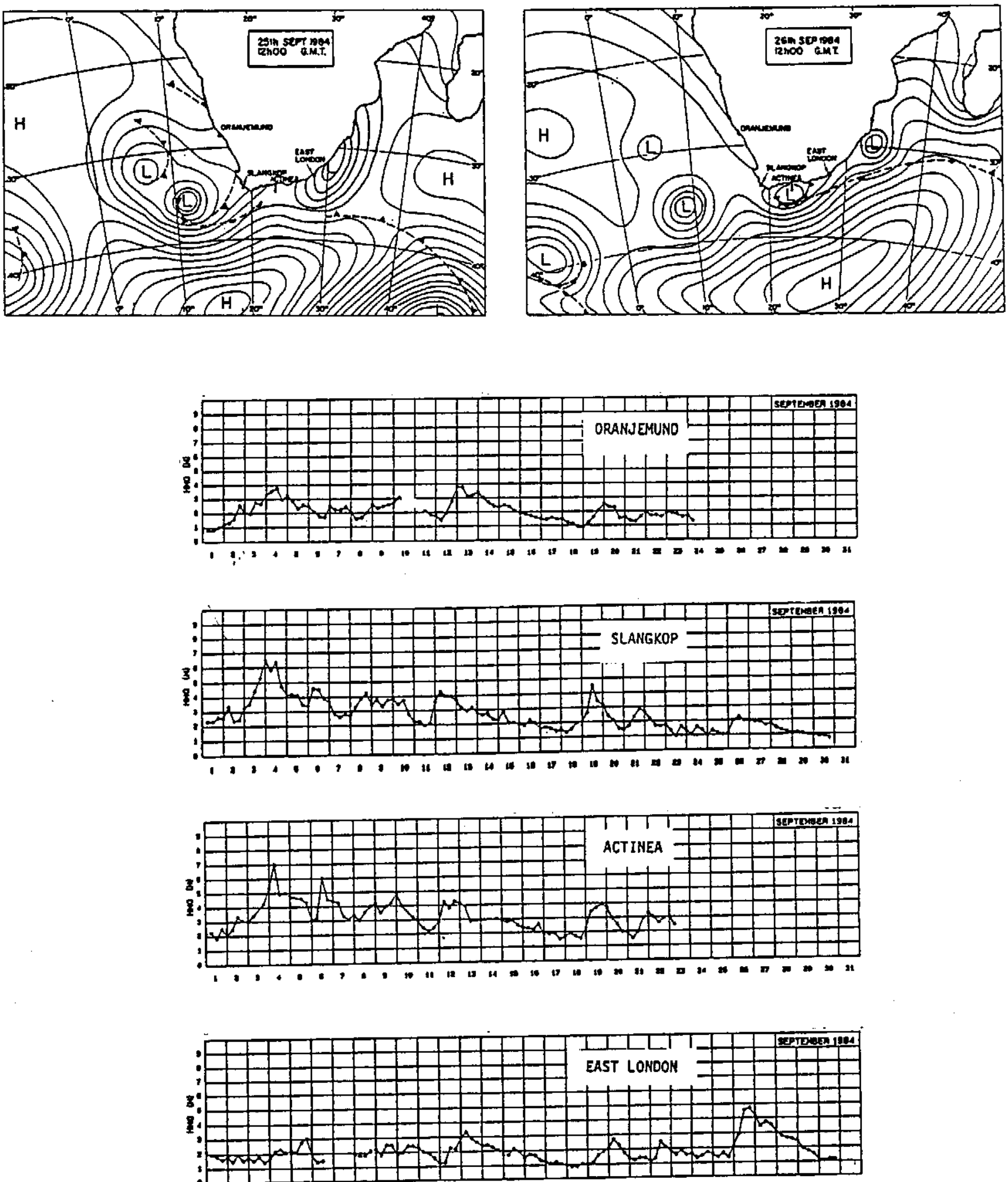


FIGURE 6.12 : WAVES RECORDED DURING SEPTEMBER 1984

The wave patterns recorded during these events show very similar wave heights occurring along the Cape Town - Port Elizabeth stretch of coast with a reduction in height towards the north along both the west and east coasts. The reduction in height up the west coast is much more gradual than that along the east coast where a sudden change in wave climate occurs between Port Elizabeth and East London.

Tropical cyclones are shown to occasionally cause high waves along the northern Natal coast.

7. DETERMINATION OF DESIGN WAVE HEIGHTS

7.1 INTRODUCTION

The prediction of design wave heights from recorded data is a subject that has received the attention of many researchers over the past 40 to 50 years and hundreds of publications on the subject are available. Despite all this attention no agreement has been reached on the methodology to be used for this purpose. The main reason for the controversy is that insufficient data are available to be able to reach firm conclusions on aspects such as the long term distribution of significant wave height, possible long term changes in this distribution, etc. The problem is normally that a few years of available data have to be used to predict the largest waves to be expected in the next hundred years or more. Even the best recorded data sets in the world seldom cover a period of more than ten years.

The approach that is adopted in this study is to systematically work through each step in the process of design wave height determination and to determine the sensitivity of the answers to each assumption that is made along the way. The focus is very much on the methodologies used in South Africa and how these methodologies can be improved.

The process that is followed to investigate the various steps involved in the estimation of design wave heights is as follows :

(i) The best available data set for the South African coast is selected.
(ii) Methods of sampling from this data set to ensure independent and identically distributed samples are investigated. The influence of the method of sampling on the estimates of design wave heights is tested.
(iii) A comparison of the three most popular methods of estimating the parameters of the distributions describing the long term distribution of wave height is made.
(iv) Methods of selecting the most appropriate model for describing the long term distribution of wave height is tested.
(v) The various uncertainties involved in the process and the influence thereof on the reliability of the answers is discussed.

Based on the above, recommendations about an appropriate methodology for the South African situation are made. This method is then applied to all available wave data and the pattern emerging from this exercise is compared to the pattern expected from the weather systems described in Chapter 6.

7.2 SELECTION OF A DATA SET FOR DEVELOPING METHODOLOGY

For the purposes of estimating design wave heights from recorded data it is important to have data that covers the longest possible time span with as few gaps as possible in the data. The available data at the various recording sites in South Africa, described in Section 5, were either of short duration or contained many gaps in the records. The similarity in simultaneously recorded wave heights from Slangkop in the west to Port Elizabeth in the east, which was described in Section 6, afforded the opportunity to combine data from various stations to extend the duration and coverage of the record. For this purpose the data recorded off the oil drilling platforms Sedco K and Actinea, operating mainly off Mossel Bay, were used as a basis. Gaps in the records at these stations were filled by using data collected at Slangkop and Gouritzmond, in that order of preference. In this way a total of 10537 records were obtained for the period June 1978 to May 1986, giving a data coverage of approximately 90 per cent over this eight year period. The combined data set consisted of

	6227	Sedco K and Actinea records off Mossel Bay
	322	Sedco K records off Port Elizabeth
	3389	Slangkop records
	589	Gouritzmond records
TOTAL	10537	

This combined data set is referred to as Agulhas Bank data.

Examples of the similarity in simultaneously recorded wave heights at these stations were given in Section 6.5. A more formal comparison of the Sedco K and Slangkop data for the period October 1978 to August 1980 was presented by Coetzee in CSIR (1981).

The scatter diagram of simultaneously recorded wave heights at these two stations as well as the correlation coefficients between the heights as a function of lag, as produced by Coetzee, are reproduced as Figures 7.1 and 7.2 respectively.

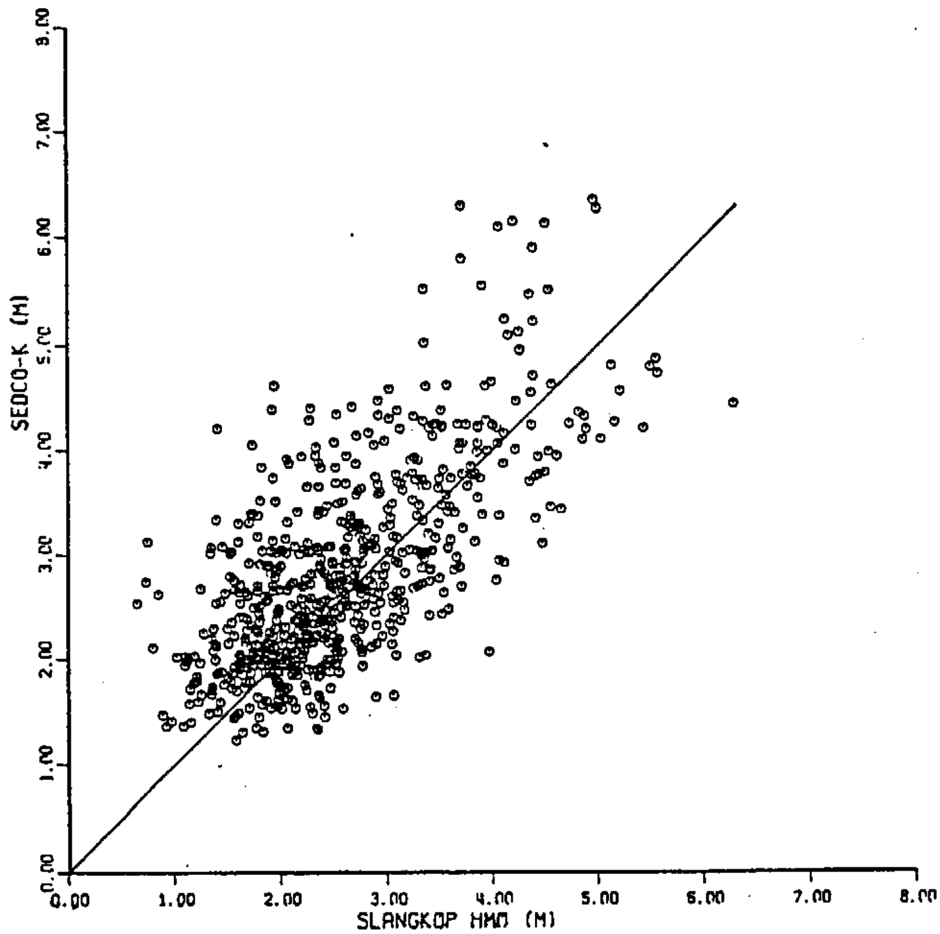


FIGURE 7.1 : COMPARISON OF H_{mo} AT SLANGKOP AND SEDCO K (CSIR 1981)

As can be seen in Figure 7.1 quite a large scatter in simultaneously recorded heights occurs with a tendency for slightly higher heights at Sedco K. Efforts by Coetzee to improve the height correlation by shifting the time series relative to each other (Figure 7.2) were unsuccessful and the highest correlation was found to occur at a lag of 0. The fact that the correlation reduces more quickly when the lag between Slangkop and Sedco K is negative obviously indicates that there is a tendency for the waves to increase at Slangkop before they do at Sedco K.

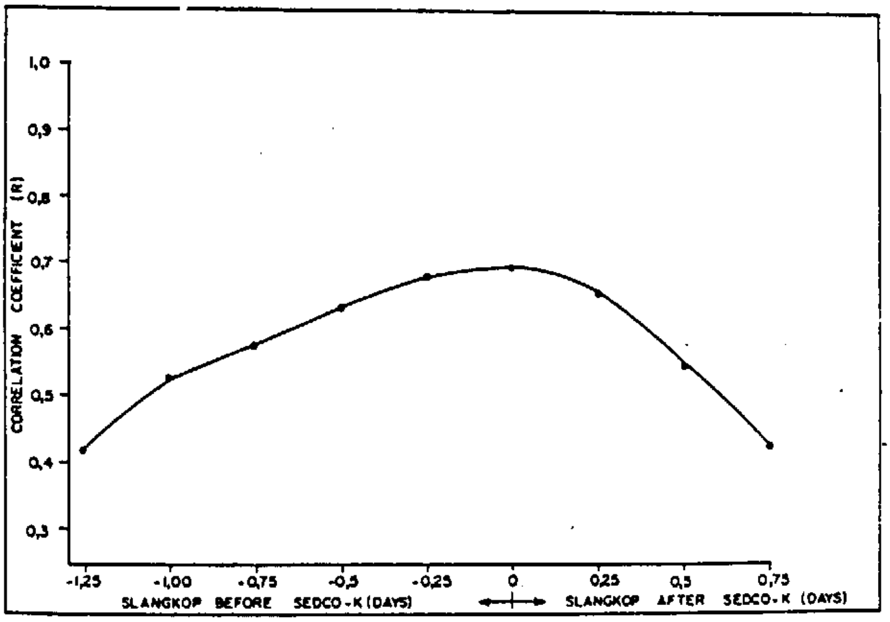


FIGURE 7.2 : CORRELATION BETWEEN H_{mo} AT SLANGKOP AND SEDCO K (CSIR 1981)

Many reasons can be found for the variation in simultaneously recorded wave heights including directional effects, smaller and more local weather systems occurring which do not influence both stations and the intrinsic accuracy of the wave height estimates (see Section 8.4). The reason for the relative insensitivity of the correlation coefficients to a positive and negative lag of six hours can also be explained by the presence of coastal lows which often precede the cold fronts (Hunter 1987) and have a more pronounced effect on the waves at Mossel Bay. This sometimes causes an increase of wave height at Mossel Bay before it starts to increase at Slangkop.

Due to the difficulties experienced with the more formal correlation of wave heights at Slangkop and Sedco K, it was decided to directly infill lost data at one station with data from the alternative station. Since the main purpose of this exercise is to develop methodologies for the estimation of design wave heights distortions caused by small inaccuracies in the infilled wave heights are considered insignificant.

During the process of filling the gaps in the Sedco K records a tendency was noticed for the Waveriders to malfunction near the peak of the storms. Examples of this phenomenon are shown in Figure 7.3. The tendency to malfunction increased with increasing storm intensity, i.e. of the 25 storms in which H_{mo} values exceeding 6,0m were recorded at Sedco K, simultaneous records were available from Slangkop on 17 occasions. Only 8 storms with H_{mo} > 7m were recorded at Sedco K and on only two of these occasions were data available from Slangkop. The two storms at Slangkop and the one storm at Sedco K where an H_{mo} of 8m was exceeded were not recorded at the alternative station.

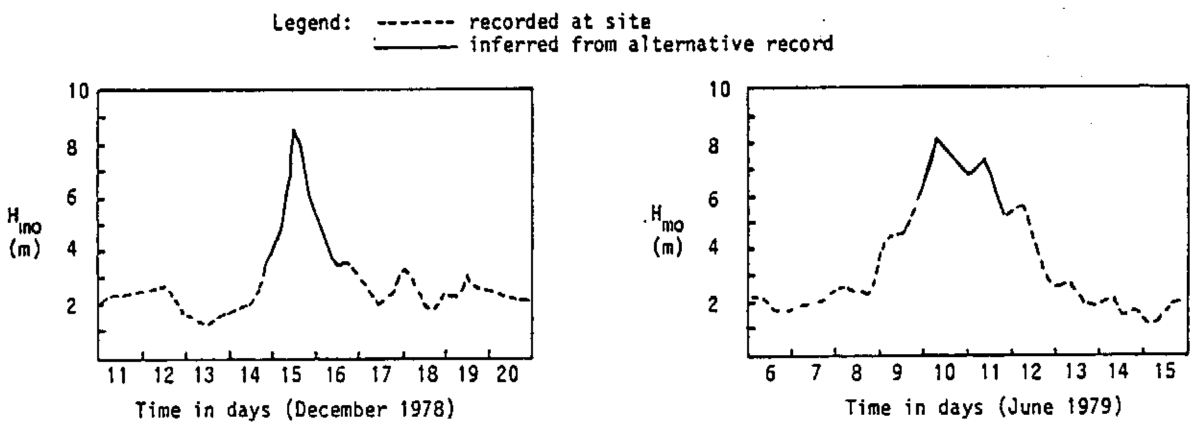


FIGURE 7.3 : EXAMPLES OF WAVERIDER MALFUNCTION DURING STORMS

Careful study of the synoptic weather charts revealed that no major storm occurred during the gaps that still existed after compiling the combined data set. The influence of the tendency for the Waveriders to malfunction during storms on the estimation of design wave heights is investigated in Section 8.3.

7.3 SAMPLING FROM THE DATA SET

7.3.1 General

The techniques involved in design wave height estimation require the fitting of probability distributions to the recorded data. The basic assumption made in all this theory is that the data used in the process should be independent and identically distributed. When studying the variation of wave height with time shown in Figures 6.5

to 6.12 for example, it is immediately clear that wave heights recorded at six hourly intervals are not independent. Similarly the more frequent occurrence of storms in the winter months results in the wave heights recorded during all months of the year not being identically distributed. A sampling plan must therefore first be developed whereby independent and identically distributed samples can be selected from the total data set described in Section 7.2.

7.3.2 Independence of Samples

The standard statistical test for independence of data recorded as a time series is to do serial correlation between the data recorded at increasing time intervals (lag). As the lag increases the serial correlation coefficient reduces and as it approaches zero the data is considered independent. Serial correlation studies using the data set described in Section 7.2 and various subsets of the data were performed. The results are summarised in Table 7.1.

TABLE 7.1 : SERIAL CORRELATION COEFFICIENT - AGULHAS BANK DATA

Data Set	Lag (hours)									
	6	12	18	24	30	36	42	48	54	60
All Data June 1978 - May 1986	.85	.72	.58	.46	.35	27	.21	.16	.12	.09
7 Sept 79 - 7 Oct 79	.88	.71	.53	.37	.16	.03	0			
1 Sept 80 - 30 Sept 80	.86	.72	.59	.48	.37	.31	.22	.16	.11	.05
1 Jan 79 - 31 Jan 79	.84	.70	.54	.40	.25	.15	.06	.04	0	

From this table it is clear that wave heights recorded at six hour intervals are highly correlated. The correlation coefficient decreases slowly with increasing lag. With a lag of 24 hours the coefficient varies from 0.37 to 0.48. This compares favourably with the values of 0.3 along the coast of Japan and 0.5 off Botany Bay, Australia, reported by Wang and Le Mehaute (1983). The correlation coefficients with a lag of 24 to 36 hours are however higher than the value of 0 reported by CERC for the waves off the Atlantic coast of the USA.

To reduce the serial correlation coefficient to below 0.1, a lag of as much as 60 hours is required for some of the data sets shown in Table 7.1. To ensure that samples taken from the data set are independent it was decided not to take more than one sample for each 60 hour interval.

Difficulty was experienced when trying to obtain an independent sample for each 60 hour period. This is illustrated in Figure 7.4 where the wave heights recorded between 7 September 1979 and 7 October 1979 are shown.

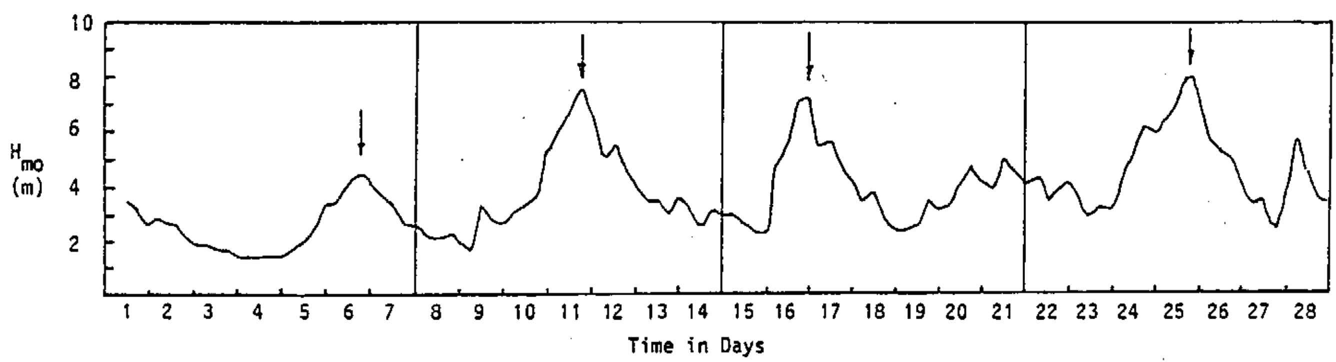


FIGURE 7.4 : SELECTION OF MAXIMUM H_{mo} PER WEEK

This is the second data set shown in Table 7.1 and has a serial correlation coefficient of 0 at a lag of 42 hours. The average duration between peaks in wave height for this period is 5 days. This corresponds to the average duration between the passage of cold fronts along this part of the coast (Hunter 1987). It was therefore decided to base the selection of independent H_{mo} values on an event basis. The maximum H_{mo} recorded in each week of recording was therefore selected as the most intense sampling which will ensure that each sample comes from a different event. Care was taken in the sampling process to ensure that events which fell near the division of the weeks did not feature in two successive weeks.

7.3.3 Identical Distribution of the Samples

Because of the changes in latitude in the paths of the cold fronts from summer to winter, seasonal differences in wave height can be expected. To obtain identically distributed samples from the data it will therefore be necessary to combine only samples from those months which show similarity in wave height. The mean and standard deviation of the maximum weekly H_{mo} values recorded in each month are given in Table 7.2

TABLE 7.2 : MEAN AND STANDARD DEVIATION OF MAXIMUM WEEKLY H_{mo} PER MONTH - AGULHAS BANK

	MAY	JUNE	JULY	AUG	SEPT	OCT	NOV	DEC	JAN	FEBR	MARCH	APRIL	YEAR
NUMBER OF WEEKS	31	28	29	30	31	28	29	31	32	29	30	31	359
MEAN	4,726	4,864	4,897	4,920	5,087	4,629	4,003	4,019	3,919	3,745	4,090	4,158	4,418
STANDARD DEVIATION	1,630	1,237	0,898	0,995	1,210	1,191	0,825	1,171	0,787	1,031	1,132	0,913	1,198

The mean values for the winter months May to September are very similar with values around 4,9m. Similarly the summer months November to April show only small variations around a value of 4,0m. The standard deviation shows no clear pattern with high waves in the months where extreme storms were recorded (i.e. May) and low values in the months which were free from such storms (i.e. January).

The mean and standard deviation of the six hourly records obtained from 7870 records out of the same data set, as summarised by Button (1988), are shown in Table 7.3.

Again the mean values for the winter months May to September are rather constant (vary from 2,8 to 3,0m) and the standard deviations for these months now become more consistent (varying from 1,0 to 1,3). The mean six hourly wave height for the summer months November to April vary from 2,4 to 2,6m and the standard deviation for these months remains around 0,8m except for April when it is 1,0m.

From the above it is concluded that by combining data from the stormy months May to September the requirement of identically distributed samples is approached.

TABLE 7.3 : MEAN AND STANDARD DEVIATION OF SIX HOURLY H_{mo} PER MONTH- AGULHAS BANK

	ALL MONTHS	MAY	JUNE	JULY	AUGUST	SEPTEMBER	OCTOBER	NOVEMBER	DECEMBER	JANUARY	FEBRUARY	MARCH	APRIL
Number of Observations	7870	632	601	629	680	654	623	606	703	705	596	712	702
Mean	2.668	2.839	2.828	2.905	2.943	3.033	2.788	2.584	2.420	2.437	2.359	2.392	2.553
Standard Deviation	1.015	1.268	1.100	1.060	1.020	1.184	1.044	0.838	0.754	0.796	0.849	0.829	1.026

7.3.4 Selection of Sampling Procedure

The most intensive sampling plan that will approach the requirements of independence and identical distribution would be to use the maximum weekly H_{mo} recorded in the stormy months May to September. From the data set described in Section 7.2, 149 such samples could be selected.

Other less intensive sampling plans which will also fulfill the requirements would be to use the maximum H_{mo} recorded in each of the stormy months (40 samples) and the maximum H_{mo} recorded in each year (8 samples).

The maximum weekly, monthly and yearly H_{mo} values are summarised in Appendix C.

Another sampling plan which was investigated briefly was the so-called peaks over threshold (POT) approach. The peak H_{mo} value reached between the up and down crossing of a chosen threshold value of H_{mo} was taken as an independent event. Answers obtained when fitting probability distributions to these samples were however found to be very sensitive to the threshold value chosen. This sampling method was therefore not pursued any further.

7.4 FITTING DISTRIBUTIONS TO THE DATA

The three most frequently used methods for fitting probability distributions to observed data are the graphical method, the method of moments and the maximum likelihood method. These methods with their advantages and disadvantages are described in many standard statistical texts (e.g. Johnson and Kotz, 1970) and will not be repeated here. A good summary of these methods as applied to wave data can be found in Carter et al (1986).

The approach that is used in this study is as follows :

(i) Investigate the fit of a few of the more popular probability distributions used for the long term distribution of wave height to the data. The Exponential, Extreme I, Log-normal and Weibul distributions are used for this purpose. The graphical method is used to fit these distributions to the total data set (i.e. all 10537 H_{mo} values).

(ii) A distribution that fits the data well is selected to test the influence of the sample plans discussed in Section 7.3.4 on the derived design wave heights. The method of moments is used to estimate the parameters of the distribution.

(iii) The three methods of parameter estimation, i.e. graphical, moments and maximum likelihood are compared using the distribution selected in (i) above.

(iv) Methods for selecting an appropriate model for the long term distribution of wave height are investigated.

The most probable values of the 1, 10 and 100 year recurrence interval waves are used to compare the results of the various methods and data sets described above.

7.5 COMPARISON OF DISTRIBUTIONS

To obtain a first indication of the fit of the distributions to the data, various distributions were fitted using the graphical method applied to all data without regard for the known correlation between the data. This method is very popular amongst designers and is probably the method most frequently used for the determination of design wave heights.

The occurrence of wave height in the form of a frequency distribution as given in Appendix C was used for this purpose.

The following probability distributions were used :

7.5.1 The Exponential Distribution

The standard technique used by the CSIR for the determination of design wave heights is to do a least squares fit of H_{mo} against ln E where E represents the percentage exceedance of H_{mo}. The fit is limited to H_{mo} values occurring less than 50 per cent of the time, i.e. E < 50 per cent. The data points used in the process are the pairs of the H_{mo} and E values corresponding to 0,5m steps in H_{mo}.

The standard fit as produced by the CSIR (1987)for the data set described in Section 7.2 is shown in Figure 7.5.

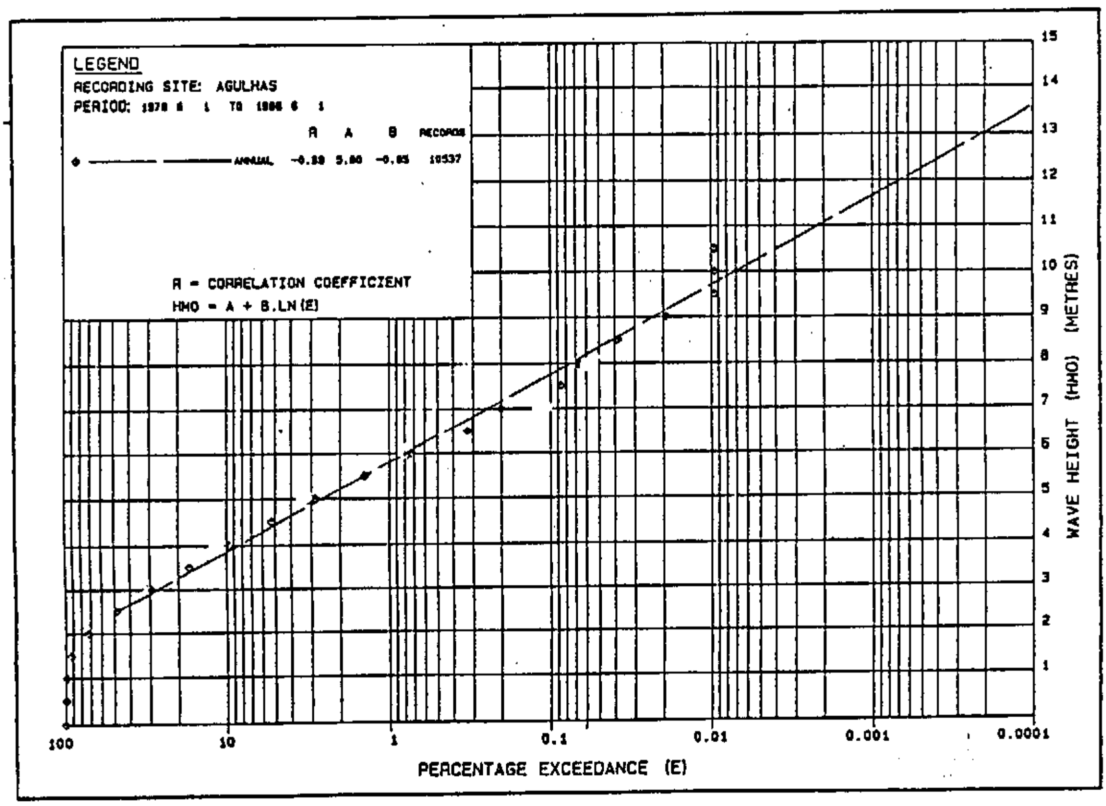


FIGURE 7.5 : EXPONENTIAL DISTRIBUTION

The percentage points of the distribution are given by

$$H_{mo}\ (E) = 5{,}80 - 0{,}85\ \ln E$$

To convert E to the more commonly used probability of non exceedance of H_{mo}, p, the following conversion is required

$$p = 1 - E/100$$

The percentage points are then given by

$$H_{mo}\ (p) = 1{,}89 - 0{,}85\ \ln\ (1 - p)$$

The most probable 100 year H_{mo} will correspond to

$$E = 1/(365 \times 4 \times 100) \times 100 = 6{,}8493 \times 10^{-4} \text{ per cent}$$

or $p = 1 - 1/(365 \times 4 \times 100) = 0{,}99999315$

Therefore $H_{mo_{100}} = 12$ m

7.5.2 The Extreme I Distribution

The Extreme I distribution will lead to a straight line on graph paper if H_{mo} is plotted against $-\ln(-\ln p)$. Such a graph showing the data points and a best fit line by eye is shown in Figure 7.6.

The percentage points of this distribution can be calculated as

$$H_{mo}\ (p) = 2.2 - 0.79\ \ln\ (-\ln p)$$

This gives $H_{mo_{100years}} = 11{,}60$ m

7.5.3 Log-Normal Distribution

The graphical fit of the log-normal distribution is obtained by plotting log H_{mo} against $\phi^{-1}(p)$ where $\phi\ (p)$ is the standard normal distribution. The fit of the data to the log-normal distribution is shown in Figure 7.7. The line representing the distribution was fitted by eye.

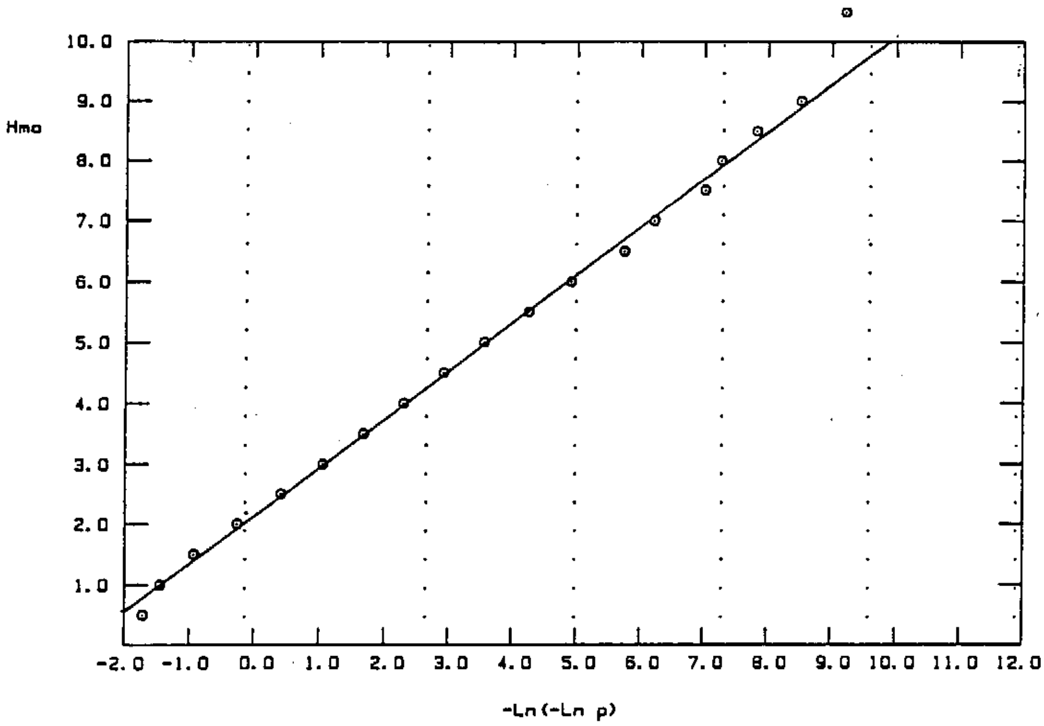


FIGURE 7.6 : EXTREME I DISTRIBUTION

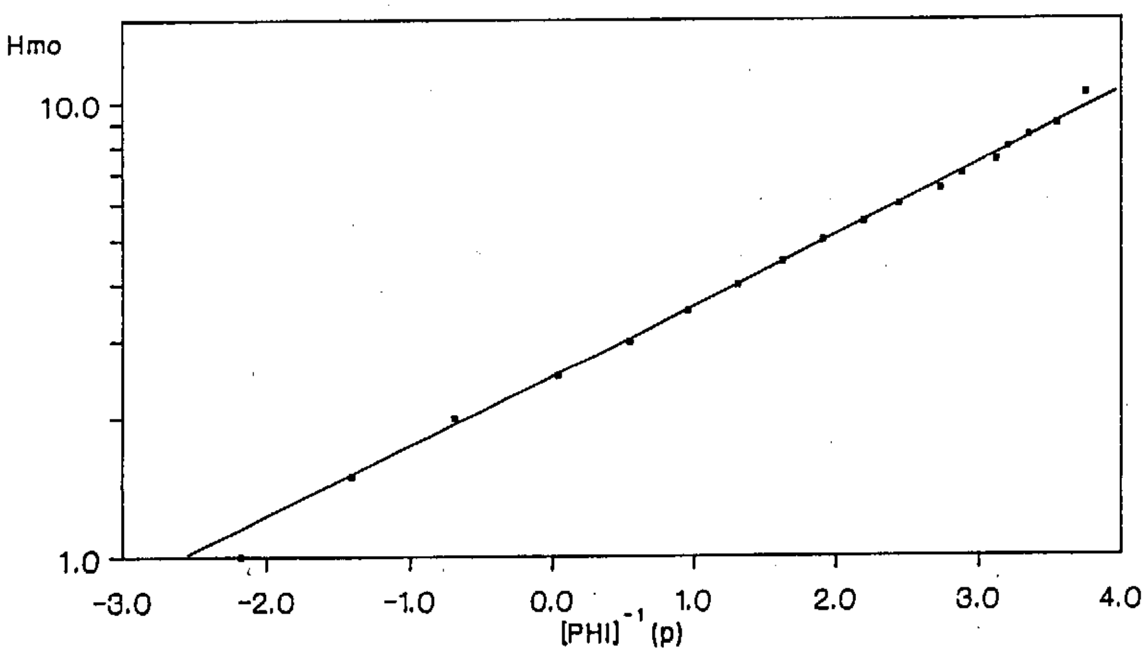


FIGURE 7.7 : LOG-NORMAL DISTRIBUTION

The percentage points of this distribution are given by

$$H_{mo}(p) = \exp\{\phi^{-1}(p)\,\sigma + \mu\}$$

with μ and σ representing the mean and standard deviation of $\ln H_{mo}$. These values can be estimated from the data or as with all the other distributions the values of the percentage points can be read directly from the graph. According to Figure 7.7

$H_{mo_{100years}}$ = 12,1 m.

7.5.4 Weibul Distribution

The Weibul distribution will lead to a straight line on a graph if $\ln H_{mo}$ is plotted against $\ln\{-\ln(1 - p)\}$. The data are plotted on such a graph in Figure 7.8. The data obviously do not follow a straight line and are therefore not Weibul distributed.

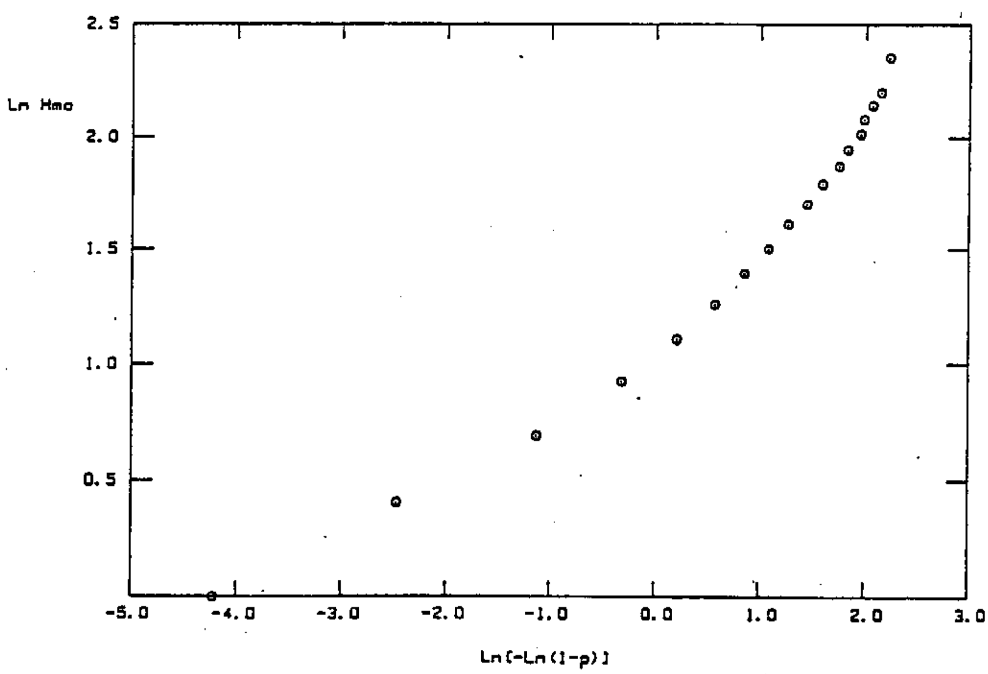


FIGURE 7.8 : WEIBUL DISTRIBUTION

A three parameter Weibul distribution was fitted to the data by subtracting a constant A from the H_{mo} values. The fit of a three parameter Weibul distribution to the data is shown in Figure 7.9. Only wave heights exceeding 3,5m were used in fitting this line to obtain the relationship

$$\ln\{-\ln[1-p(H_{mo})]\} = 0{,}1792 + 1{,}0037\ \ln(H_{mo}-2)$$

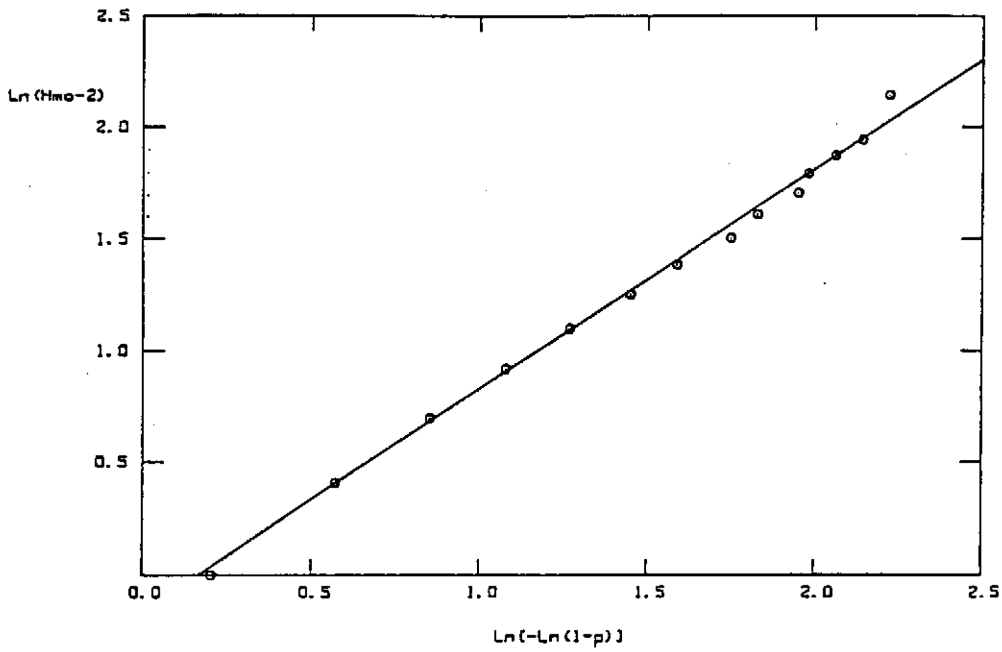


FIGURE 7.9 : THREE PARAMETER WEIBUL DISTRIBUTION

7.5.5 Comparison of Results

As can be seen from the foregoing all the distributions tested except the two parameter Weibul distribution fit the data well. The Weibul distribution can be forced to fit the data by using a third parameter. The values of the most probable 1, 10 and 100 year wave derived from the distributions are summarised in Table 7.4:

TABLE 7.4 : Values of H_{mo_1}, $H_{mo_{10}}$ and $H_{mo_{100}}$
Agulhas Bank Data - Graphical Method

Distribution	H_{mo_1}	$H_{mo_{10}}$	$H_{mo_{100}}$
Exponential	8,1	10,0	12,0
Extreme I	8,0	9,8	11,6
Log-normal	8,0	10,0	12,1
Weibul (3 par)	7,9	9,9	11,9

Differences between the design wave heights obtained by these four distributions are well within the accuracies that can be expected from the different methods of fit. This is best illustrated by the difference between the Exponential and Extreme I distribution. In Figure 7.10 both these distributions are shown on one figure. Since $\ln (1-p) \approx \ln (-\ln p)$ for $p > 0,8$, the upper tails of these two distributions are identical.

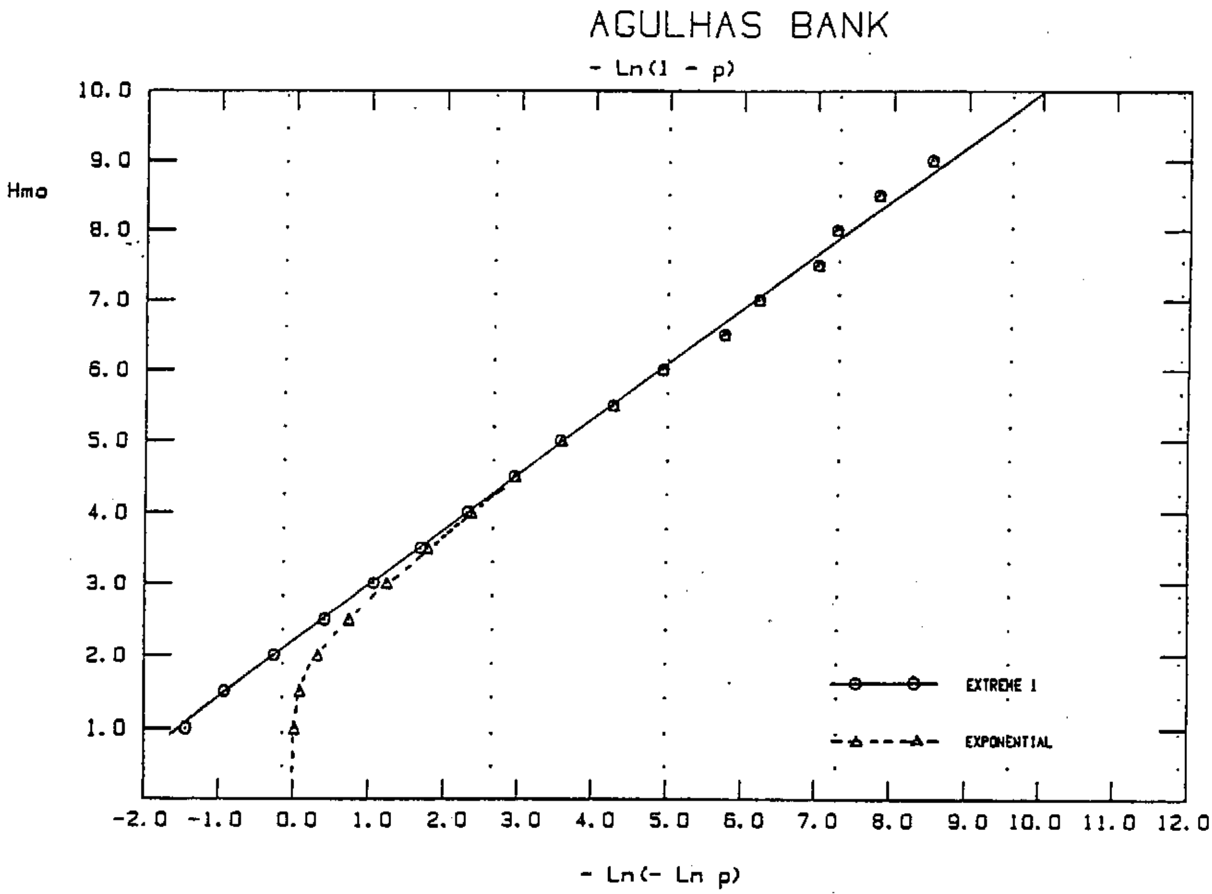


FIGURE 7.10 : COMPARISON BETWEEN EXPONENTIAL AND EXTREME I DISTRIBUTION

The differences in predicted wave heights with these distributions are purely due to the treatment of the highest H_{mo} value. In the least square fit by the CSIR this extreme value is given excessive weight by treating it as three points in the fitting process, i.e. the exceedence of H_{mo} of 10,5, 10,0 and 9,5m at $(1-p) = 0,0001$ were all caused by one H_{mo} value of 10,6m.

Of the four distributions used, the Extreme I and Log-normal distributions are most satisfactory since they fit the wave heights over the entire range of recorded heights in contrast to the Exponential and the Weibul which are only fitted to the upper tail.

Because of the good fit of the data to the Extreme I distribution as well as the ease of application of the method of moments to this distribution, it was decided to use this distribution for tests on the influence of sampling procedures and method of parameter estimation. The problem of selecting the most suitable probability distribution using a more scientific approach is addressed in Section 7.8.

7.6 INFLUENCE OF SAMPLING PROCEDURE

To test the influence of the sampling procedure on the derived design wave heights, the method of moments was used to estimate the parameters of the Extreme I distribution for the following sampling methods:

(i) All 6-hourly records
(ii) Maximum H_{mo} per week
(iii) Maximum H_{mo} per month
(iv) Maximum H_{mo} per year

The same sampling procedures were also applied to the data collected during the stormy months May to September.

The results are summarised in Table 7.5.

As can be seen from the table below, the predicted design waves are very insensitive to the method of sampling with the most probable 10 year H_{mo} varying from 9,3m to 9,9m and the 100 year H_{mo} from 11,5m to 11,9m. Neither the dependence of the data when using all the data, nor the non-identical distribution of the data when the calmer summer months are included, seem to seriously influence the result.

TABLE 7.5 : COMPARISON OF SAMPLING METHODS
EXTREME I DISTRIBUTION

(a) Data from all Months

Sampling Method	N	$H_{mo_{10}}$	$H_{mo_{100}}$
All data	10 357	9,80	11,63
Max H_{mo} per week	359	9,73	11,88
Max H_{mo} per month	96	9,52	11,68
Max H_{mo} per year	8	9,53	11,88

(b) Data from Stormy Months

Sampling Method	N	$H_{mo_{10}}$	$H_{mo_{100}}$
All data	4 463	9,92	11,91
Max H_{mo} per week	149	9,50	11,72
Max H_{mo} per month	40	9,29	11,46
Max H_{mo} per year	8	9,53	11,88

According to Wallis (1988) correlation between data should not alter the expected values but will influence the uncertainty of the estimates as measured by the confidence limits or r m s errors. In the example above however it should be considered fortuitous that such similarity in results were obtained when varying the number of samples from 8 to 10 537.

The fact that inclusion of the data from the calmer summer months in the data set did not seriously influence the result is not surprising due to the small difference in wave height between the summer and winter and the fact that the winter storms will dominate in the total data set.

7.7 COMPARISON OF METHODS OF PARAMETER ESTIMATION

To compare the influence of the method of parameter estimation on the derived design wave heights, the parameters of the Extreme I distribution were estimated using the method of moments and the maximum likelihood method. The graphical methods were not included in this exercise and are not used again in this dissertation except for parameter estimation.

The data samples used in this comparison were the maximum H_{mo} recorded per week, per month and per year. The software used in the maximum likelihood method were developed by Zucchini (1984).

The most probable values for the 10 and 100 year H_{mo} derived by the two methods are compared in Table 7.6.

TABLE 7.6 : MOST PROBABLE 10 AND 100 YEAR H_{mo} VALUES FROM METHOD OF MOMENTS AND MAXIMUM LIKELIHOOD METHOD

Method of parameter estimate	Sampling Method					
	Max H_{mo} per week N = 359		Max H_{mo} per month N = 96		Max H_{mo} per year N = 8	
	$H_{mo_{10}}$	$H_{mo_{100}}$	$H_{mo_{10}}$	$H_{mo_{100}}$	$H_{mo_{10}}$	$H_{mo_{100}}$
Moments	9,80	11,88	9,52	11,68	9,53	11,88
Likelihood	9,98	12,23	9,65	11,86	8,97	11,17

In the two cases where the number of records were 359 and 96 respectively differences in the predicted heights were in all cases less than 3 per cent and well within acceptable limits. When using only 8 data points, the difference between the methods increased to more than 6 per cent.

7.8 MODEL SELECTION

Up to this stage in this study, the selection of an appropriate model for the long term distribution of wave height was based entirely on the apparent goodness of fit of the data to a straight line on the given probability paper. In many engineering applications this is the standard procedure that is followed. In Section 7.5 an example of this approach was given and it was concluded that either the Extreme I or Log-normal distribution would be suitable to describe the long term distribution of wave heights recorded at six hourly intervals.

A more formal approach of model selection could be accomplished by making use of some goodness of fit criteria in which the discrepancies between the data and the fitted distributions are used to quantify the goodness of fit. A large number of so-called "goodness of fit" criteria have been proposed and a summary of these can be found in Zucchini and Adamson (1984). In an effort to improve the fit of the data to the distributions, more parameters are often added to the distributions (i.e. Ochi 1982).

Criticism of the technique of basing the selection of a model only on the goodness of fit of that model to a given set of data has been expressed by a number of researchers (i.e. Linhart and Zucchini (1986) and Wallis (1988)). Wallis (1988) and Goda (1988) have recently shown that if a given distribution is used to generate synthetic data and distributions are fitted to this data, the wrong distribution is often chosen based on some goodness of fit criteria. This even happens for relatively large data sets.

In an effort to improve the model selection process, Linhart and Zucchini (1986) suggested the use of bootstrap sampling. In principle this involves the fitting of a number of distributions not only to the recorded data but also to bootstrap samples from the data. The following procedure is proposed by them:

(i) Select a number of likely models (Weibul, Extreme I, Log-normal, etc.).

(ii) Select a goodness of fit criterion - say Kolmogoroff discrepancy.

(iii) Select a random sample of size n (with replacement) from the original observations to obtain a bootstrap sample.

(iv) Calculate the parameters of the models selected in (i) using the method of maximum likelihood.

(v) Calculate the discrepancy for each distribution.

(vi) Repeat steps (iii) to (v) a large number of times (say 100) and keep track of the discrepancies.

(vii) The model which gives the lowest average discrepancy over the 100 repetitions is chosen as the most appropriate.

This bootstrap sampling technique was applied to the 149 weekly maximum H_{mo} values recorded during the stormy months. Six probability distributions were considered, i.e. the Gamma, Normal, Log-normal, Exponential, Weibul, and Extreme I distributions. The Kolmogoroff discrepancy was used as a goodness of fit criteria and the method of maximum likelihood for parameter estimation. The results are summarised in the Table 7.7 showing the number of the times that the various models were selected (as percentage), the average discrepancy for each model and the most probable value of the 100 year wave ($H_{mo_{100}}$) for each model.

TABLE 7.7 : Bootstrap Selection of Model - Real Data

Model	% Percentage Selected	Av. Discrepancy	$H_{mo_{100}}$
Log-normal	54	0,066	10,7
Gamma	28	0,071	9,9
Extreme I	12	0,074	12,3
Normal	6	0,096	8,9

The result of this bootstrap method clearly illustrates the dilemma that one faces in model selection. The goodness of fit criteria chosen indicate that the first three models listed above all fit the bulk of the data reasonably well. The upper tail of these models do however differ significantly which results in large differences when these distributions are extrapolated to obtain design values. Emphasis on the upper tail of the distribution can be incorporated in the goodness of fit criteria but a sound statistical criteria whereby the degree of emphasis could be decided, is not available.

To test whether bootstrap sampling is really effective in selecting the correct model for typical wave data, a simple simulation was performed. Data were generated according to an Extreme I distribution where the maximum H_{mo} per week were given by

$$H_{mo} = 3,87 - 0,977 \ln(-\ln R) \quad \text{--(1)}$$

where R is a random number between 0 and 1.

The number of data points generated were increased from 50 to 100 to 200 and each of these three samples were subjected to the bootstrap procedure described above. Again the two main contenders were the Log-normal and Extreme I distributions. The results are shown in Table 7.8.

TABLE 7.8 : Model Selection by Bootstrap
Data Generated with Extreme I Distribution

Number of Samples	Extreme I			Log-Normal		
	% Selected	Average Discrep	$H_{mo_{100}}$	% Selected	Average Discrep	$H_{mo_{100}}$
50	39	.1039	12,5	18	.1042	10,8
100	30	.0814	12,4	33	.0787	11,1
200	39	.0585	12,3	43	.0577	11,1

The true value of the most probable $H_{mo_{100}}$ given by (1) above equals 12,23m.

The results shown above indicate that bootstrap sampling is unable to consistently select the correct model. The tendency to select the Log-normal distribution which under-estimates the true $H_{mo_{100}}$ remains even for 200 samples.

7.9 CONCLUSIONS

(i) The significant wave height distribution of a combined data set for the Southern Cape gives a good visual fit to the Extreme I and Log-normal distribution over the total range of recorded wave heights.

(ii) The maximum H_{mo} recorded per week in the stormy months May to September approach the conditions of independent and identically distributed samples

(iii) Parameter estimation for the Extreme I distribution using the method of moments is found to be insensitive to assumptions regarding the independence and identical distribution of the wave height samples.

(iv) Bootstrap techniques were unsuccessful in selecting the correct model for the long term distribution of wave heights.

8. RECOMMENDED PROCEDURE FOR DESIGN WAVE HEIGHT ESTIMATION FOR THE SOUTH AFRICAN COAST

8.1 INTRODUCTION

In this chapter a procedure for the estimation of design wave height is recommended. This recommendation is based on the analysis of the best available data set for the South African coastline given in the previous chapter. The effects of the loss of data during storms and of inaccuracies on the estimates of H_{mo} on the derived wave heights are investigated.

8.2 RECOMMENDED PROCEDURE

Based on the analysis in the previous chapter it is recommended that the method of moments applied to all data be used to determine the parameters of the Extreme I distribution. This distribution can then be extrapolated to the required probability to determine the design wave heights. It is further recommended that the data should be plotted on Extreme I probability paper and that the distribution determined by the method of moments be drawn on that plot. Under no circumstances must design wave heights be reduced if the highest 1 per cent of the data do not fit the distribution.

The rationale behind the above recommendations is as follows :

(i) The reason for using all data is that it was shown that an unbiased estimate of the parameters can be obtained in this way. With the scarcity of properly recorded wave data at many stations this method of sampling will ensure that maximum use is made of the available data.

(ii) The reason for choosing the method of moments for parameter estimation is its ease of application especially to grouped data. Graphical methods are not favoured and the maximum likelihood method is impractical with the large data samples proposed in (i) above.

(iii) The Extreme I distribution is chosen because

  (a) It fits the best available data set extremely well over the entire range of wave heights;

  (b) Application of the method of moments to this distribution is extremely simple;

(c) It is a true two parameter distribution having a linear wave height term. This means that constants can be added or subtracted from the wave heights without changing the essence of the distribution. This is extremely useful when the sampling method results in wave heights with a lower limit higher than zero;

(d) The statistical theory for the distribution is well developed and can be applied to estimate confidence bands and other properties of the distribution.

(iv) The recommendations regarding the graphical representation of the distribution and the warning not to adjust the design waves downwards are motivated later in this section after looking at the effect of lost storms due to Waverider malfunctions.

The procedure used to estimate the parameters of the distribution is as follows:

(1) Calculate the mean H_{mo} and the variance of H_{mo} from the available data. This can be done directly or from grouped data.

(ii) Estimate the parameters of the distribution α and β from the following equations:

Mean μ $= \alpha + 0{,}5772\ \beta$

Variance σ^2 $= \beta^2\ \pi^2/6$

(iii) The percentage points of the distribution are given by

$$H_{mo}(p) = \alpha - \beta \ln(-\ln p)$$

8.3 EFFECT OF LOST STORMS ON THE DESIGN WAVE HEIGHTS

8.3.1 Introduction

In Section 7.2 examples were shown of the tendency of Waveriders to malfunction near the peak of the storms. Similar experiences have been reported by others, e.g. Stanton (1984). To investigate the effect of the preferential loss of records during storms on the design wave height predicted with the method recommended in 8.2 above, a simple simulation procedure is followed. The effect of the same loss if a graphical procedure is followed is also illustrated.

Assume that over a two year period wave heights occurred which, if they were recorded six hourly, would have produced 2920 records of H_{mo} which followed an Extreme I distribution given by $H_{mo}(p) = 2,66 - 1,03 \ln(-\ln p)$. The number of records in each 0,5m wave height interval obtained from these two years are given in Table 8.1, column 2.

TABLE 8.1 : EXTREME I DISTRIBUTION
LOSS OF RECORDS DURING STORM

H_{mo} (m)	N	N_{lost}	N_{new}
0 0.5	1	0	1
0.5 - 1.0	32	0	32
1.0 - 1.5	232	0	232
1.5 - 2.0	544	0	544
2.0 - 2.5	660	0	660
2.5 - 3.0	552	0	552
3.0 - 3.5	377	0	377
3.5 - 4.0	230	0	230
4.0 - 4.5	132	0	132
4.5 - 5.0	73	0	73
5.0 - 5.5	40	0	40
5.5 - 6.0	22	0	22
6.0 - 6.5	12	2	10
6.5 - 7.0	6	3	3
7.0 - 7.5	3	3	0
7.5 - 8.0	2	2	0
8.0 - 8.5	1	1	0
8.5 - 9.0	1	1	0
TOTAL	2920	12	2908

N = Number of records in wave height group - ideal Extreme I distribution

N_{lost} = Number of records lost in group

N_{new} = Number of records recorded in group

Further assume that during this two year period only 12 records were lost and that these losses occurred only during storms. The assumed distribution of these 12 lost records is shown in Table 8.1, column 3. All seven records exceeding 7m are assumed lost as well as 5 records in the range 6m to 7m. Due to the high correlation between six hourly records such a loss can easily occur during only one or two storms.

The effect of the loss of these 12 records out of a total of 2920 on the H_{mo} distribution is illustrated in Figure 8.1. The loss indicates a rather drastic deviation from the upper tail of the Extreme I distribution.

The effect of these 12 lost records on the predicted design waves using the recommended method above and the method currently being used by the CSIR is investigated below.

8.3.2 Extreme I Distribution - Method of Moments

The effect of the 12 lost records on the parameters of Extreme I distribution as well as on the most probable 1, 10 and 100 year H_{mo} values is tabulated below.

TABLE 8.2 : EFFECT OF LOST RECORDS
EXTREME I DISTRIBUTION (METHOD OF MOMENTS)

Number of Lost Records	$\bar{H}_{mo}$ (m)	$\sigma\ H_{mo}$ (m)	α (m)	β (m)	H_{mo1} (m)	H_{mo10} (m)	H_{mo100} (m)
0	2,6618	1,0260	2,200	0,800	8,03	9,87	11,71
12	2,6427	0,9915	2,197	0,773	7,83	9,61	11,39
% Reduction	0,7	3,4	0,14	3,4	2,5	2,6	2,7

The distribution fitted to the data after losing the 12 records is also shown in Figure 8.1.

As can be seen from Table 8.2 and Figure 8.1, the effect of losing the 12 records is to reduce typical design wave heights by less than 3 per cent.

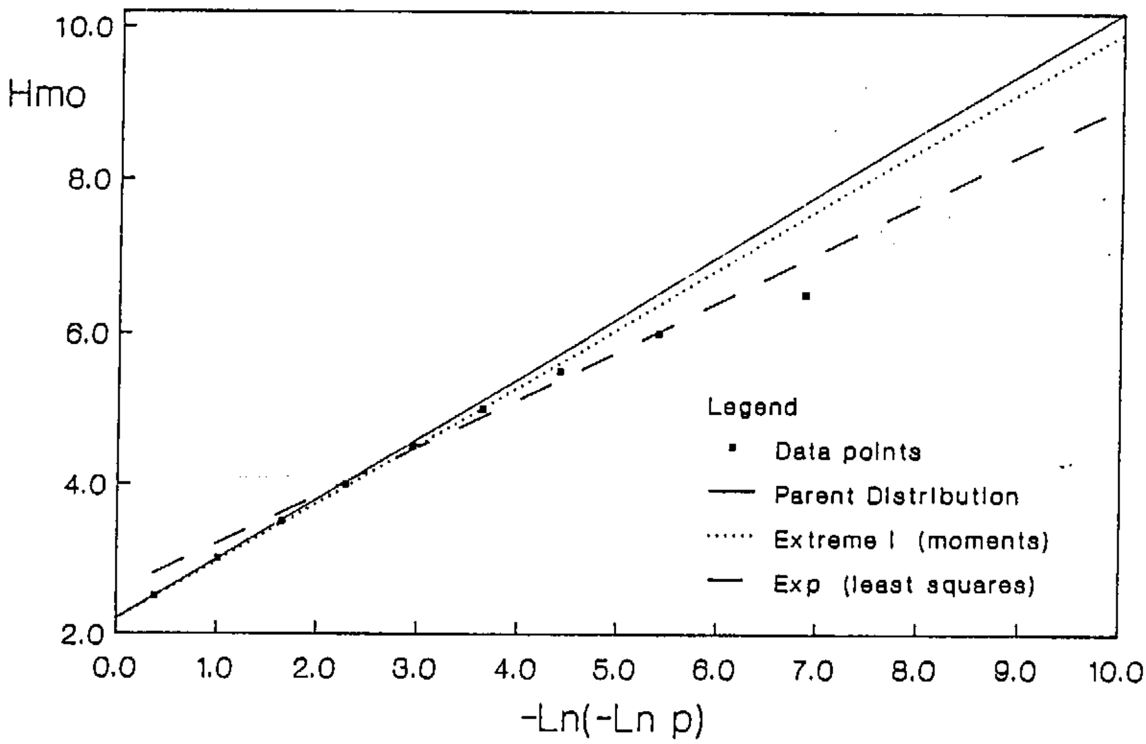


FIGURE 8.1 : EFFECT OF LOSS OF DATA DURING STORMS

8.3.3 Graphical Fit to Upper Tail of Exponential Distribution

The standard method used by the CSIR (Rossouw 1984) to estimate design wave heights is to fit an Exponential distribution to the form

$$H_{mo}\ (E) \quad = \quad a + b \ln E \quad \text{for } E < 50 \text{ per cent}$$

where E = percentage exceedence of H_{mo}

A least square fit for E < 50 per cent or p > 0,5 on Figure 8.1 is used to determine a and b. Each "data" point in this range (shown on Figure 8.1) is given an equal weighting.

The result of this method applied to the data set with the 12 lost records is summarised in table 8.3 below.

TABLE 8.3 EFFECT OF LOST RECORDS
EXPONENTIAL DISTRIBUTION (GRAPHICAL METHOD)

Number of lost records	a (m)	b (m)	H_{mo_1} (m)	$H_{mo_{10}}$ (m)	$H_{mo_{100}}$ (m)
0	5,837	-,8178	8,03	9,91	11,80
12	5,392	-,6618	7,17	8,69	10,21
% Reduction			10,7	12,3	13,5

The distribution fitted to the data after losing the 12 records is shown in Figure 8.1.

A rather drastic reduction in design wave heights occurs due to the loss of only a few high records. The reason for this is obviously the high weighting given to the higher waves with this method, i.e. the 552 waves in the range 2,5m to 3,0m are given the same weight as the 3 waves in the range 6,5m to 7,0m (see Table 8.1). This makes this method extremely sensitive to the distribution of the few highest waves recorded and since these waves can even be highly correlated, this method is not acceptable.

8.3.4 Conclusions on Effect of Lost Records

The design waves derived via the Extreme I distribution using the method of moments is fairly robust and is not seriously influenced by a few lost records during storms. Similarly it will not be seriously influenced by an exceptionally high storm.

The graphical methods applied to grouped data are misleading in that the upper tail has an unreasonably high influence on the results.

8.4 EFFECT OF INACCURACIES OF H_{mo} ESTIMATES ON DERIVED DESIGN WAVE HEIGHTS

The main source of inaccuracy when estimating H_{mo} from record lengths of short duration (i.e. 1024s or less) is the intrinsic variation in wave height. In many storms it was found that large differences in H_{mo} estimated from the two halves of a 1024s record were registered. Examples of such differences are shown in Table 8.4.

TABLE 8.4 EXAMPLES OF DIFFERENCES IN H_{mo} DURING TWO SUCCESSIVE 512s PERIODS

Date	Station	H_{mo} (m)		% Difference
		1st Half	2nd Half	in H_{mo}
16.12.78.	Slangkop	7,85	9,46	19
17.12.78.	Slangkop	6,05	4,73	24
26.06.79.	Slangkop	7,40	8,54	14
04.02.83.	Slangkop	5,00	6,33	23
04.02.83.	Slangkop	7,27	5,71	24
04.09.84.	Slangkop	5,47	7,10	26
17.09.79.	Actinea	5,32	6,51	20
23.09.79.	Actinea	6,36	8,05	23

These differences are not caused by recording errors but result from variation in roughness of the sea surface from one 512s period to the next. This clearly shows that a 512s recording length is often insufficient for an accurate estimate of H_{mo}. Since a record length of 512s is often used if one half of the record is rejected or nonexistent, these inaccuracies in the estimation of H_{mo} should be taken into account in the analysis.

To test what influence these variations in wave height would have on the predicted design wave heights, a number of records at Slangkop, Sedco K and Koeberg were all split into halves and the most probable 1, 10 and 100 year H_{mo} were calculated from each set of half records. The method recommended in Section 8.2 was used for this purpose. The results are summarised in Table 8.5 below:

TABLE 8.5 : DESIGN WAVES CALCULATED FROM HALF RECORDS

Station	Half	Number of records	$\bar{H}_{mo}$	$\sigma_{H_{mo}}$	H_{mo_1}	$H_{mo_{10}}$	$H_{mo_{100}}$
Slangkop	1st	2257	2,260	0,941	7,52	9,21	10,90
	2nd	2155	2,613	0,970	7,69	9,43	11,17
Sedco K	1st	2358	2,790	1,006	8,05	9,86	11,66
	2nd	2349	2,810	1,023	8,16	10,00	11,83
Koeberg	1st	8299	1,754	0,774	5,80	7,19	8,58
	2nd	8744	1,756	0,777	5,82	7,23	8,61

Differences in the design wave heights derived from the two halves vary from a maximum of 2,4 per cent in the case of Slangkop to 0,4 per cent in the case of Koeberg. Again the method of moments applied to all the data proves to be robust and unbiased inaccuracies in H_{mo} estimates do not reflect badly on the accuracy of the derived design wave heights. In alternative analysis such as sampling the maximum H_{mo} per week, per month or per year these inaccuracies can be expected to have a far greater influence.

8.5 CONCLUSIONS

Parameter estimation for the Extreme I distribution using the method of moments applied to six-hourly wave records proves to be very robust and is insensitive to the loss of records during storms and unbiased inaccuracies in the recorded wave heights.

9. DESIGN WAVE HEIGHTS FOR THE SOUTH AFRICAN COAST

9.1 INTRODUCTION

In Rossouw (1984) design wave heights were estimated from all the available data up to July 1981. A least squares fit to the upper tail of an Exponential distribution as described in Sections 7.5.1 and 8.3.3 of this study, was used for this purpose.

Since the publication of Rossouw (1984) considerable quantities of additional Waverider data have become available. A summary of the data available in usable form as on September 1988 was given in Table 5.1. As explained in Section 8.2 the technique now proposed for the determination of design wave heights involves the fitting of an Extreme I distribution to all six hourly records using the method of moments. The preferred wave height parameter to be used in this process is H_{mo}. Where an H_{mo} analysis was not available H_1^{10} or H_s (Draper) were used. The distributions were fitted to these height parameters and the fitted distributions were then converted to H_{mo} using the relationships established in Section 9.2.

9.2 CONVERTING WAVE HEIGHT PARAMETERS TO H_{mo}

9.2.1 H_1^{10} to H_{mo}

At Walvis Bay and Luderitz the only wave height parameter available is H_1^{10}. To be able to convert H_1^{10} to H_{mo}, correlation studies were undertaken at Slangkop and Sedco K where both these parameters were available. The relationships established at these stations were compared with those tabulated in Draper (1966) based on the work of Cartwright and Longuett-Higgins (1956) and Cartwright (1958). For a record length of 512s the following relationships were established from the Slangkop and Sedco K data:

$H_1/H_{mo} = 1{,}710 - 0{,}0247\ T_z$ based on 4707 Slangkop records
and $H_1/H_{mo} = 1{,}703 - 0{,}0247\ T_z$ based on 4412 Sedco K records

This is compared to the values derived from Draper in Table 9.1.

TABLE 9.1 : RATIOS OF H_1/H_s FOR 512s RECORD LENGTH

T_z	H_1/H_s (Slangkop)	H_1/H_s (Sedco K)	H_1/H_s (Draper)
6	1,56	1,56	1,56
8	1,51	1,51	1,54
10	1,46	1,46	1,49
12	1,41	1,41	1,45

In general agreement between the data recorded in the South African waters and those tabulated by Draper were excellent. Allowing for the fact that record lengths of 600s were used in the analysis of the Walvis Bay and Luderitz data and that the dominant T_z values are in the range 7s to 9s, the relationship $H_{mo} = H_1^{10}/1,53$ was used in this study.

9.2.2 H_s (Draper) to H_{mo}

The study referred to in Section 9.2.1 above also warrants the use of $H_{mo} = H_s$ for the old Oranjemund and Saldanha data since Draper analysis uses the same relationship between H_1 and T_z to convert the recorded H_1 to H_s.

9.2.3 $H_{s_{inc}}$ (Draper) to H_{mo}

Due to errors made in the original Draper analysis procedure used by the CSIR in South Africa where H_1 and H_2 were confused, the correction

$$H_{mo} = 1.1\ H_{s_{inc}}$$

had to be applied to the Weathership and Port Zimbali data (see Rossouw 1984).

9.3 PRESENTATION OF THE HEIGHT DISTRIBUTIONS

In Figure 9.1 the data for each station and the Extreme I fit to the data are presented. The parameters of the distribution as well as the most probable values of the 1, 10 and 100 year H_{mo} values are summarised in Table 9.2.

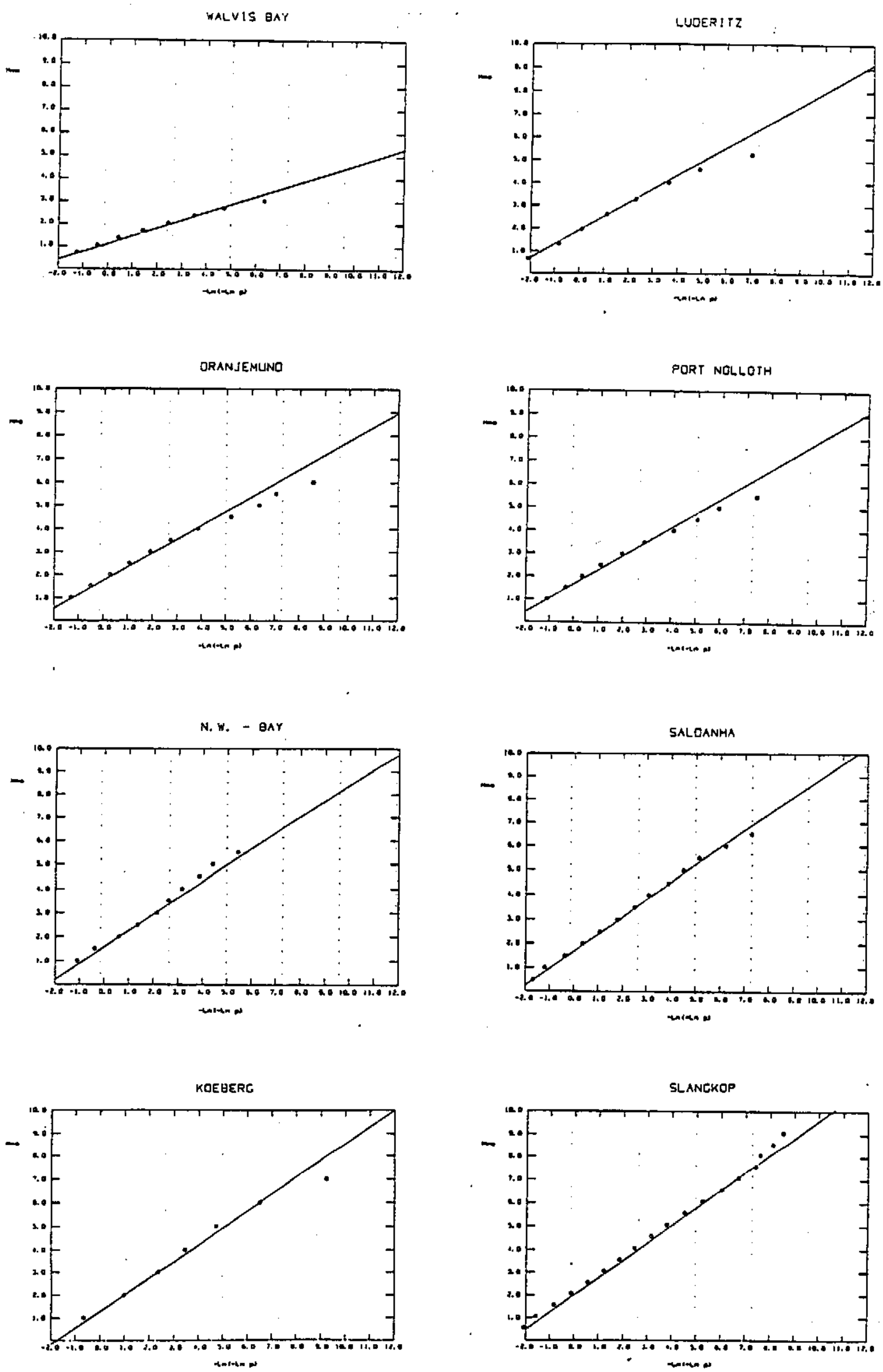


FIGURE 9.1 : EXTREME I FIT TO DATA AT EACH STATION

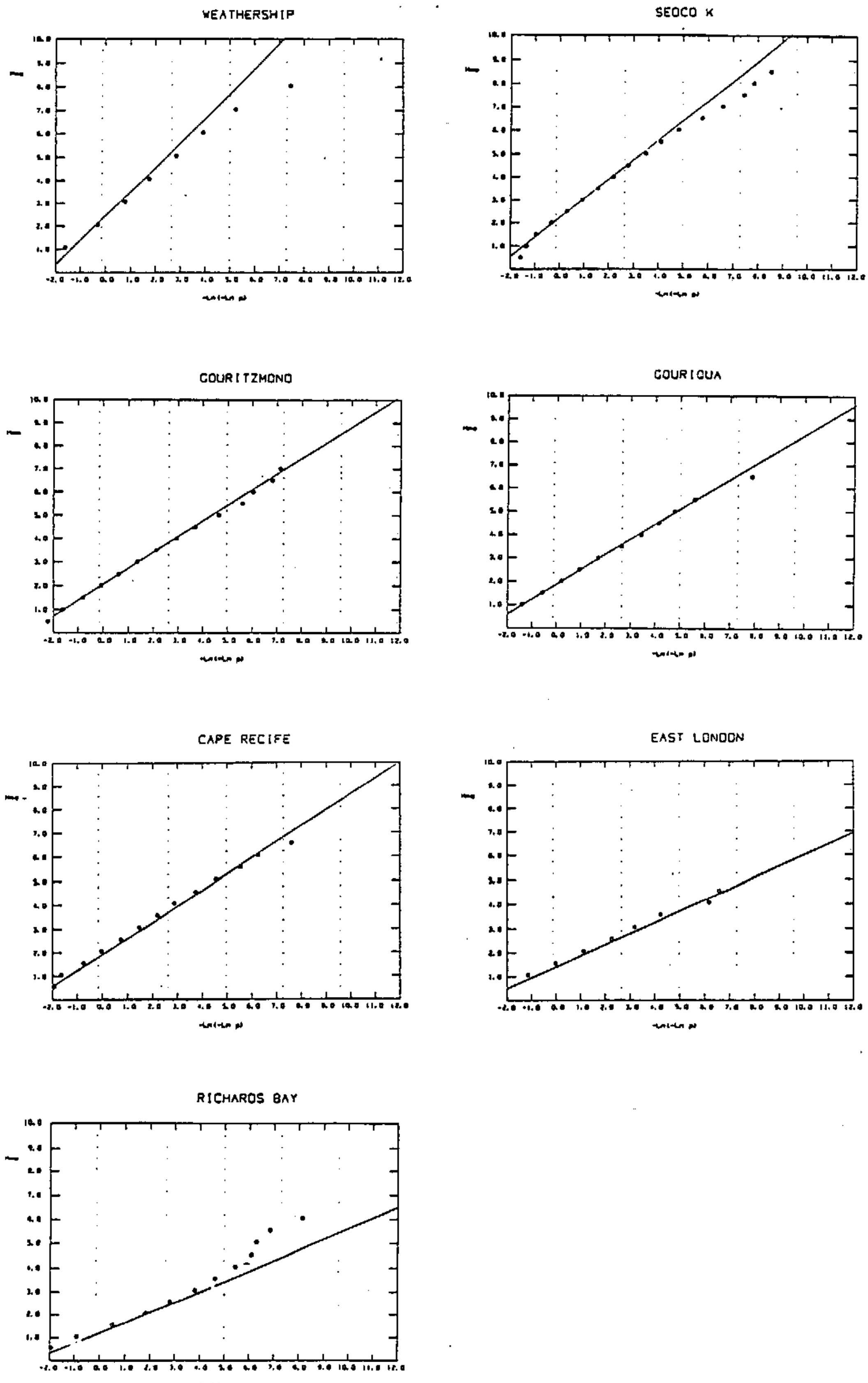


FIGURE 9.1 : EXTREME I FIT TO DATA AT EACH STATION

TABLE 9.2 : DESIGN H_{mo} VALUES FOR SOUTH AFRICAN AND SOUTH WEST AFRICAN COASTLINE

Station	Number of Records	Water Depth (m)	$\bar{H}_{mo}$ (m)	$\sigma_{H_{mo}}$ (m)	α (m)	β (m)	H_{mo_1} (m)	$H_{mo_{10}}$ (m)	$H_{mo_{100}}$ (m)
Walvis Bay	2194	50	1.32	0.44	1.12	0.3432	3.62	4.42	5.21
Luderitz	2281	108	2.21	0.78	1.86	0.6097	6.30	7.71	9.11
Oranjemund (old)	1429	20	1.80	0.77	1.45	0.6014	5.83	7.22	8.60
Oranjemund	5495	106	2.18	0.77	1.83	0.6019	6.21	7.60	8.99
Port Nolloth	1815	100	2.12	0.80	1.76	0.6227	6.30	7.73	9.17
North West Bay	1139	35	2.05	0.87	1.66	0.6818	6.62	8.19	9.76
Saldanha old)	1523	80	2.18	0.92	1.77	0.7171	7.00	8.65	10.30
Koeberg	8327	11 to 23	1.81	0.93	1.39	0.7265	6.68	8.36	10.03
Slangkop	11424	170	2.54	0.97	2.10	0.7574	7.61	9.37	11.11
Weathership	1617	deep	3.14	1.34	2.54	1.0486	10.18	12.59	15.01
Sedco K	4924	100	2.71	1.07	2.23	0.8345	8.31	10.23	12.15
Actinea	1105		2.78	1.00	2.33	0.7809	8.02	9.82	11.61
Gouritzmond	3895	76	2.45	0.87	2.06	0.6744	6.98	8.53	10.08
Gouriqua	2273	33	2.27	0.83	1.89	0.6452	6.59	8.08	9.57
Cape Recife	2206	85	2.40	0.87	2.01	0.6745	6.93	8.48	10.03
East London	1457	27	1.78	0.59	1.51	0.4596	4.86	5.92	6.98
Richards Bay	7521		1.57	0.56	1.31	0.4398	4.52	5.53	6.54

In general the data fits the Extreme I distribution well over the range of frequently occurring wave heights. Deviations from the fitted distribution do however occur towards the higher wave heights at a number of sites, especially for those wave heights that are exceeded less than 1 per cent of the time. Most of the highest recorded wave heights fall below the upper tail of the Extreme I distribution. This pattern is consistent with the tendency for the Waverider to malfunction during major storms. The deviation is seldom more severe than the deviation caused by the loss of data during one or two storms over a two year period as illustrated in the example of Section 8.3.

Two cases occur where the highest recorded waves are higher than the fitted Extreme I distribution, i.e. Slangkop and Richards Bay. The reason for this upward excursion at Slangkop is undoubtedly the storm of 16 May 1984. This storm was to all accounts a very extreme storm with an expected return period far in excess of the equivalent of 8 years of available data at this site. Seven out of a total of 27 records over the recording period in which H_{mo} exceeded 6,5m occurred during this one storm. The highest and second highest H_{mo} (i.e. 10,6m and 9,5m) were recorded during this storm and the occurrence of such a storm is a good example of how much the upper tail of a distribution can be influenced if correlated data are used. As with the loss of records during storms, it illustrates the fallacy of paying too much attention to the apparent lack of visual fit of the upper tail of a distribution to the data.

At Richards Bay the upward deviation of the data at the upper tail of the distribution is much more serious. This deviation is mainly caused by the occurrence of cyclones, most notably IMBOA (see Section 6.4.4). These cyclones are from a totally different origin as the rest of the waves and no information about the extreme waves can be gained from extrapolating the distributions as were done at the other stations. The highest H_{mo} recorded during IMBOA equalled 8,5m which corresponds to a return period of more than 8000 years according to the fit of the Extreme I distribution to the data at this station. It should further be stressed that it will be just as incorrect to try and fit a distribution to just the upper tail of the data in this case since it only represents one or two independent events. A totally different approach will have to be employed to derive design wave heights in this area. An example of such an approach is given in Sobey (1982).

9.4 DESIGN WAVE HEIGHT PATTERN AROUND THE COAST

In Figure 9.2 the design wave height curves for all the deepsea Waverider stations along the west coast are summarised. Only stations with water depths exceeding 50m are included in this summary. These stations are considered to be relatively free from shallow water and refraction effects. The design curve for Sedco K is also included in Figure 9.2 since it is also fully exposed to the same weather systems as the west coast stations.

The pattern that emerges from Figure 9.2 is very clear and encouraging. There is a steady increase in wave height from Walvis Bay in the north to the Weathership in the south. The increase is most marked between Walvis Bay and Luderitz. From Luderitz to Port Nolloth virtually identical design curves are obtained. The increase south of Port Nolloth is more gradual all the way down to the Weathership. A clear relationship between design wave height and latitude is evident from Figure 9.2. This will be pursued further in Section 9.5.

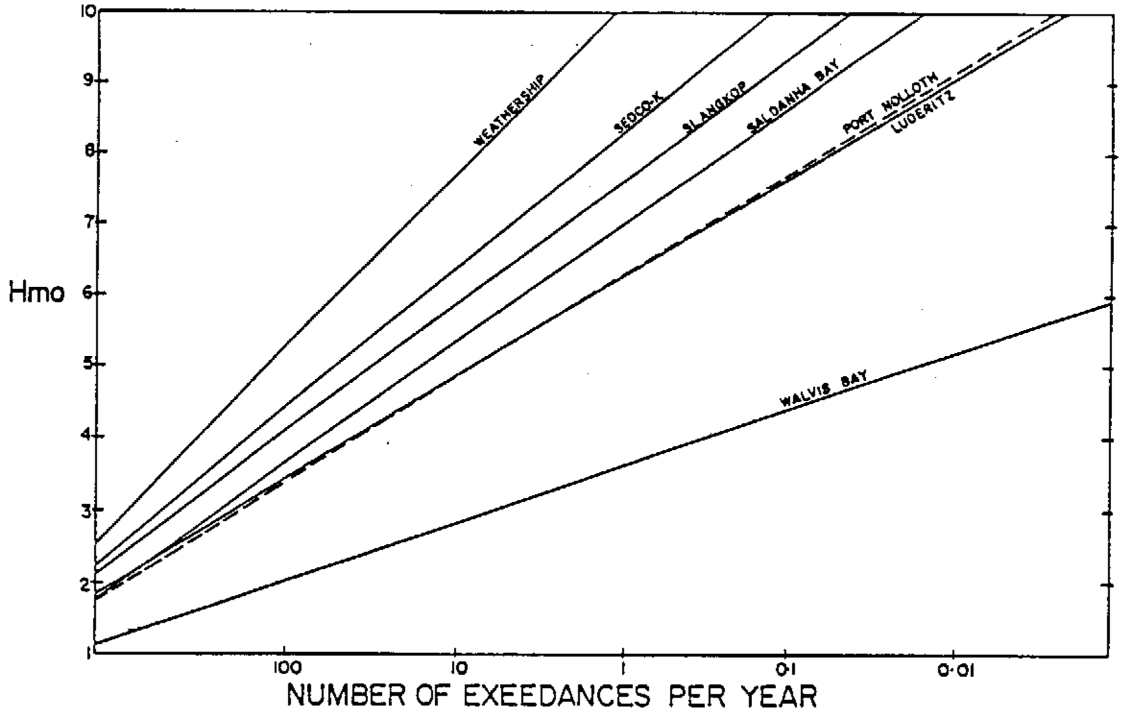


FIGURE 9.2 : DESIGN WAVE HEIGHTS ALONG THE WEST COAST

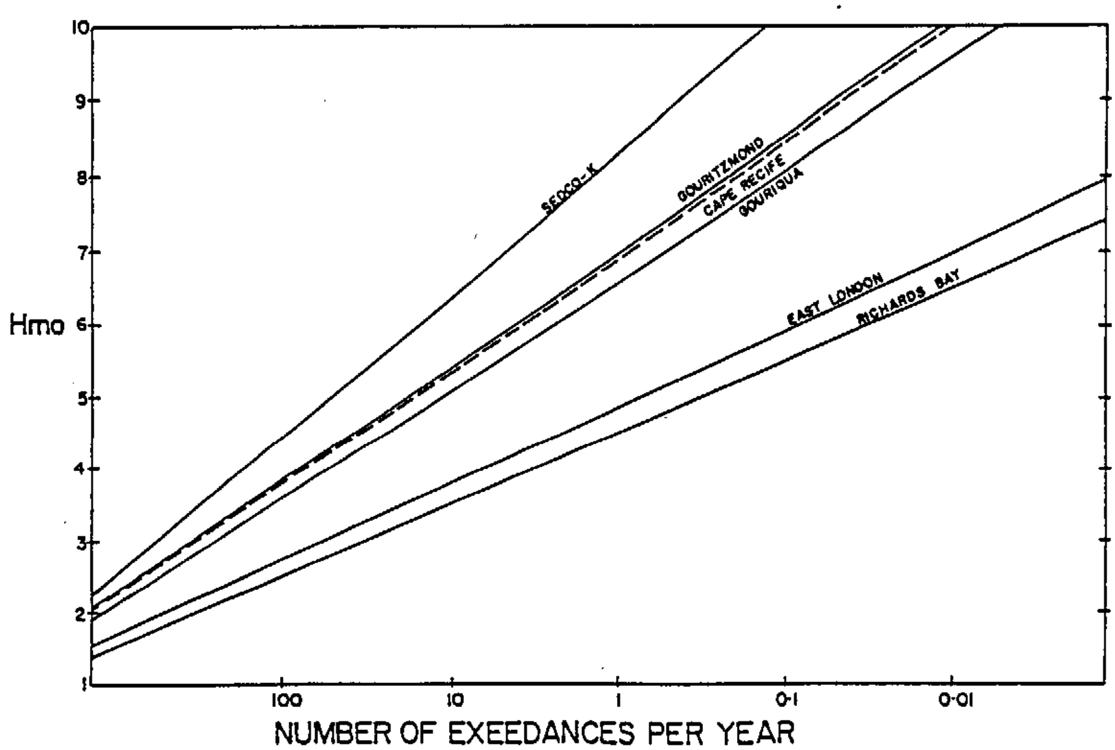


FIGURE 9.3 : DESIGN WAVE HEIGHTS ALONG THE SOUTH COAST

The design curves for the Cape south and east coast from Mossel Bay to East London are shown in Figure 9.3. Again a very clear pattern emerges. Waves are highest at Sedco K. Directly north of Sedco K wave heights reduce towards Gouritzmond as the exposure to the west diminishes due to the protection afforded by the land mass. This reduction continues towards Gouriqua which, in a depth of 33m, will also be influenced by refraction affects.

At Gouritzmond and Cape Recife the design curves are virtually identical which is not unexpected if the similarity in simultaneously recorded wave heights is considered (see Section 6.5).

From Cape Recife to East London a drastic reduction in wave height occurs. Some of this reduction can be attributed to refraction effects due to the shallow water depth at the East London site. Most of the reduction is however caused by the weakening and offshore deflection of the cold fronts discussed in Section 6.

No data exist between East London and Ballitoville on the Natal north coast. Shallow water records are available at Ballitoville and Richards Bay. The water depth at both these stations is less than 20m. This area of the coast is further exposed to the occasional southward migration of tropical cyclones, making the prediction of design waves in this area extremely hazardous.

The lack of deepwater wave records east of Port Elizabeth and the occurrence of cyclones along the Natal north coast make it impossible to make firm recommendations about the choice of deepwater design wave heights east of Port Elizabeth. A few years of deepwater recording off, say Port St Johns and Durban, plus an intensive study of the occurrence of cyclones off the Natal coast will be required to complete the picture of deepsea design conditions around the South African coastline.

9.5 CORRELATION STUDIES

The obvious pattern evident from the design curves for the west and south coast from Walvis Bay to Mossel Bay (see Figure 9.2) gave rise to the idea of correlating design wave heights with latitude.

As a first step the parameter β (see Table 9.2) of the Extreme I distribution was correlated with the latitude expressed in °S (see Table 5.1). The eight recording sites represented in Figure 9.2 were used and a linear regression was performed between these two parameters. The result is shown in Figure 9.4.

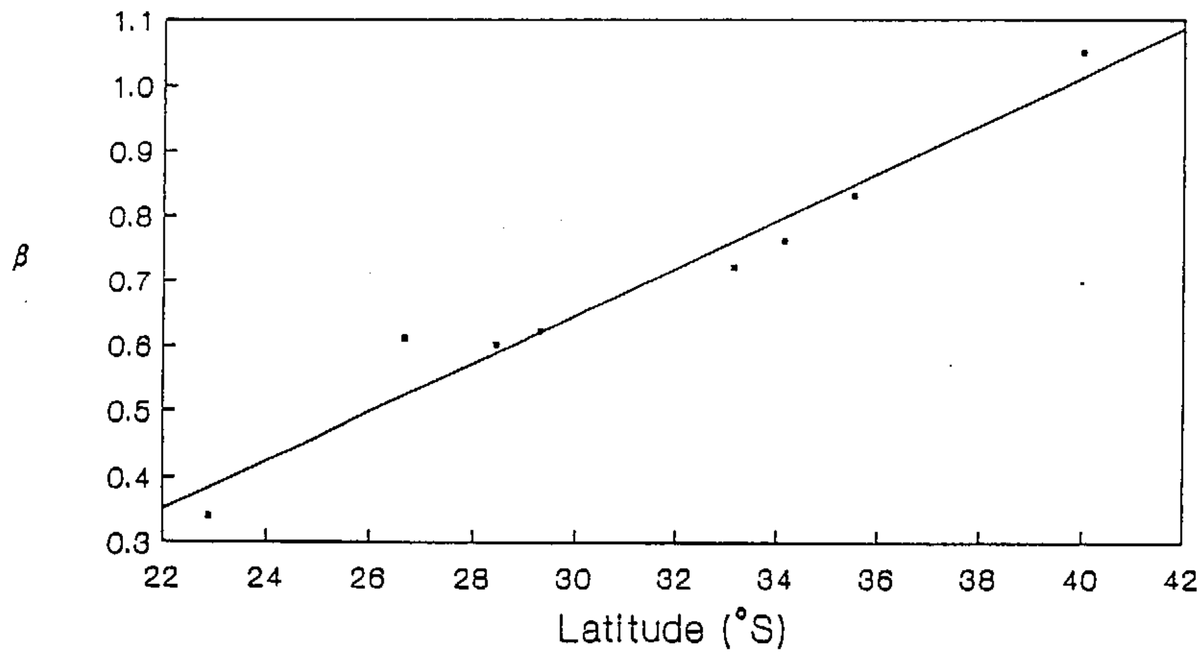


FIGURE 9.4 : CORRELATION BETWEEN β AND LATITUDE

The relationship β = 0,0366x(°S) - 0,4509 .... Eq 9.1
was established with a correlation coefficient of 0,975. A similar correlation between the parameters α and β of the Extreme I distribution was also attempted for these stations. The results are shown In Figure 9.5.

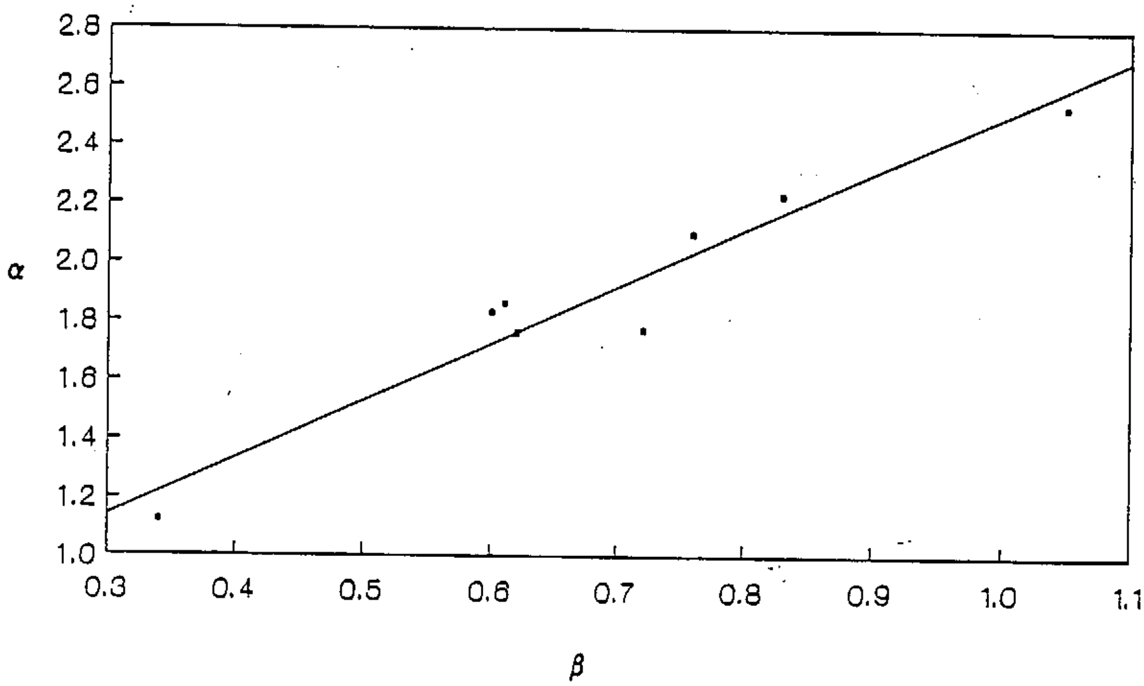


FIGURE 9.5 : CORRELATION BETWEEN α AND β

The relationship $\alpha = 0{,}5492 + 1{,}9268\ \beta$ .... Eq 9.2
was found with a correlation coefficient of 0,967.

With these two correlations it is now a very simple matter to determine the parameters of the Extreme I distribution for any position along this coast, i.e.

(i) Determine the latitude in °S
(ii) Calculate β from equation 9.1
(iii) Calculate α from equation 9.2

To test the accuracy which can be achieved by this simple procedure, the most probable 1, 10 and 100 year waves were calculated for all the recording sites along this coast and the results are shown in Table 9.3.

TABLE 9.3 : COMPARISON OF DESIGN WAVE HEIGHTS FROM DATA AND MODEL

Station	Latitude (°S)	α	β	H_{mo1}	H_{mo10}	H_{mo100}	
Walvis Bay		1.31	0.39	4.12	5.02	5.91	Model
	22.89	1.12	0.34	3.62	4.42	5.21	Data
				12.9	12.7	12.6	% Diff
Luderitz		1.58	0.53	5.40	6.61	7.82	Model
	26.67	1.86	0.61	6.30	7.71	9.11	Data
				-15.4	-15.4	-15.2	% Diff
Oranjemund		1.70	0.59	6.00	7.36	8.72	Model
	28.44	1.83	0.60	6.21	7.60	8.99	Data
				- 3.4	- 3.2	- 3.0	% Diff
Port Nolloth		1.76	0.62	6.29	7.72	9.15	Model
	29.29	1.76	0.62	6.30	7.73	9.17	Data
				- 0.2	- 0.1	- 0.2	% Diff
Saldanha		2.04	0.76	7.59	9.34	11.10	Model
	33.13	1.77	0.72	7.00	8.65	10.30	Data
				8.1	7.7	7.5	% Diff
Slangkop		2.11	0.80	7.93	9.77	11.60	Model
	34.13	2.10	0.76	7.62	9.37	11.11	Data
				4.0	4.2	4.3	% Diff
Weathership		2.53	1.01	9.91	12.25	14.58	Model
	40	2.54	1.05	10.18	12.59	15.01	Data
				- 2.7	- 2.7	- 2.9	% Diff
Sedco K		2.21	0.85	8.39	10.34	12.30	Model
	35.5	2.23	0.83	8.31	10.23	12.15	Data
				1.0	1.1	1.2	% Diff

At all of the stations from Oranjemund to Sedco K, including the Weathership but excluding Saldanha, the predicted design wave heights up to the 100 year wave are within 5 per cent of those derived directly from the data. At Saldanha the model over-estimates the wave heights by 8 per cent. North of Oranjemund the result is much worse leading to an under-estimate of approximately 15 per cent at Luderitz and an over-estimate of 13 per cent at Walvis Bay.

It is interesting to note that at the three stations where the largest deviations occur, i.e. Walvis Bay, Luderitz and Saldanha, the data used for the estimation of the parameters were collected between 1972 and 1974 (see Table 5.1). Furthermore these data were hand analysed to produce H_1^{10} (Walvis Bay and Luderitz) and H_s (Saldanha Bay) and the quality checks performed on the latter data were not performed on these data sets. The temptation is therefore to reject this data. This is not recommended since a physical explanation can be found for these deviations.

A physical explanation for the higher than expected heights at Luderitz and the lower than expected heights at Walvis Bay can be found if the recording locations are considered. At Luderitz the Waverider buoy is situated 11km offshore and the orientation of the coast such that it affords no protection from the southerly and south-southeasterly waves (see Figure 9.6). Due to the presence of the South Atlantic high, winds in this area are dominant from the SSW to the SSE sector and high waves are caused by these winds in the offshore areas (Quayle and Elms (1979)). The South Atlantic high is the dominant wave generating system in these offshore areas.

Similarly the Waverider position at Walvis Bay is much closer inshore and the orientation of the coast is such that the southerly and south-southeasterly waves are cut off from this site (see Figure 9.7).

The 8 per cent difference between the model and the data at Saldanha is within the typical year to year variation to be expected if only one year of data is used to determine design conditions (see Section 10.7). Only one year of data was used to determine design conditions at Saldanha Bay.

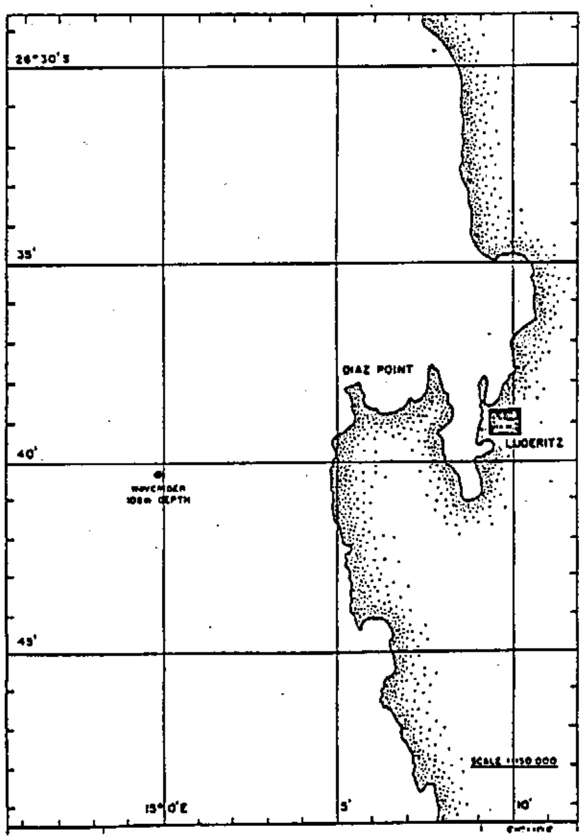


FIGURE 9.6 : POSITION OF WAVERIDER BUOY AT LUDERITZ

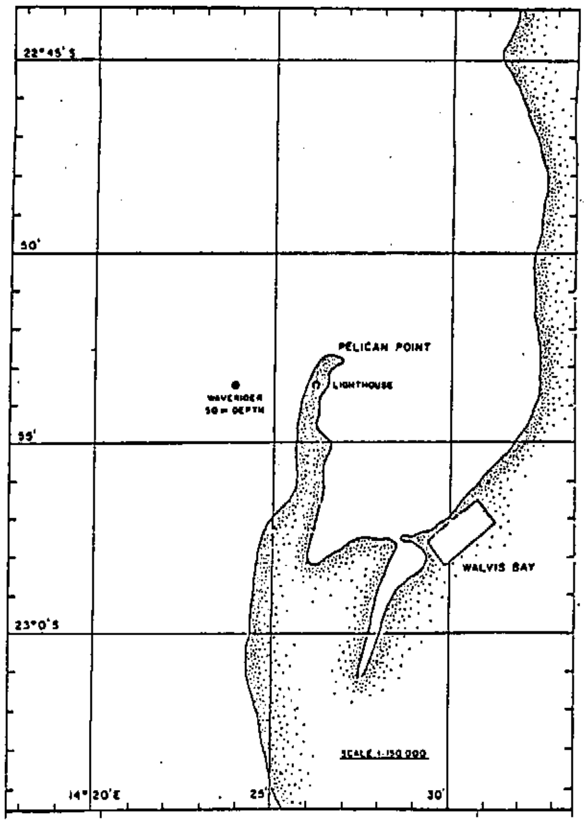


FIGURE 9.7 : POSITION OF WAVERIDER BUOY AT WALVIS BAY

In general the good correlation between the weather patterns and the derived design wave heights is very encouraging and increases the confidence in the data and the techniques used for design wave height estimation.

9.6 CONCLUSION

(i) A strong correlation exists between the design wave heights along the west and south Cape coast and the latitude with an increase in wave heights towards the south.

(ii) North of Oranjemund the South Atlantic High starts to influence the wave heights and an increase in wave heights can be expected with increasing distance offshore or increasing exposure to the waves from the south and south-east.

(iii) East of Port Elizabeth a reduction in wave height occurs. Insufficient deepsea data are available between Port Elizabeth and Durban to be able to make firm conclusions about the wave climate in this area.

(iv) North of Durban to the Mozambique border extreme wave events are caused by tropical cyclones. Extrapolation of recorded wave data will lead to erroneous estimates of design wave heights in this area.

10. CONFIDENCE IN PREDICTED DESIGN WAVE HEIGHTS

10.1 GENERAL

When considering the reliability with which a wave height with a given probability of occurrence can be predicted, a number of sources of uncertainty need to be considered. It has become customary in engineering practice to estimate the wave height with a specified recurrence interval (say once in 100 years) and to express the uncertainty of the estimate in terms of a confidence interval.

There is a large degree of confusion amongst engineers and their clients regarding concepts such as return period, risk and confidence bands. The various sources of uncertainty are therefore systematically analysed here. The process that is followed is to start with an ideal situation such as would exist if an infinitely long and perfect record were available. The various sources of uncertainty are then considered to illustrate how these sources influence the confidence attached to the estimates.

10.2 CONCEPT OF RECURRENCE INTERVAL OR RETURN PERIOD

To examine the concept of recurrence interval or return period, it is handy to assume that the model describing the long term distribution of wave height and its parameters is perfectly known. This would be the case if an infinitely long and perfect wave record which showed no long term trends were available. Let us assume that these waves follow an Extreme I distribution where the distribution of the maximum H_{mo} per year is given by

$$H_{mo}(p) = 8{,}09 - 0{,}85 \ln(-\ln p)$$

where $p = 1 - 1/T_R$

and T_R is the average duration in years between the occurrence of a given wave height. T_R is called the recurrence interval or the return period.

The most probable value of the wave height with a return period of 100 years corresponds to p = 0,99 which gives

$$H_{mo_{100\ years}} = H_{mo}\ (0{,}99) = 12{,}0m$$

Obviously this wave height will vary from one 100 year period to the next due to the random nature of the process. The distribution of the 100 year wave will therefore be as shown in Figure 10.1.

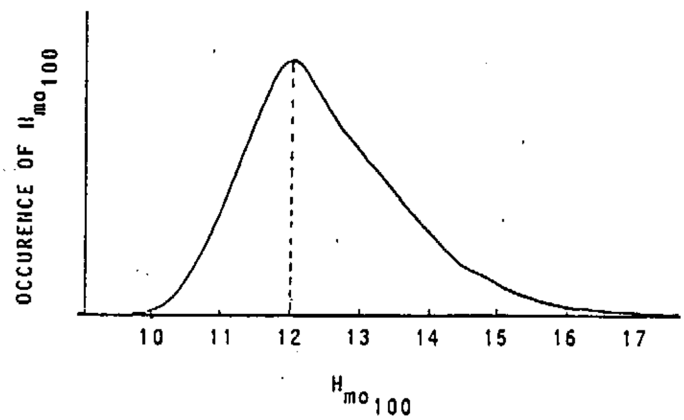


FIGURE 10.1 : DISTRIBUTION OF $H_{mo_{100}}$

From this figure it can be seen that the risk of exceeding the most probable value of the 100 year wave within any 100 year period, is more than 50 per cent (in fact it is 63 per cent). The general expression relating the wave height with a given risk r of being exceeded in T years with the probability is

$$p_T^r = (1 - r)^{1/T}$$

If a designer is therefore only willing to accept a 10 per cent risk of exceeding a given wave height within the next 100 years, he will find that he must design for a height of

$$H_{mo}\ (0{,}999) = 13{,}92m$$

This height is the same as the most probable value of the wave with a recurrence interval of 1000 years and therefore is often called the 1000 year wave.

Although there is in principle nothing wrong with the concept of a return period, its use in design specifications is to be discouraged. A much better approach would be to specify a wave height with a given risk of being exceeded within the design life of the structure.

The risk that has been discussed up to now only refers to the risk inherent in the process and has nothing to do with inaccuracies that may be caused by a multitude of factors in the case where the long term distribution of wave height is not perfectly known. In all cases of design the real risk will be higher than the risk specified above.

10.3 CONCEPT OF CONFIDENCE INTERVALS

In cases where an infinitely long and perfect record is not available, assumptions have to be made about the model to be used for the long term distribution of the wave height and the parameters of the distribution must be estimated from a limited record length. In the case of wave records very few recorded data sets of longer than 10 years exist.

If the model that describes the long term distribution of wave height is chosen correctly and if a representative sample of these wave heights is available, it is possible to estimate the parameters of this distribution from the data. In Section 8.2 a method of estimating the parameters of the Extreme I distribution was recommended. If the assumption of an Extreme I distribution is correct, and if the data is representative, it is possible to estimate the accuracy with which a given design wave height can be predicted (i.e. a height with a given probability of exceedance). This estimate of the accuracy of the estimate is normally expressed in terms of confidence limits and is a function of the number of data points available, the parameters of the distribution and the probability to which the fitted distribution is extrapolated.

Several techniques of estimating the confidence limits are available. They include theoretical derivation, Monte Carlo simulation and Bootstrap techniques. It must however be stressed that these confidence limits will only have meaning if we have chosen the correct model for the long term distribution of the wave heights and if the parameters of the distribution were estimated from a representative sample of wave heights.

It is further of importance to distinguish between one and two sided confidence limits. Since in the selection of design conditions the interest lies in the risk of a given wave height being exceeded, one sided confidence limits are used throughout this study.

10.4 ESTIMATING CONFIDENCE LIMITS

10.4.1 Influence of Sampling Method

When estimating confidence limits it is of utmost importance that the data used in the process should be independent and identically distributed. The number of independent data points has a large influence on the estimate of the confidence bands. This can be illustrated by using the 8 years of data for the Cape south coast described in Section 7.2. Bootstrap techniques using software developed by Zucchini (1984) based on the work of Effron (1982 and 1987) were used to determine the 95 per cent confidence limits for the most probable 100 year H_{mo}. Three sampling methods which fulfilled the requirement of independence and identical distribution were used, i.e. the maximum H_{mo} recorded per year, per stormy month and per week in the stormy months. The results are summarised in Table 10.1.

TABLE 10.1 : 95% CONFIDENCE LIMITS FOR $H_{mo_{100}}$

Sampling Method	Number of Samples	Parameters of Distribution		Probability associated with $H_{mo_{100}}$	$\frac{H_{mo_{100}}(95)}{H_{mo_{100}}}$
		α	β		
Maximum per year	8	7.28	1,000	1 - 1/100 = 0,99	1,35
Maximum per month in stormy months	40	5,61	0,952	1 - 1/500 = 0,998	1,20
Maximum per week in stormy months	149	4,33	1,044	1 - 1/2166 = 0,9995	1,09

The answers obtained by the Bootstrap techniques were also compared to those contained in Challenor (1979) based on the work of Lawless (1974) and excellent agreement was found after correcting for bias in the Bootstrap estimates (Effron 1987).

From Table 10.1 the drastic influence of the sampling method on the derived confidence limits is seen. If maximum use is made of the available data the 95 per cent confidence limit for the most probable 100 year wave height is only 9 per cent higher than the most probable value for this data set.

10.4.2 Influence of Record Length

To study the influence of the record length or quantity of available data on the confidence limits, the method described by Carter and Challenor (1986) was used to estimate the confidence limits. The maximum H_{mo} recorded per week in the stormy months May to September were again used to represent independent and identically distributed events. The parameters of the Extreme I distribution for the Cape south coast as determined in Section 7.6, i.e. $\alpha = 3{,}87$ $\beta = 0{,}977$ were used for this purpose. The process that was followed to estimate the confidence bands was as follows :

(i) Select a value of N (N = 22 for each year of data i.e. 22 weeks during 5 stormy months)

(ii) Calculate $\mathrm{Var}(\tilde{\alpha}) = 1{,}1678\ \beta^2 N^{-1}$

(iii) Calculate $\mathrm{Var}(\tilde{\beta}) = 1{,}10\ \beta^2 N^{-1}$

(iv) Calculate $\mathrm{Cov}(\tilde{\alpha}, \tilde{\beta}) = 0{,}123\ \beta^2 N^{-1}$

(v) Calculate $R = -\ln(-\ln(p))$

$= -\ln(-\ln(1 - 1/22T))$

where T = return period in year

for T = 100 years, R = 7,696

(vi) Calculate $\mathrm{Var}(\tilde{H}_{mo_T}) = \mathrm{Var}(\tilde{\alpha} + R\tilde{\beta})$

$= \mathrm{Var}(\tilde{\alpha}) + 2R\ \mathrm{cov}(\tilde{\alpha}, \tilde{\beta}) + R^2\ \mathrm{Var}(\tilde{\beta})$

(vii) For large N, estimates of H_{mo_T} become normally distributed and the confidence limits can be estimated as follows:

95% confidence limit = $H_{mo_T} + 1{,}645\ [\mathrm{VAR}(\tilde{H}_{mo_T})]^{\frac{1}{2}}$

90% confidence limit = $H_{mo_T} + 1{,}282\ [\mathrm{VAR}(\tilde{H}_{mo_T})]^{\frac{1}{2}}$

Etc.

Using the above approach the 95%, 90% and 80% confidence limits were calculated for the 1, 10 and 100 year H_{mo} estimates and the results are presented graphically in Figure 10.2.

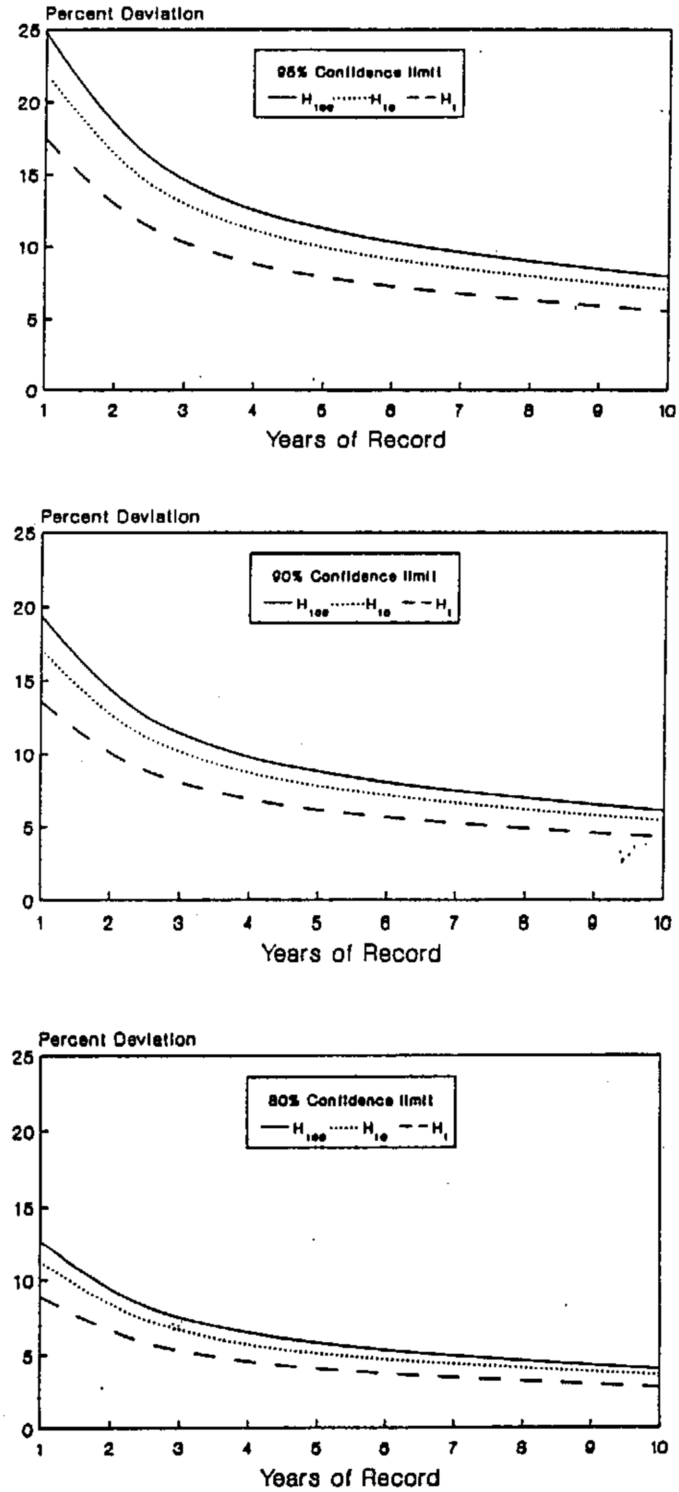


FIGURE 10.2 CONFIDENCE LIMITS AS FUNCTION OF NUMBER OF YEARS OF RECORD

As a check on the abovementioned calculation a very simple Monte Carlo simulation was also performed for a few cases. The simulation was done as follows:

(i) Select a number of data points N, using 22 data points to represent 1 year of data (weekly maximum during 5 stormy months)

(ii) Generate N data points by

$$H_{mo} = \alpha - \beta\ (-\ln R)$$

where R is a random number between 0 and 1

(iii) Use the method of moments to estimate the parameters α and β from the generated data set

(iv) Estimate the most probable 1, 10 and 100 year wave from

$$H_{mo_T} = \alpha - \beta \ln \{-\ln (1 - 1/22\ T)\}$$

(v) Repeat steps (ii) to (iv) a large number of times (say 1000) and keep track of H_{mo_1}, $H_{mo_{10}}$ and $H_{mo_{100}}$ for each loop

(vi) Sort H_{mo_1}, $H_{mo_{10}}$ and $H_{mo_{100}}$. The 95%, 90% and 80% confidence bands are given by the H_{mo} values that are exceeded 50, 100 and 200 times.

Excellent agreement was found between the confidence bands predicted by these two methods.

From Figures 10.2 it can be seen that the width of the confidence bands reduces rather rapidly for the first four to five years of data whereafter the reduction becomes much more gradual. The 80%, 90% and 95% confidence bands reduce to within 10 per cent of the most probable value for the 100 year wave within 2, 4 and 6 years of recording.

If the assumptions of an Extreme I distribution, of a representative sample and of independence and identical distribution of weekly maxima during the stormy months are all valid, this analysis indicates that fairly accurate estimates of up to the most probable 100 year wave can be obtained from only five years of good data.

10.5 CONCEPT OF PREDICTION LIMITS

It is possible to combine the risk of exceedance of a wave height at a given recurrence interval discussed in Section 10.2 with the confidence limits discussed in Section 10.3. This combination of the risk inherent in the process and the inaccuracies caused by the estimate of the parameters of the distribution from a limited record length, is termed a prediction limit. Tables for estimating the prediction limits for the Extreme I distribution are contained in Carter and Challenor (1982). Although the combination of these two uncertainties have merit, especially in the case of risk analysis, such a combination is not recommended for general use. The reason for this recommendation is that whereas there is general consensus amongst engineers and researchers about the inherent risk due to the randomness of the process, no such agreement exists with regard to the confidence bands. The confidence bands also only express part of the total uncertainty with regard to the estimation of design wave heights.

It is therefore felt that if the design wave height is selected on the basis of some acceptable risk of exceedance during the design life of the structure, and confidence bands are provided for this estimate, there will be a much better chance of general acceptance by the engineering community.

10.6 UNCERTAINTIES AND ERRORS NOT INCLUDED IN THE CONFIDENCE LIMITS

Factors which might influence the accuracy with which a given design wave height can be selected which are not included in the estimates of the confidence limits are:

(i) The selection of an appropriate model. In the analysis in Section 10.4 we assumed that the Extreme I distribution described the long term distribution of wave height. Although some evidence has been provided to support this assumption (see Section 7.5) data over many years are required to confirm the correctness thereof.

(ii) The assumption that a representative sample of waves was used to estimate the parameters of the distribution. Factors which will influence whether the samples are really representative are;

(a) Errors and inaccuracies in measurements of H_{mo}

(b) Loss of records during storms

(c) Recording during non-typical years

(d) Long term trends in the climate

It has been shown that the estimates of the parameters of the Extreme I distribution with the method of moments are not very sensitive to (a) and (b) above. The influence of (c) is more difficult to assess but comparison of the record period with records of longer duration can provide some guidance in this respect. Some evidence of long term changes in the wave climate of the North Sea has been reported (i.e. Rye (1976) and Lamb & Weiss (1979)). The recording history in South Africa is too short to be able to make any conclusions with regard to long term changes in climate.

(iii) It was further assumed that the extreme events will come from the same family as the typical storms that were recorded in the data set. There is reason to believe that this assumption is valid for the west and south coast especially due to the total absence of cyclones in the South Atlantic. This assumption is however not valid along the northern Natal coast.

10.7 YEAR TO YEAR VARIATION

To develop some feel for the variation in wave condition from year to year, the data described in Section 7.2 were divided into calendar years. Since January is normally a calm month, such a division will ensure that each stormy season will fall in a separate year. Using the method of moments applied to all data, the most probable 1, 10 and 100 year H_{mo} were estimated from each year of data. The results are summarised in Table 10.2.

TABLE 10.2 : VARIATION IN DESIGN WAVE HEIGHTS FROM YEAR TO YEAR

Year	Number of Records	$\bar{H}_{mo}$	σH_{mo}	α	β	H_{mo_1}	$H_{mo_{10}}$	$H_{mo_{100}}$
1979	1243	2.684	1.0559	2.21	0.823	8.21	10.10	12.00
1980	1318	2.709	0.9429	2.28	0.735	7.64	9.33	11.03
1981	1391	2.663	0.9701	2.23	0.756	7.74	9.48	11.22
1982	1292	2.640	0.9138	2.23	0.713	7.24	9.06	10.70
1983	1420	2.620	1.0559	2.14	0.823	8.14	10.04	11.93
1984	1363	2.669	1.0910	2.18	0.851	8.38	10.34	12.29
1985	1271	2.526	0.9225	2.11	0.719	7.35	9.01	10.66
Mean (m)						7.84	9.62	11.40
σ						0.405	0.535	0.663
$\frac{1,645\ \sigma}{\text{Mean}}$						0.085	0.091	0.096

A large degree of consistency is evident from this table. The mean H_{mo} in the seven year period varied between 2.5m and 2.7m and the standard deviation of H_{mo} between 0,91m and 1,09m. This consistency in wave height from year to year resulted in a rather small variation in the design wave heights derived from the separate years of data. If the estimates of the design wave heights from individual years of data are assumed to be normally distributed, the 95 per cent confidence limit of design wave height will be 1,645 x standard error above the mean value. This implies that the 95 per cent confidence limit for the 100 year H_{mo} will only be 9,6 per cent above the mean value if the estimate is based on only one year of data. This consistency in the estimate of design wave heights from relatively short data sets explains, to a large degree, why such a clear pattern of design wave heights could be established from data sets of mostly less than two years of data (see Sections 9.4 and 9.5).

10.8 CONCLUSION

(i) The use of a wave height with a given risk of being exceeded during the design lifetime of the structure is recommended for design specifications.

(ii) Calculation of confidence limits shows that the 90 percent confidence limit for the 100 year H_{mo} will reduce to within 10 per cent of the most probable value within 4 years of recording. This is based on the assumption that the Extreme I distribution describes the long term distribution of wave height and that the weekly maximum H_{mo} is independent and identically distributed.

(iii) Analysis of the year to year variation on recorded wave heights substantiate the conclusion in (ii) above and indicate even narrower confidence limits.

11. WAVE PERIOD DISTRIBUTION

A summary of various sources of wave period information was given in Rossouw (1984). Wave periods obtained from visual estimates by clinometer and VOS were described as well as the T_z and T_p parameters extracted from recorded data. Correlations between these parameters were poor and many errors and uncertainties contaminated the data.

Since the publication of Rossouw (1984) a large quantity of high quality Waverider data became available (see Table 5.1). These data were analysed by "Waves" to provide, amongst others, the wave period parameters T_z and T_p.

As explained in Section 4.5, the preferred wave period parameter is T_p. It is realised that T_z is in more general use in many countries (e.g. the United Kingdom) but its sensitivity to the response of the recording instrument and the analysis techniques makes it unreliable. The main reason for its popularity in these countries is the large data banks with hand analysed data where only time domain parameters are available of which T_z is the preferred period parameter.

In this study the wave period distribution around the coast is described using T_p as parameter. Only periods obtained through analysis by "Waves" are considered thereby eliminating the need for the corrections and adjustments required for the older data sets as described in Rossouw (1984).

In this section only the general pattern of wave periods around the coast are described. A more detailed study of the wave periods and spectra associated with design wave conditions is undertaken in Section 15.3.

The distribution of T_p at the six deepwater (depth $\geq$ 50m) Waverider stations from Oranjemund in the west to Cape Recife in the east, is summarised in Table 11.1. Since no deepwater records are available east of Cape Recife, the T_p distributions from the shallower East London and Richard Bay records are also included in this table.

TABLE 11.1 : PERCENTAGE OCCURRENCE OF WAVE PERIOD (T_p)

Wave Period T_p	ORANJEMUND	PORT NOLLOTH	SLANGKOP	SEDCO K	CAPE RECIFE	EAST LONDON	RICHARDS BAY
< 5.22	.49	.22	.55	.45	.41	.34	2.23
5.22	.07	.11	.13	0	.14	.48	.72
5.51	.16	.11	.18	.04	.32	.69	1.09
5.82	.27	.44	.32	.14	.41	1.10	1.05
6.17	.38	.50	.49	.51	.54	1.03	1.12
6.56	.62	.77	.77	.49	.95	1.78	1.86
7.01	.95	.61	1.22	1.46	1.54	1.72	3.06
7.53	2.06	1.10	1.31	2.13	1.59	2.47	3.99
8.13	2.80	1.49	1.72	2.82	2.54	3.84	5.70
8.83	4.11	3.03	3.56	4.83	4.13	5.97	9.23
9.66	8.23	8.65	8.70	8.61	8.07	9.06	11.96
10.67	16.47	19.78	19.16	18.56	15.87	18.94	12.79
11.91	26.90	32.01	29.81	28.03	28.60	23.95	17.27
13.47	24.31	24.24	23.00	22.50	26.61	22.10	20.12
15.52	9.81	6.45	7.88	8.06	7.75	5.70	7.21
18.29	2.15	0.50	1.14	1.18	.54	0.82	0.70
22.26	0.15	0	0.05	0.14	0	0	0.17
> 22.26	0.07	0	0.01	0.04	0	0	0

Exceedance curves of T_p derived from Table 11.1 are shown in Figure 11.1 and the T_p values equalled or exceeded for 90, 50 and 10 per cent of the time are summarised in Table 11.2.

TABLE 11.2 : WAVE PERIOD (T_p) EXCEEDED FOR 90, 50 AND 10 PERCENT OF THE TIME

Station	T_p		
	90%	50%	10%
Oranjemund	9.3	12.6	15.8
Port Nolloth	9.9	12.5	15.2
Slangkop	9.7	12.4	15.5
Sedco K	9.2	12.4	15.5
Gouritzmond	9.3	12.4	15.1
Cape Recife	9.3	12.6	15.3
East London	8.3	12.0	15.0
Richards Bay	7.3	11.5	15.4

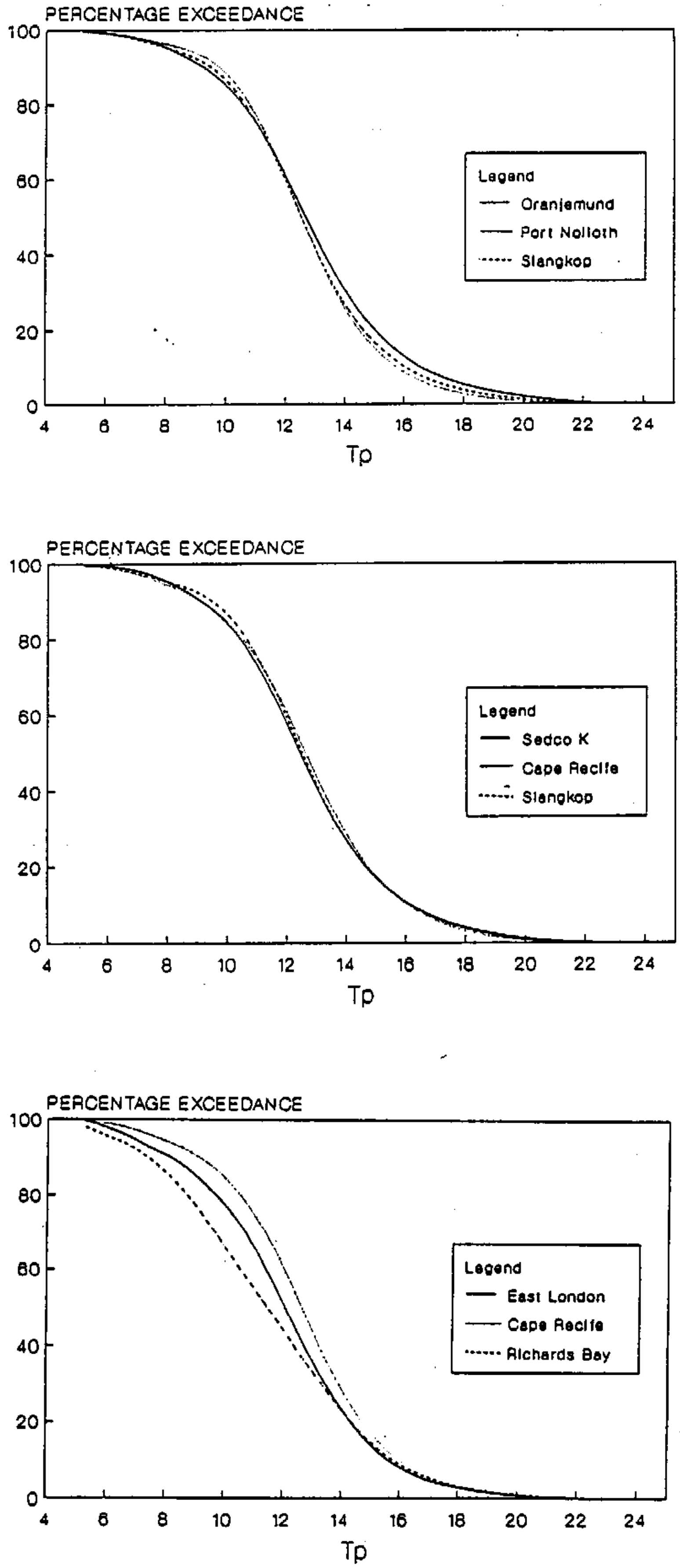


FIGURE 11.1 : EXCEEDANCE OF T_p

The wave period distribution at the six deepwater stations between Oranjemund and Cape Recife are almost identical. The median T_p varies between 12,4 and 12,6s, the T_p exceeded for 90 per cent of the time between 9,3 and 9,7s and T_p exceeded for 10 per cent of the time between 15,1s and 15,8s. These differences are so small that for all purposes a median period of 12,4s and a 80 per cent range of 9,5s to 15,5s can be used for the entire coast between Oranjemund and Port Elizabeth.

East of Port Elizabeth a change in wave period distribution is observed.

The distribution of the longer periods, i.e. T_p values exceeded for less than 30 per cent of the time, is very similar to those for the rest of the coast. Lower wave periods however occur more frequently towards the east, i.e. the median T_p value of 12,6s at Cape Recife reduces to 12,0s and 11,5s at East London and Richards Bay respectively, whereas the T_p exceeded for 90 per cent of the time drops from 9,3s at Port Elizabeth to 8,3s at East London and 7,3s at Richards Bay.

This change of wave period distribution east of Port Elizabeth corresponds to the change in wave height pattern described in Section 9.4.

12. ENERGY DENSITY SPECTRA

12.1 GENERAL

A single wave period parameter such as T_p or T_z is insufficient for describing the periodicity of the waves. A much better understanding of the periodicity of the waves can be gained from their energy density spectra.

In this section the general distribution of wave energy as a function of frequency at two deepsea sites, i.e. Slangkop and Sedco K is described. This description is in terms of average spectra as well as the occurrence of various energy levels at each frequency. Although this should lead to a better understanding of the periodicity of the waves, the information is not particularly useful for design purposes. Selection of spectra for design purposes is discussed in Section 15.4.

The work described in this section is mainly a summary of data contained in Rossouw (1984). No effort was made to update the data sets since no change in the general pattern is expected. The spectral analysis program "Waves" was used in all the analysis. Available data for Slangkop and Sedco K up to February 1981 were used. Record lengths of 512s were treated as a wave record and the number of half records used at each station are shown in Table 12.1.

TABLE 12.1 : NUMBER OF HALF RECORDS USED TO CALCULATE AVERAGE SPECTRA

Station	Recording Period	Number of Acceptable Records
Slangkop	Oct 1978 - Feb 1981	4412
Sedco K	May 1978 - Feb 1981	4707

Summarising this large number of spectra in a meaningful way presented a major problem. A number of attempts to obtain a meaningful summary were made including the use of average spectra, normalised average spectra and the energy density distribution at each frequency. Examples of these summaries are shown below. A more complete summary can be found in Rossouw (1984).

12.2 AVERAGE SPECTRA

The available spectral data for Slangkop up to February 1981 are summarised as average spectra in one diagram, shown as Figure 12.1. The average spectra were calculated by dividing the data into 0,5m wave height (H_{mo}) groups, that is, H_{mo} from 0,0 - 0,5m, 0,5 - 1,0m, 1,0 - 1.5m, etc. and by calculating the average energy density at each frequency for each wave height group. Figure 12.1 shows that a very smooth curve is obtained for each wave height group with the peak frequency decreasing as the wave height increases.

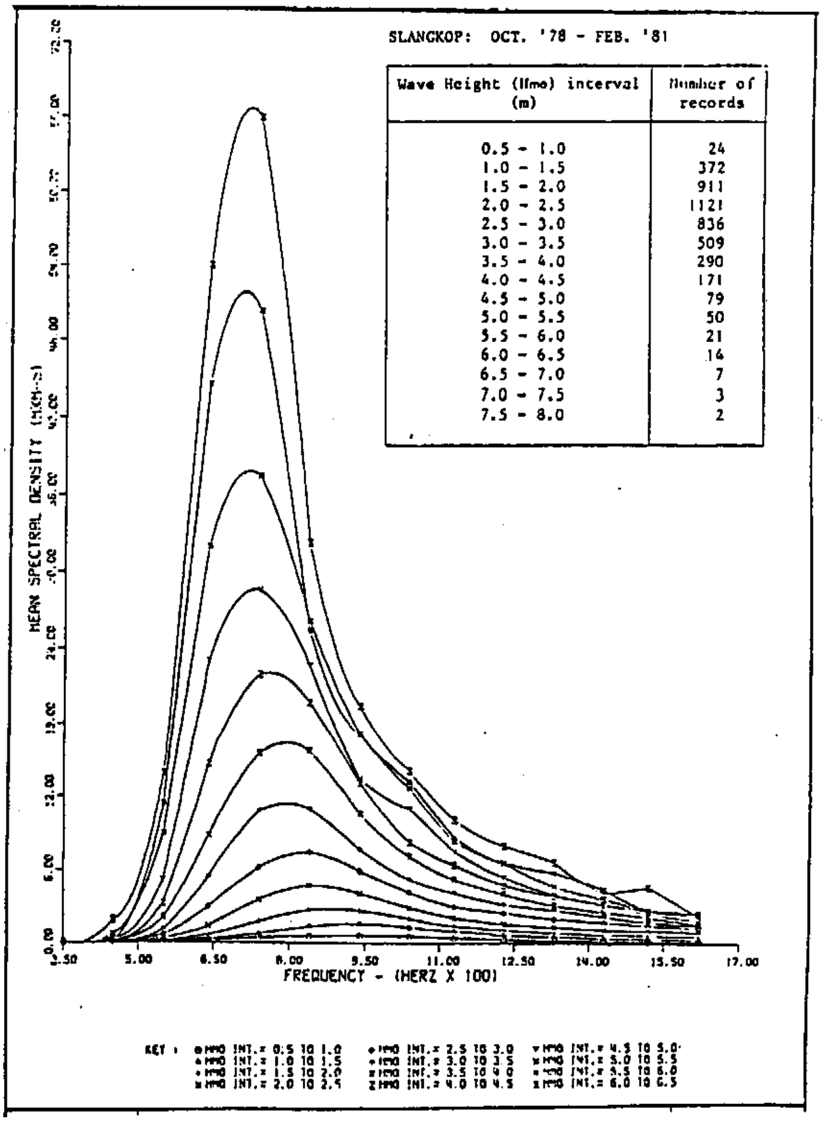


FIGURE 12.1 : AVERAGE SPECTRA - SLANGKOP

12.3 NORMALISED AVERAGE SPECTRA

The spectral shapes for the various wave height groups shown in Figure 12.1 were normalised by dividing the frequencies by the peak frequency and the average energy density at each frequency by the peak energy density. This was done for both stations and the resulting normalised average spectra are shown in Figures 12.2 and 12.3. These figures show that the normalised average spectra for the various wave height groups are very similar.

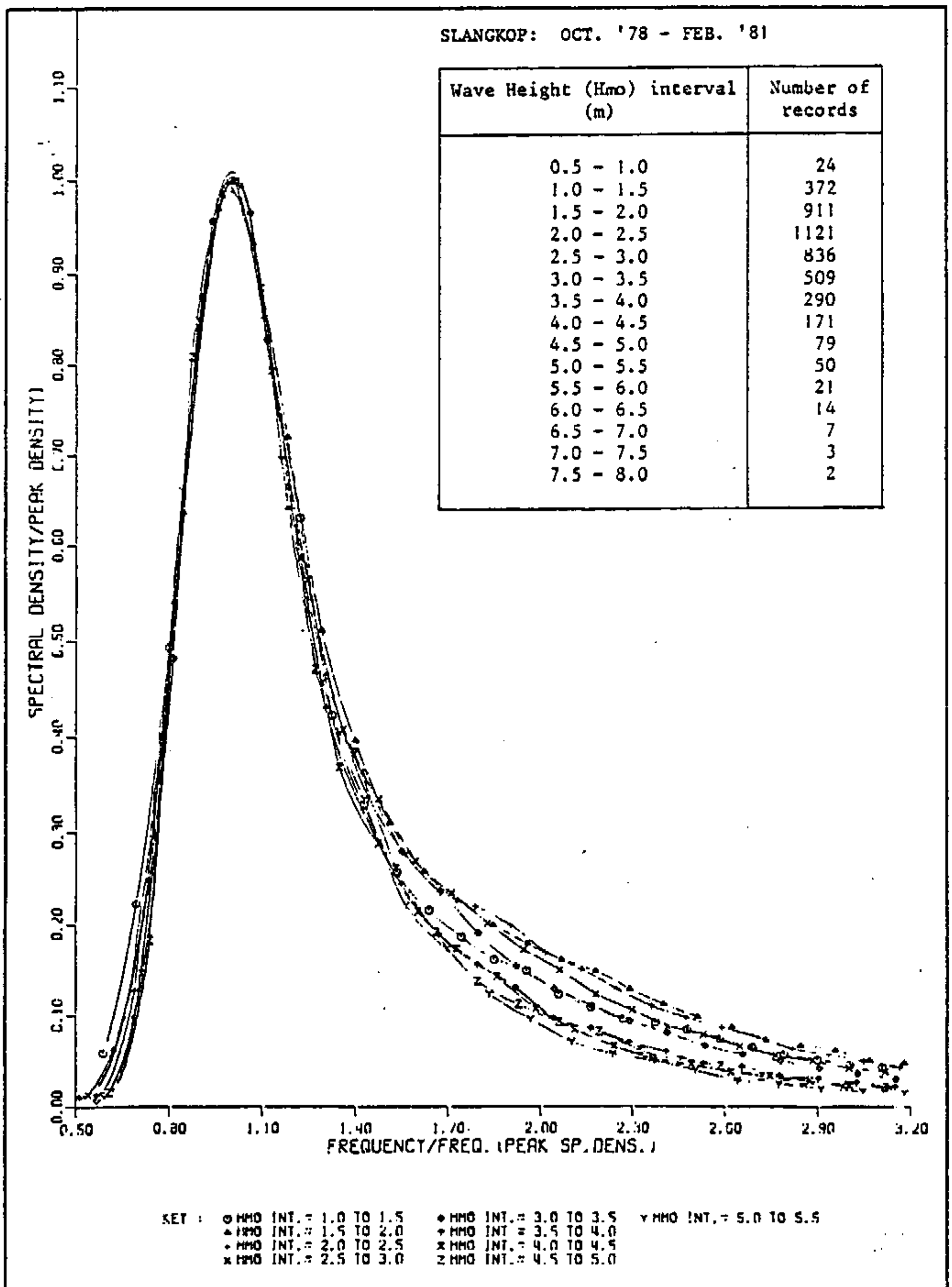
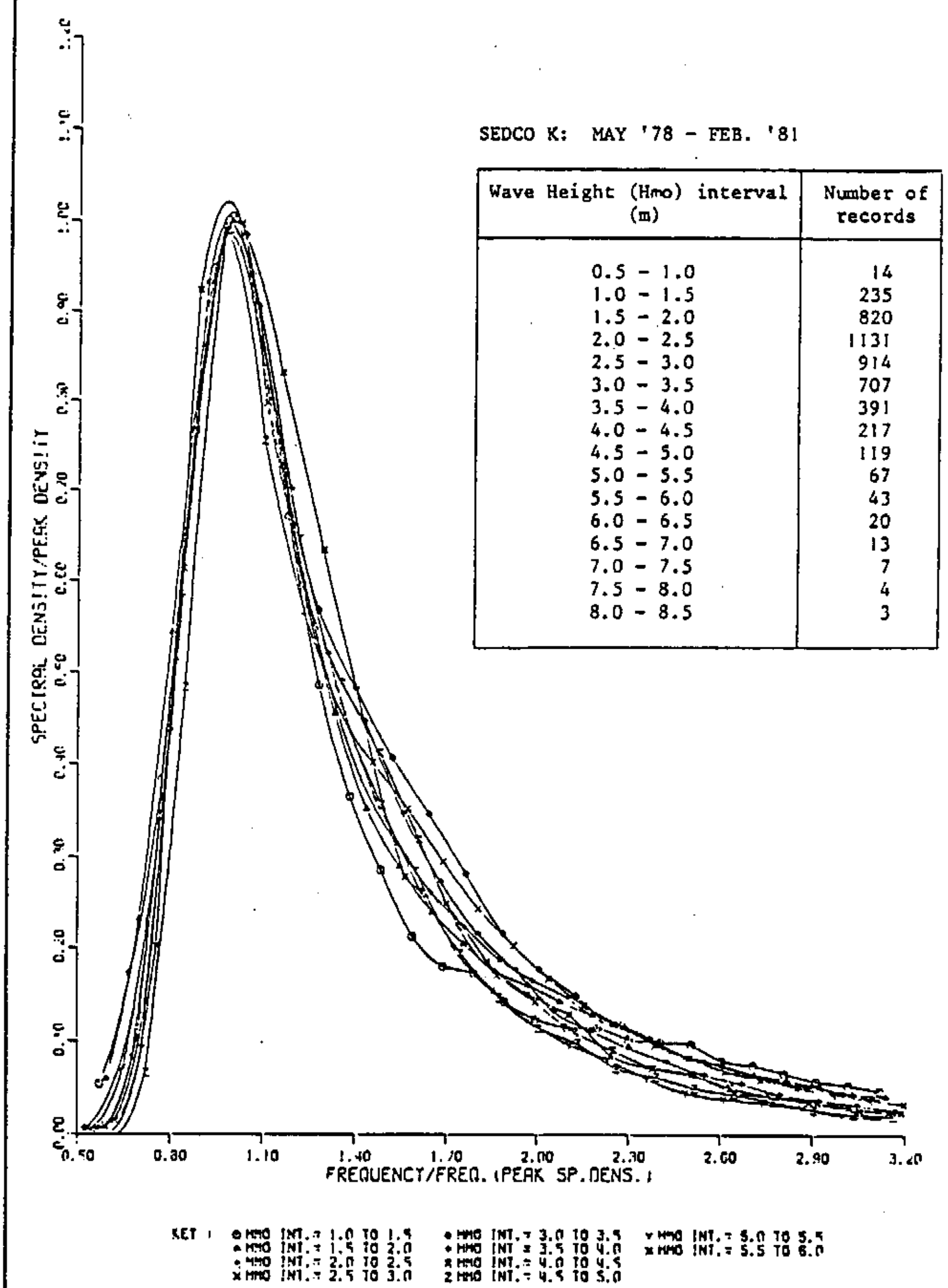


FIGURE 12.2 : NORMALISED AVERAGE SPECTRA - SLANGKOP



Wave Height (Hmo) interval (m)	Number of records
0.5 - 1.0	14
1.0 - 1.5	235
1.5 - 2.0	820
2.0 - 2.5	1131
2.5 - 3.0	914
3.0 - 3.5	707
3.5 - 4.0	391
4.0 - 4.5	217
4.5 - 5.0	119
5.0 - 5.5	67
5.5 - 6.0	43
6.0 - 6.5	20
6.5 - 7.0	13
7.0 - 7.5	7
7.5 - 8.0	4
8.0 - 8.5	3

FIGURE 12.3 : NORMALISED AVERAGE SPECTRA - SEDCO K

The peak energy frequencies and the peak energy densities that were used in the preparation of the normalised spectra were obtained by interpolation from the average spectral shapes and are summarised in Table 12.2. From this Table and Figures 12.2 and 12.3 the average spectra for each station for each wave height group can be reconstructed.

TABLE 12.2 : AVERAGE PEAK ENERGY FREQUENCY AND PEAK ENERGY DENSITY FOR SLANGKOP AND SEDCO K

Wave Height Group (H_{mo}) m - m	Average Peak Energy Frequency (Fp) H_z		Average Peak Energy Density m^2/H_z	
	Slangkop	Sedco K	Slangkop	Sedco K
1,0 - 1,5	0,093	0,096	1,60	1,57
1,5 - 2,0	0,088	0,092	2,80	1,57
2,0 - 2,5	0,085	0,086	4,74	1,57
2,5 - 3,0	0,083	0,084	7,39	1,57
3,0 - 3,5	0,079	0,081	11,32	10,09
3,5 - 4,0	0,079	0,079	16,34	14,70
4,0 - 4,5	0,077	0,076	22,29	20,32
4,5 - 5,0	0,074	0,076	28,56	28,75
5,0 - 5,5	0,072	0,077	37,63	32,77
5,5 - 6,0	0,071	0,072	51,68	37,11

Table 12.2 also shows that, as was to be expected, there is a decrease in frequency with increasing wave height at all the stations. There are also no marked differences in average peak energy frequencies between the stations.

12.4 ENERGY DENSITY DISTRIBUTION PER FREQUENCY

The most satisfactory way found to summarise the large quantity of spectral information was to determine the energy density distribution at each frequency for which energy densities were estimated. The energy density distribution was calculated by dividing the energy density estimates from each record at each frequency, into chosen intervals. By counting the number of occurrences of energy density in each chosen interval and expressing it in parts per 10 000, energy density distributions in the form of Table 12.3 were obtained. These tables were also used to compile the wave energy exceedance tables shown as Table 12.4.

TABLE 12.3 : OCCURRENCE OF SPECTRAL ENERGY DENSITY TO WAVE FREQUENCY

SLANGKOP

SPECTRAL DENSITY (MXM-S)	WAVE FREQUENCY (HERTZ)																			
	.045	.055	.064	.074	.084	.094	.104	.113	.123	.133	.143	.152	.162	.172	.182	.191	.201	.211	.221	TOTAL
0.0- .3	9542	6974	2897	839	233	34	25	661	258	503	834	125	1689	2074	2387	2872	3298	3679	4132	43586
.3- 1.0	340	1723	2860	2142	1036	757	1179	2010	3089	3781	4223	4374	4563	4823	5034	5039	5125	5147	5043	62287
1.0- 2.3	52	560	1310	1442	1469	1997	2840	3434	3343	3230	2996	2845	2634	2212	2024	1750	1453	1106	805	37500
2.3- 4.0	32	231	791	1206	1682	2380	2545	2307	1899	1548	1292	1099	850	725	483	311	122	66	181	19587
4.0- 6.3	14	145	596	959	1575	1850	1700	1226	850	560	447	333	206	150	70	25	2	2	2	10712
6.3- 9.0	5	73	360	848	1278	1242	936	569	347	272	147	73	57	14	2	5				6226
9.0- 12.3	5	86	272	646	907	771	392	231	141	63	43	23	2	2						3583
12.3- 16.0	2	50	195	623	657	453	195	93	43	27	18	2								2359
16.0- 20.3	5	43	161	431	469	272	100	43	16	16										1555
20.3- 25.0	5	36	150	277	279	138	57	14	9											963
25.0- 30.3		14	100	197	188	54	16	5	2											576
30.3- 36.0		29	75	111	88	23	2	2	2											333
36.0- 42.3		11	48	95	59	18	11													243
42.3- 49.0		9	50	48	25	9														141
49.0- 56.3			32	43	27	2	2													107
56.3- 64.0		2	29	23	9															63
64.0- 72.3		5	23	27	7															61
72.3- 81.0		5	16	14	7															41
81.0- 90.3		2	11	11	2															27
90.3-100.0			9	5	2															16
100.0-110.3			5																	5
110.3-121.0			5	5																9
121.0-132.3			2	5																7
132.3-144.0		2		5																7
144.0-156.3																				
>156.3			5	2																7
TOTAL =	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	190000

SEDKO K

SPECTRAL DENSITY (MXM-S)	WAVE FREQUENCY (HERTZ)																			
	.045	.055	.064	.074	.084	.094	.104	.113	.123	.133	.143	.152	.162	.172	.182	.191	.201	.211	.221	TOTAL
0.0- .3	9567	7136	3021	765	142	34	19	47	93	255	467	699	992	1349	1793	2212	2724	3238	3745	38298
.3- 1.0	323	1557	2664	1884	854	497	661	1300	2129	2883	3550	4015	4549	4789	4986	5095	5216	5211	5254	57417
1.0- 2.3	76	508	1368	1528	1421	1583	2473	3100	3340	3263	3010	2947	2766	2636	2405	2237	1808	1445	971	38885
2.3- 4.0	19	306	807	1232	1695	2333	2656	2556	2103	1827	1719	1487	1209	954	714	425	236	1061	281	22411
4.0- 6.3	6	149	514	1111	1593	2065	1863	1447	1173	948	767	608	370	236	891	30	15		2	12985
6.3- 9.0	2	106	323	816	1290	1389	1143	701	593	474	319	189	96	36	13	2	2			7493
9.0- 12.3	2	93	283	712	1016	933	537	376	291	217	123	38	19							4640
12.3- 16.0		34	287	571	678	467	285	221	149	96	38	17								2843
16.0- 20.3	2	17	193	361	487	317	155	130	57	28	6									1753
20.3- 25.0	2	25	176	312	310	172	93	55	45	6										1198
25.0- 30.3		6	89	187	200	91	42	40	21	4										682
30.3- 36.0		30	57	159	125	64	32	17	6											491
36.0- 42.3		2	64	115	83	23	21	61												314
42.3- 49.0		2	28	93	40	11	13	2												189
49.0- 56.3			32	49	34	8	4	2												130
56.3- 64.0		6	21	36	15	6														85
64.0- 72.3		4	23	20	6	6														72
72.3- 81.0			11	15	4															30
81.0- 90.3		2	17	4																23
90.3-100.0		2	2	8	4															17
100.0-110.3		2	11	4	2															19
110.3-121.0			2	4																6
121.0-132.3		2	6																	8
132.3-144.0		6																		6
144.0-156.3				2																2
>156.3		2																		2
TOTAL =	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	190000

TABLE 12.4 : EXCEEDANCE OF SPECTRAL ENERGY DENSITY TO WAVE FREQUENCY

SLANGKOP

SPECTRAL DENSITY (HXH-S)	WAVE FREQUENCY (HERTZ)																			
	.045	.055	.064	.074	.084	.094	.104	.113	.123	.133	.143	.152	.162	.172	.182	.191	.201	.211	.221	TOTAL
> 0.0	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	190000
> .3	458	3026	7103	4161	9767	9966	9975	9934	9742	9497	9166	8749	8311	7926	7613	7128	6702	6321	5868	146414
> 1.0	118	1303	4243	7019	8731	9209	8796	7924	6652	5716	4943	4374	3749	3103	2579	2090	1578	1174	825	84127
> 2.3	66	743	2933	5578	7262	7212	5956	4490	3309	2486	1947	1530	1115	8911	555	340	125	68	201	46627
> 4.0	34	512	2142	4372	5580	4832	3411	2183	1410	938	655	431	265	165	73	29	2	2	2	27040
> 6.3	20	367	1546	3413	4005	2983	1711	956	560	379	209	971	59	16	2	5				16328
> 9.0	16	295	1185	2566	2727	1741	775	388	213	107	61	25	2	2						10102
> 12.3	11	209	913	1920	1820	970	383	156	73	43	18	2								6519
> 16.0	9	159	718	1296	1163	517	188	63	29	16										4159
> 20.3	5	116	558	866	694	245	88	20	14											2604
> 25.0		79	408	589	415	107	32	7	5											1641
> 30.3		66	308	392	227	52	16	2	2											1065
> 36.0		36	233	281	138	29	14													732
> 42.3		25	186	186	79	11	2													490
> 49.0		16	136	138	54	2	2													349
> 56.3		16	104	95	27															243
> 64.0		14	75	73	18															179
> 72.3		9	52	45	11															118
> 81.0		5	36	32	5															77
> 90.3		2	25	20	2															50
> 100.0		2	16	16																34
> 110.3		2	11	16																29
> 121.0		2	7	11																20
> 132.3		2	5	7																14
> 144.0			5	2																7
> 156.3			5	2																7

SEDCO K

SPECTRAL DENSITY (HXH-S)	WAVE FREQUENCY (HERTZ)																			
	.045	.055	.064	.074	.084	.094	.104	.113	.123	.133	.143	.152	.162	.172	.182	.191	.201	.211	.221	TOTAL
> 0.0	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	10000	190000
> .3	433	2864	6979	8235	9858	9966	9981	9953	9907	9745	9533	9301	9008	8651	8207	7788	7276	6762	6255	151702
> 1.0	110	1307	4315	7351	9004	9469	9320	8653	7778	6862	5983	5286	4459	3862	3221	2694	2061	1551	1001	94285
> 2.3	34	799	2947	5823	7582	7886	6847	5553	4438	3599	2972	2339	1693	1226	8161	457	253	106	30	55400
> 4.0	15	493	2139	4581	5887	5553	4192	2998	2335	1772	1253	8521	4841	272	102	32	17		2	32989
> 6.3	8	344	1625	3480	4294	3488	2328	1551	1162	824	487	244	115	36	13	2	2			20004
> 9.0	6	238	1302	2664	3004	2099	1185	850	569	351	168	55	19							12511
> 12.3	4	144	1020	1952	1989	1166	648	474	278	134	45	17								7871
> 16.0	4	110	733	1381	1311	699	363	253	130	38	6									5029
> 20.3	2	93	540	1020	824	382	208	123	72	11										3276
> 25.0		68	363	707	514	210	115	68	28	4										2078
> 30.3		62	274	521	314	119	72	28	6											1396
> 36.0		32	217	361	189	55	40	11												905
> 42.3		30	153	246	106	32	19	4												591
> 49.0		28	125	153	66	21	6	2												402
> 56.3		28	93	104	32	13	2													272
> 64.0		21	72	68	17	6	2													187
> 72.3		17	49	38	11															115
> 81.0		17	38	23	6															85
> 90.3		15	21	19	6															62
> 100.0		13	19	11	2															45
> 110.3		11	8	6																25
> 121.0		11	6	2																19
> 132.3		8		2																11
> 144.0		2		2																4
> 156.3		2																		2

Table 12.3 shows the distribution of wave energy at each frequency whereas Table 12.4 shows the fraction of the time for which certain energy levels will be exceeded in an average year.

Inspection of Table 12.4 reveals that the higher energy levels always occur in the frequency range 0,055 to 0,104 hertz or in the wave period range 10s to 18s. The highest energy level that is exceeded for 1 per cent of the time (100 parts per 10 000) occurs at frequencies of 0,064 and 0,074 or wave periods of 13s and 16s at Slangkop and Sedco K respectively.

12.5 RESPONSE CORRECTIONS

As mentioned earlier, no correction for the decrease in amplitude response of the Waverider at lower frequencies are made in the standard wave analysis program "Waves". As was shown in Figure 3.1, the amplitudes recorded at frequencies lower than 0,07 H_z (or 14s) are underestimated by the Waverider. To see what influence this reduced response at low frequencies has on the recorded wave heights, response corrections were applied to a number of spectra.

The response corrections were made by dividing the energy density S(f) at each frequency by the square of the amplitude response at that frequency. A new mo was calculated by summation of the new S(f) values and multiplying by Δf.

It was found that when using the average spectra that the response corrections increased the H_{mo} values by up to 1,8 per cent for the larger H_{mo} values (i.e. 5,5m > H_{mo} > 5m). Obviously the influence of the response correction will increase with increasing T_p values. The storm of 2 September 1978, described in Section 6.4.2, recorded a H_{mo} = 8,67m with T_p = 18,3s at the peak of the storm. Response corrections to this exceptionally long wave period storm had the influence of increasing the wave height H_{mo} to 8,97m or by 3,5 percent.

Although the influence of the response corrections on the desired wave heights will be small (i.e. most probably less than 2 per cent) no reason exists why response correction should not be applied as a standard procedure in the analysis process.

13. WAVE DIRECTION DISTRIBUTIONS

Three sources of wave direction information are considered here. All these sources are based on visual estimates of wave direction. The major source that are used is the wave direction estimates from Voluntary Observing Ships. These will be augmented by visual direction estimates made from a Weathership that was stationed south-west of the continent at 40°S, 10°E between September 1972 and August 1973.

A more detailed account of these and other sources of wave direction data can be found in Rossouw (1984).

The most useful summary of VOS data for the South African coast is that by Swart and Serdyn (1981) and Swart and Serdyn (1986). Swart and Serdyn divided the coast into 1° x 1° squares and in areas where records were insufficient, combined squares to give better data coverage.

For the purposes of this study, data from the areas shown in Figure 13.1 were used. The direction distribution for these areas was extracted from the tables of Swart and Serdyn and are summarised in Table 13.1. Roses drawn from this table are shown in Figure 13.2.

Wave roses obtained from the Weathership data contained in WSAC (1973) are also included in Figure 13.2.

The pattern of observed wave directions is very much in keeping with the weather patterns described in Section 6.3. At the position of the Weathership the directions are dominated by the Ferrel westerlies and the most frequently occurring directions are WNW, W and WSW. Moving north from the Weathership up the west coast, the occurrence of waves from north of west gradually reduces and virtually disappears north of Oranjemund. The occurrences of southerly waves increase towards the north as the influence of the South Atlantic high increases. North of Luderitz there is a drastic reduction in the waves from the W to the SW sector and virtually all waves fall in the SW to SE sector with the southerly waves totally dominant in the areas adjacent to the coast. Further offshore (i.e. areas 62 to 72) the southeasterly waves dominate as the protection afforded by the land mass reduces. An increase in wave height with increasing distance offshore can therefore also be expected in this area.

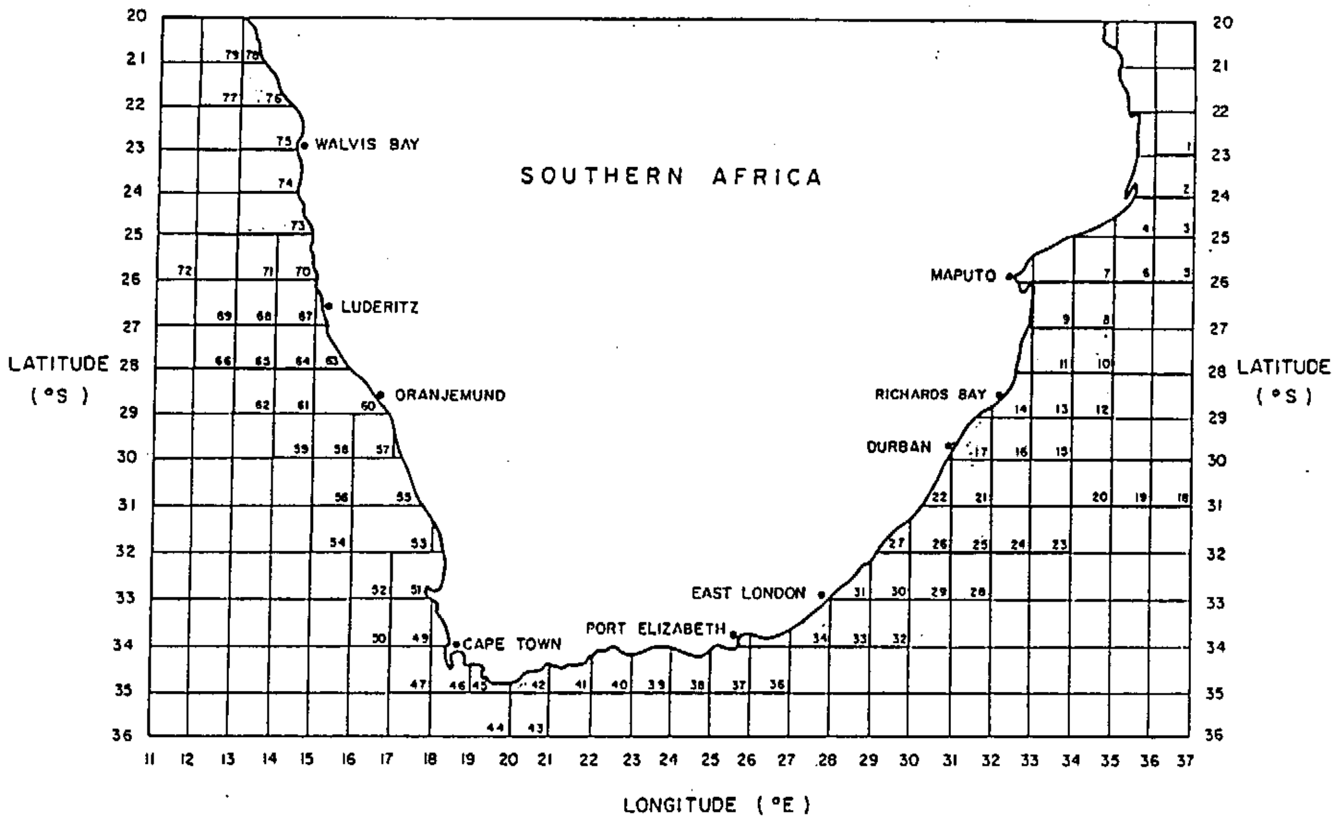


FIGURE 13.1 : AREA USED IN ANALYSIS OF VOS DATA

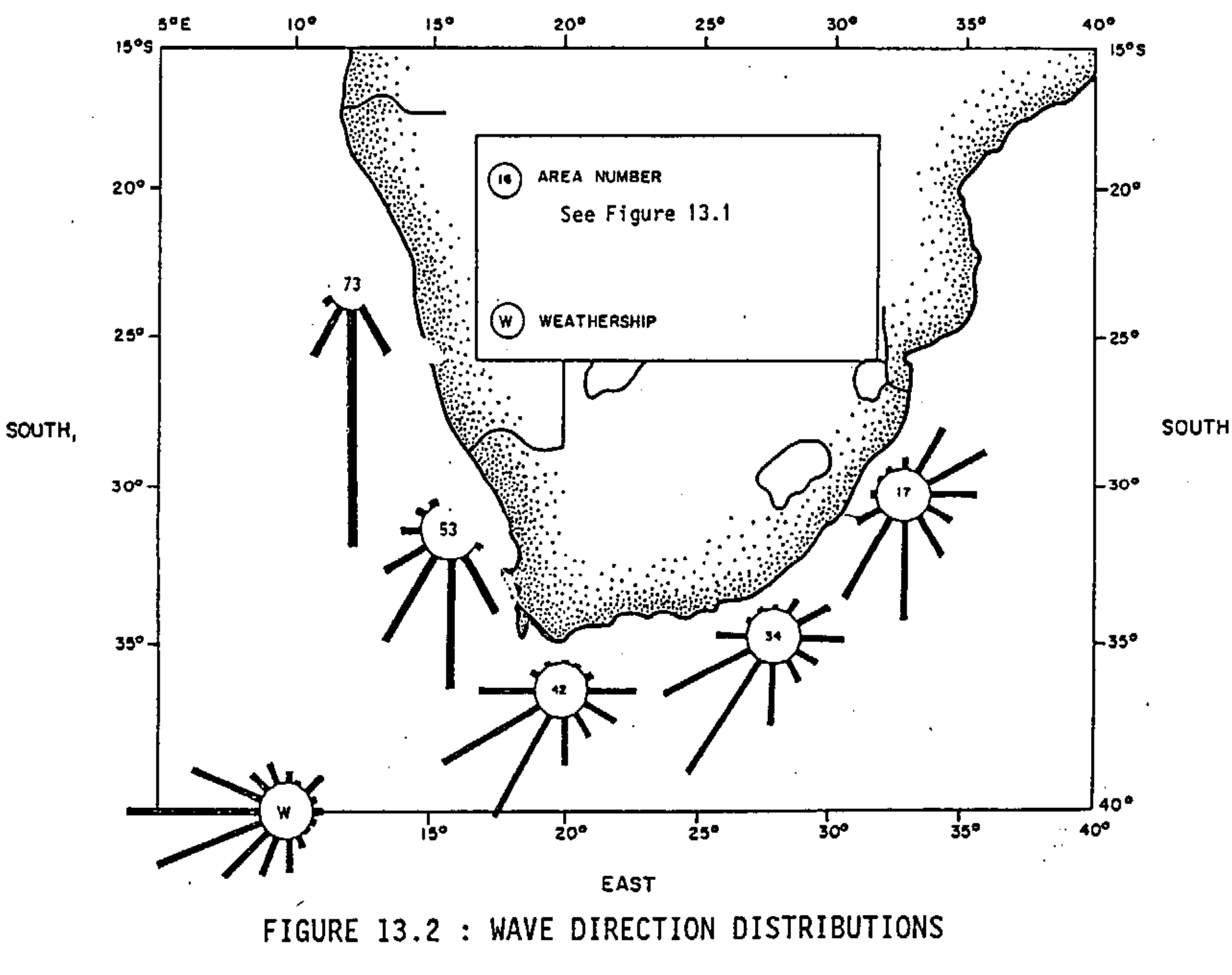


FIGURE 13.2 : WAVE DIRECTION DISTRIBUTIONS

TABLE 13.1 : PERCENTAGE OCCURRENCE OF WAVE DIRECTION - VOS DATA

Area (see Figure 13.1)	76	73	72	67	62	53	49	42	34	17
Latitude Longitude	21-22 13-15	24-25 13-15	25-26 11-12	26-27 14-15	28-29 13-14	31-32 16-18	33-34 17-18	34-35 20-21	33-34 27-28	29-30 31-32
Wave Direction										
20 - 40	0,38	0,34	0,07	0.26	0.11	0.26	0.57	0.39	2.16	8.98
50 - 70	0,49	0,19	0,07	0.24	0.07	0.19	0.30	1.14	6.03	11.75
80 - 100 (E)	0,30	0,15	0,12	0.19	0.21	0.50	0.36	8.21	7.75	7.80
110 - 130	0,53	0,19	2,47	0.38	2.72	1.26	1.53	6.33	3.78	4.65
140 - 160	16,11	13,49	44,94	17.94	33.00	16.86	14.55	4.73	4.41	8.18
170 - 70 (S)	51,59	61,28	28,58	34.43	29.99	32.81	26.50	8.80	11.08	17.60
200 - 220	19,48	13,92	14,26	23.20	18.10	24.48	23.03	20.65	23.68	16.66
230 - 250	1,82	2,09	4,77	10.71	8.12	11.52	15.26	20.36	17.86	4.07
260 - 280 (W)	0,68	0,27	1,85	4.38	3.07	4.25	7.05	10.04	5.47	0.66
290 - 310	0,68	0,15	0,35	0.52	0.86	2.28	3.02	1.52	0.63	0.17
320 - 340	0,95	0,46	0,32	0.83	0.62	1.67	2.31	0.56	0.34	0.39
350 - 010 (N)	0,68	1,60	0,10	2.39	0.29	0.26	1.26	0.13	0.48	1.91
Undefined	6.24	5.89	2.08	4.53	2.84	3.22	4.27	17.12	16.32	17.18
TOTAL OBSERVATIONS	2638	2632	4045	4220	10453	7979	8907	7646	10672	5181

Moving east from the Weathership towards the Cape south coast, there is a gradual swing in direction from the dominant westerlies towards a dominance of southwesterly waves. The directions north of the E-W line are cut off by the land mass and an increase in easterly conditions occurs. This pattern continues further towards the east with the dominant directions swinging towards the south along the Natal north coast. The occurrence of easterly and northeasterly waves also increases at the more easterly locations.

The pattern for the entire coast is totally compatible with the known weather patterns. It also clearly indicates a resultant energy from south to north along the west coast and from west to east along the south and east coasts. This is compatible with the known sediment transport directions along these coasts.

14. GENERAL DISCUSSION ON THE SOUTH AFRICAN WAVE CLIMATE

From the data and analyses presented in the preceding sections, a very clear and thorough understanding of the general wave climate for the South African coast especially for the area between Oranjemund and Port Elizabeth emerges. Along this part of the coast all the major wave events are caused by the regular passage of cold fronts from west to east past the southern tip of the continent. The effect of the more severe events can easily be traced to all wave recording sites between Oranjemund and Port Elizabeth with a reduction in wave height towards the northern latitudes. This pattern is so consistent that an extremely simple model could be devised with which the design wave heights along this coastline could be predicted to within a few per cent from only the latitude of the site.

The very regular passage of these cold fronts at intervals of between 3 and 5 days means that independent events occur very regularly. This regularity of independent events plus the extremely consistent pattern of derived design wave heights around the coast, indicate that if long term changes in the climate do not occur, wave heights with recurrence intervals of up to 100 years can most probably be predicted with accuracies within the order of 10 per cent from wave records of only a few years.

Insufficient data are available north of Oranjemund and east of Port Elizabeth to have the same faith in predictions of design wave heights in these areas. Along the northern Natal coast the occurrence of cyclones further complicates the picture and special studies will be required to estimate design waves in this area.

The distribution of wave period from Oranjemund to Port Elizabeth is virtually identical with peak energy periods (T_p) in the range of 9s to 16s for 80 per cent of the time and a median T_p of around 12,5s. East of Port Elizabeth the wave periods reduce slightly and this reduction continues all the way to Richards Bay in the east.

Wave directions also show a very clear pattern around the coast which is totally compatible with the known weather patterns. The Ferrel westerly wind belt south of the South African mainland causes dominant westerly waves south of the country at latitudes around 40°S. The irregular passage of cold fronts in this westerly system causes wave directions to swing from NW to SW along the south western coast during the passage of the front. Further north along the west coast the north-westerly waves disappear as the direct influence of the cold fronts towards the south diminish. The SW component of the waves generated by the cold front do, however, reach this coast resulting in a dominance of south-westerly waves along this coast. Still further north the influence of the semi-permanent south Atlantic high causes the waves to become more southerly along the coast and south-easterly further away from the coast.

Wave directions along the Cape south coast up to Port Elizabeth are also totally dominated by the passage of the cold fronts. South-westerly waves dominate along this coastline with a gradual swing towards the south as one moves towards the east. East of Port Elizabeth the cold fronts are mostly deflected away from the coast due to the presence of the south Indian Ocean high. The coastline also swings more towards the north. Here smaller systems like coastal lows dominate and a gradual swing of the wave direction towards the south occurs coupled with an increase in easterly and north-easterly wave conditions.

15. SELECTION OF DESIGN WAVE CONDITIONS

15.1 GENERAL

In this section a few examples are given of how the data contained in this report can be used to select wave conditions for design purposes. The main emphasis will be on the selection of an appropriate design wave height (H_{mo}) although aspects such as maximum individual wave height, groups of waves, wave period and wave spectra associated with design wave height are also discussed. The latter aspects are based mainly on some exploratory analysis of major storms contained in the data sets and serve mainly to show how this data can be used to give guidance in this repect. Specific selection of design conditions at a site can only be based on a more detailed analysis of the local data taking full cognisance of the design problem at hand.

The examples given are applicable to the Cape west and south coast where a sufficient understanding of the wave climate exists. The Natal coast is excluded from these examples since a totally different approach will be required due to the occurrence of cyclones along this coast.

15.2 SELECTION OF DESIGN WAVE HEIGHT

15.2.1 Significant Wave Height (H_{mo})

An estimate of the maximum H_{mo} that can be reasonably expected during the design lifetime of the structure is a basic requirement in the design of virtually any offshore or coastal structure. Here the designer and his client must agree on the risk (r) they are willing to tolerate of the design H_{mo} being exceeded during the design life (h) of the structure. Assume, for example, r = 0,10 and h = 20 years. The next step is to estimate the parameters of the distribution describing the long term distribution of wave height. If data for the site are available it is suggested that the method of moments (described in Section 8.2) should be used to estimate the parameters of the Extreme I distribution using all available six hourly records at the site. For a deepsea location between Oranjemund and Cape

Recife the simple model described in Section 9.5 could be used to check these parameters. Let us assume that design conditions are required for a station directly south of Cape Agulhas, say at 35°S, 20°E. The parameters are estimated as follows:

$$\beta = 0{,}0366 \times (\text{latitude}) - 0{,}4509$$
$$= 0{,}0366 \times 35 - 0{,}4509 \qquad = 0{,}83$$
$$\alpha = 0{,}5492 + 1{,}9568\,\beta \qquad = 2{,}17$$

The percentage points of the Extreme I distribution are then given by

$$H_{mo}(p_h^r) = \alpha - \beta \ln(-\ln p_h^r)$$
$$= 2{,}17 - 0{,}83 \ln(-\ln p_h^r)$$

The probability of non exceedance of H_{mo} is given by

$$p_h^r = (1 - r)^{1/(365x4xh)}$$
$$= (1-0{,}1)^{1/(365x4x20)} = 0{,}9999964$$

$$H_{mo}(.9999964) = H^{0.1}_{mo_{20yrs}}$$
$$= 2{,}17 - 0{,}83 \ln(-\ln .9999964)$$
$$= 12{,}57\text{m}$$

If the H_{mo} value derived from data at the site differs significantly from the value derived from the simple model, the data require careful scrutiny. Unless a good reason can be found for the difference, the use of the higher of the two values is recommended. Places where a legitimate reduction in height can be expected is the area between Cape Agulhas and Port Elizabeth north of 34°50'S. Here some protection from the westerly conditions will occur and the parameters for Gouritzmond and Cape Recife, i.e. $\alpha = 2{,}05$, $\beta = 0{,}67$ should be more appropriate.

15.2.2 Maximum Individual Wave Height (H_{max})

For many designs it is necessary to know the maximum individual wave height that is associated with the design significant wave height selected in Section 15.2.1 above. This height is normally estimated

by assuming that the significant wave height remains constant for the six hour interval between records and that the individual wave heights during this six hour period are Rayleigh distributed,
i.e. $p\ (H > \hat{H}) = e^{-2(\hat{H}/H_{mo})^2} = n/N$ ....Eq. 15.1

The maximum wave in six hours therefore corresponds to $p = 1/N$ where N is the total number of waves in six hours, calculated via T_z. Using this approach the ratio H_{max}/H_{mo} for six hours is tabulated in Table 15.1 below.

TABLE 15.1 : Ratio H_{max}/H_{mo} from Rayleigh distribution

T_z (s)	8	10	12	14
H_{max}/H_{mo}	1.99	1.96	1.94	1.92

Several researchers have however found that the Rayleigh distribution over-estimates the maximum wave height. A reduction factor K has been proposed by Longuet-Higgins (1980) as follows :
$p\ (H > \hat{H}) = e^{-2(\hat{H}/(H_{mo} \cdot K))^2}$...........Eq. 15.2
with K = 1 for narrow band seas
and K = 0,885 for broad band seas

Analyses of the distribution of individual waves during a number of the more extreme events along the Cape south coast were also undertaken. Using the Statsgraphics package available on an IBM compatible desk top computer, a two parameter Weibull distribution was fitted to the individual wave heights. The average parameters from approximately 20 records were used to derive the expression

$p(H > \hat{H}) = e^{-2.48(\hat{H}/H_{mo})^{2.08}}$..Eq. 15.3

This closely corresponds to Equation 15.2 above if a value of K = 0,9 is used.

The maximum wave heights derived from Equation 15.3 are shown in Table 15.2.

TABLE 15.2 : RATIO H_{max}/H_{mo} FROM EQUATION 15.3

T_z (s)	8	10	12	14
H_{max}/H_{mo}	1.75	1.72	1.70	1.68

Using Equation 15.3 to obtain H_{max} values will result in H_{max} values that are an average about 14 per cent lower than when a Rayleigh distribution is used.

Before recommending a conversion factor from H_{mo} to H_{max}, the assumption of a constant H_{mo} during the six hour period needs to be investigated. Unfortunately no continuous records during storms are available which could be used for this purpose. The rapid change in H_{mo} that often occurs with the duration limited storms, examples of which were given in Figures 6.4 to 6.11, illustrate that this assumption is questionable. The variation of H_{mo} of up to 26 per cent during two successive 512s periods shown in Table 8.4 place further doubts on this assumption. These two factors indicate that H_{mo} values higher than those recorded at six hour intervals can be expected during the six hour gap between records.

It is therefore recommended to take a conservative approach and use H_{max} = 2 H_{mo} until such time as continuous records are obtained during storms. Such records will enable a much more realistic recommendation to be made.

It is interesting to note that Nolte (1973) does away with the assumption of constant H_{mo} values over a six hour period. He allows a linear variation in wave heights between the recorded values. This assumption does, however, not allow for higher H_{mo} values during the gap in recording and should lead to an underestimation of the maximum H_{mo} value occurring during the storm.

15.2.3 Heights of Groups of Successive Waves

In some design work especially where resonance between the structure and the waves can be expected, the occurrence of groups of high waves can become an important design consideration. This so-called groupiness of the waves has received much attention recently and has led to the definition of a number of groupiness parameters in the analysis of waves.

The standard analysis program "Waves" that was used in all the data sets contained in this report did not include any groupiness analysis. Some exploratory analysis of wave groups as they occurred during a few of the major storms were however undertaken.

As a first step the mean wave height of the highest one, two, three, four and five consecutive individual waves were calculated as they occurred in a 1024s record. This analysis was based on 31 records in which H_{mo} exceeded 6,0m. The results are summarised in Table 15.3 below.

TABLE 15.3 : MEAN AND STANDARD DEVIATION OF HEIGHT OF WAVE GROUPS IN 1024s RECORD

Number of Waves in Group	Average Group Height / H_{mo}	
	Mean	Standard Deviation
1	1,428	0,110
2	1,266	0,111
3	1,131	0,105
4	1,051	0,092
5	1,004	0,093

When the number of waves in the group equals one, the wave height obviously equals H_{max}. It is interesting to note that if Equations 15.2 and 15.3 are used to calculate H_{max}/H_{mo} for T_z = 12s and a 1024 record length, values of 1,49 and 1,33 are obtained. The Rayleigh distribution (Equation 15.2) therefore comes closer in predicting the observed H_{max}/H_{mo} ratio of 1,43m given in Table 15.3.

It is also interesting to note that on average the mean height of 5 consecutive waves exceeded H_{mo} during these storms.

In the above analysis no effort was made to select particularly groupy waves. More groupiness can be expected from narrow band seas. To study groupiness of such seas 19 records from narrow banded seas in which H_{mo} varied from 3,8m to 7,5m were selected. A group was defined as a number of consecutive waves which all exceeded H_{mo} and it was found by Moes in CSIR (1988) that on average these groups consisted of three waves which had an average H_{group}/H_{mo} ratio of 1,18 with a standard deviation of 0,11. This ratio is only slightly higher than the value of 1,13 given in Table 15.3 above for groups of three waves.

It is important to realise that the groups discussed above all occurred in a 1024s wave record. Higher groups can obviously be expected during the six hour period for which the H_{mo} is assumed to be stationary. A proper estimate of the highest group to be expected in six hours will only be possible after continuous records during storms become available.

Nelson et al (1988) recommends the use of 80 minute record lengths as a compromise between the minimum length required for groupiness analysis and the maximum length during which waves can be considered stationary.

A conservative approach that is consistent with the recommendation to use $H_{max.}$ = 2 H_{mo} in Section 15.2.2 would be to multiply the average group heights of Table 15.3 with a factor 2.0/1.43 to obtain the average group heights for a six hour period.

15.3 SELECTION OF DESIGN WAVE PERIODS

15.3.1 Period Associated with design H_{mo}

The bivariate H_{mo} - T_p distribution for the data set described in Section 7.2 was given in Appendix C3. Efforts by Button (1988) to fit mathematical expressions to the distribution was not very successful and extrapolation of these fitted distributions to obtain wave periods that will correspond to typical values of design wave heights (H_{mo}) were totally unsatisfactory.

In an effort to obtain estimates of the most likely range of T_p values to be associated with design H_{mo} values, the mean and standard deviation of T_p was calculated for each wave height group in Appendix C3. A linear regression of T_p^2 as function of H_{mo} with H_{mo} the independent variable for $1{,}0m < H_{mo} < 6{,}5m$ gave the relationship

$$T_p^2 = 94{,}34 + 15{,}91\ H_{mo}$$

with a correlation coefficient of 0,988. The data and the fitted relationship is shown in Figure 15.1.

The reason for correlating H_{mo} with T_p^2 is that wave steepness is a function of H_{mo}/T_p^2 and since wave steepness has a theoretical upper bound it was hoped that some guidance as to the range of T_p could be obtained from this correlation.

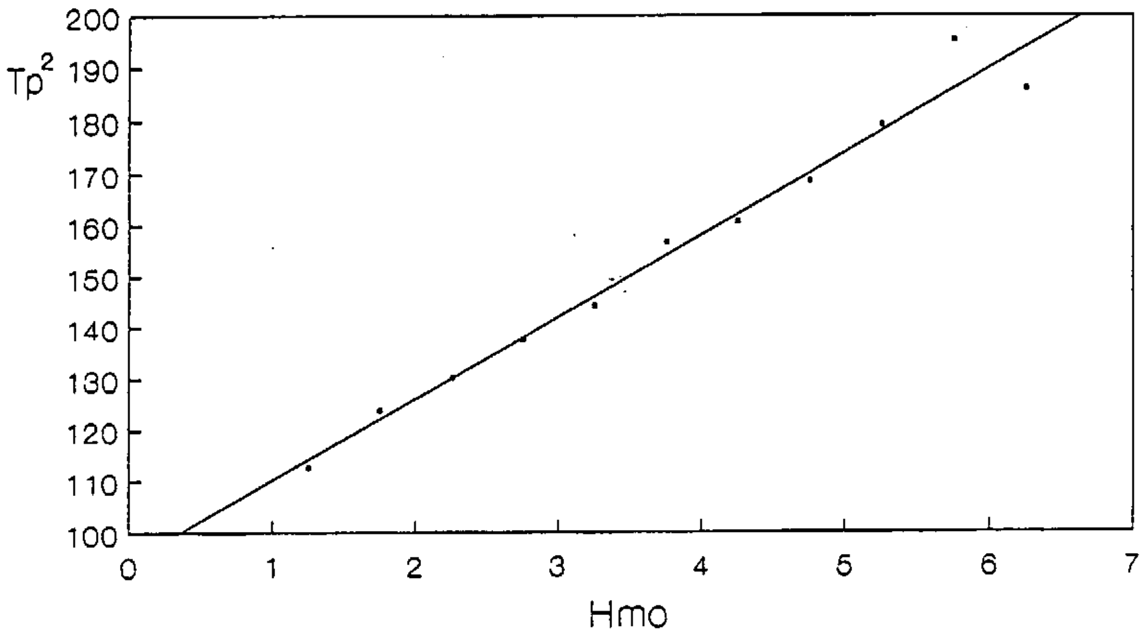


FIGURE 15.1 : CORRELATION BETWEEN T_p^2 AND H_{mo}

The mean and standard deviation of T_p as a function of H_{mo} is shown in Table 15.4. There is a tendency for the standard deviation to decrease with increasing H_{mo} but the number of records at heights exceeding 6m are insufficient to make definite conclusions. The poor resolution of T_p at higher T_p values (see Section 4.4) also complicates the issue. If the standard deviation of 2,16s calculated for all H_{mo} values in the range 1 to 6,5m is also accepted for the higher wave heights, and if it is assumed that T_p at each wave height is normally distributed, the range of T_p that will contain 95 percent of the data will be given

by $\quad T_p \pm 1{,}96\sigma$

or $\quad T_p \pm 4{,}2s$

TABLE 15.4 : MEAN AND STANDARD DEVIATION OF T_p

H_{mo} (m)	T_p (s)	σ T_p (s)
1,0 - 1,5	10,61	2,16
1,5 - 2,0	11,13	2,12
2,0 - 2,5	11,41	2,17
2,5 - 3,0	11,74	2,24
3,0 - 3,5	12,01	2,24
3,5 - 4,0	12,51	2,10
4,0 - 4,5	12,67	2,36
4,5 - 5,0	12,98	1,82
5,0 - 5,5	13,38	1,16
5,5 - 6,0	13,97	0,92
6,0 - 6,5	13,63	1,75

Estimates of the most probable value of T_p and the range of T_p to be associated with the H_{mo} value of 12,57m in the example of Section 15.2.1 are

$$\hat{T}_p = 17{,}1\text{s} \quad \text{and } 12{,}9\text{s} < T_p < 21{,}3\text{s}$$

This agrees favourably with the range of periods specified by Del Norske Veritas (1977) of

$$3{,}6\ \sqrt{H_{mo}} < T_p < 5{,}5\ \sqrt{H_{mo}}$$

or $12{,}6 < T_p < 19{,}5\text{s}$ for our example.

The choice of the most probable wave period and the range of wave periods to be associated with the design H_{mo} are mostly not critical design parameters.

15.3.2 Period Associated with the Maximum Individual Design Wave and Groups of High Waves

The period ($T_{H_{max}}$) associated with the maximum individual wave height (H_{max}) is normally very similar to the peak energy period T_p. Bell (1971) suggests the relationship $T_{H_{max}} = 1{,}01\ T_p$.

Similarly the average period of a group of high waves in a record (T_G) is also expected to be similar to T_p. Analysis by Moes in CSIR (1988) show that the average value of T_G/T_p for 25 selected records for groupiness analysis (see Section 15.2.3) is 0,985 with a standard deviation of 0,045.

For all practical purposes it will be acceptable to use

$$T_{H_{max}} = T_G = T_p$$

The range of $T_{H_{max}}$ and T_G can therefore be taken to be the same as the range of T_p and the specified range of

$$3{,}6\ \sqrt{H_{mo}} < T_p < 5{,}5\ \sqrt{H_{mo}}$$

by Det Norske Veritas (1977) will be suitable for South African conditions.

If $H_{max} = 2{,}0\ H_{mo}$ is used as suggested in Section 15.2.3, the range of periods suggested above will lead to a maximum wave steepness of

$$\frac{Hmax}{Lo} = \frac{2 \times Hmo}{1{,}56T^2} = \frac{2\ Hmo}{1{,}56(3{,}6)^2 x H_{mo}} = \frac{1}{10{,}1}$$

which is less than the maximum theoretical steepness of 1/7 but consistent with the maximum steepnesses observed in real seas (Carter et al (1986)).

The selection of a critical period within the range of period specified above can best be achieved by systematically changing the wave period in the analysis process until the maximum applied force is experienced. Burrows (1985) derived expressions for the determination of critical wave period for forces in a submerged cylinder from the Morison equation.

15.4 ENERGY DENSITY SPECTRUM ASSOCIATED WITH THE DESIGN WAVE HEIGHT

The selection of an energy density spectrum to be associated with the design wave height is not a trivial problem. Not only were a great variation in spectral shapes recorded during the storms but the choice of a critical spectrum is also very dependent on the response of the structure that is being designed.

To develop a feel for the shape of the design spectrum, Jonswap spectra (Hasselmann (1973)) were fitted to 43 recorded spectra from Slangkop where H_{mo} exceeded 6m to produce the correct peak energy at the peak energy period. The value of the peak enhancement parameter γ varied from 1 to 6 with a mean value of 2,2 and a standard deviation of 1,0. A similar range of γ was reported by Ochi (1979) for fetch limited North Sea waves but there the mean was 3,3 and the standard deviation 0,79.

A useful specification of the Jonswap spectrum for the South African situation is in terms of H_{mo} and T_p. For γ = 2,2 the expression is

$$S(f) = 0{,}242\ H_{mo}^2\ T_p\ (T_p.f)^{-5} \exp\{-1{,}25/(T_p.f)^4\} 2{,}2^q$$

with $q = \exp[-(T_p.f - 1)^2/2\sigma^2]$

and $\sigma = 0{,}07$ for $T_p.f \leq 1$

$\quad\;\; = 0{,}09$ for $T_p.f > 1$

as given in Carter (1982).

The expression of the spectrum for any other value of γ can easily be derived from Carter (1982).

The selection of an appropriate T_p value for a given design H_{mo} was discussed in Section 15.3.1.

If the resonance period of the structure being designed falls in the range $3{,}6\ \sqrt{H_{mo}} < T_p < 5{,}5\ \sqrt{H_{mo}}$ a conservative approach would be to select a spectrum with T_p at the resonance period and a high γ value.

If a structure shows tendencies to resonate, it will be advisable to take a much closer look at the spectra which show high energy values near the resonance frequencies. It is further of extreme importance to ensure that the groupiness structure of the waves are maintained when reproducing the spectrum in either a physical or mathematical model. Care should also be taken to ensure that the random nature of the waves is maintained and especially that some of the common errors made in reproducing the waves are avoided (Tucker, 1984).

16. SUMMARY AND CONCLUSIONS

(i) A wave analysis and data qualification program was developed for digital Waverider data. This program proved invaluable in eliminating all erroneous data from the data banks.

(ii) Good quality Waverider data covering the coast between Oranjemund and Port Elizabeth were available. The wave records were in sufficient depths to be relatively free from shallow water effects such as refraction.

(iii) A surprising similarity in simultaneously recorded wave heights at all deepsea stations between Cape Town and Port Elizabeth enabled the compilation of a near continuous data set over an eight year period. Study of synoptic weather maps indicated that no major storms occurred during the gaps that remained in this combined data set.

(iv) The data set described in (iii) above was used to develop methodologies for design wave height determination.

(v) It was found that if all six hourly recorded data were used a good visual fit of the data to Exponential, Extreme I, Log-normal and three parameter Weibul distributions could be found. The design wave heights produced in this way were found to be virtually identical for all four of the abovementioned distributions. Of these four distributions the Extreme I distribution was favoured.

(vi) Serial correlation studies and careful examination of the time series of recorded wave heights revealed that the maximum H_{mo} recorded in each week of the stormy months May to September should provide data that are independent and identically distributed. The mean of the maximum H_{mo} recorded per week varies from approximately 4,9m in the months May to September to 4,0m in the summer months November to April.

(vii) Using the method of moments to estimate the parameters of the Extreme I distribution, it was found that the derived design wave heights were insensitive to assumptions of independent and identically distributed data. Similar design wave heights were obtained when varying the data from the 10537 records at six hourly intervals to the maximum H_{mo} values recorded per week, per month and per year.

(viii) Using the maximum recorded H_{mo} values per week in the stormy months as the best estimate of independent and identically distributed data, it was found that the 95 per cent confidence limits of up to the 100 year return H_{mo} reduced to within 10 percent of the most probable value, within six years of recording.

(ix) If the method of moments is used to estimate the parameters of the Extreme I distribution using all available six hourly records, it is shown that the loss of a few major storms will not lead to a serious under-estimation of the design wave heights. Similarly the derived design wave heights are not very sensitive to inaccuracies in the estimates of H_{mo}, provided these estimates are not biased.

(x) Using the method of moments applied to all six hourly records to estimate the parameters of the Extreme I distribution for all the available deepsea and open sea Waverider records revealed a very clear pattern in the long term distribution of wave heights around the coast. A strong correlation was found between the parameter β of the Extreme I distribution and the latitude (°S) of the recording site,

i.e. $\beta = 0{,}0366 \times (°S) - 0{,}4509$

with a correlation coefficient of 0,975.

Similarly a correlation between the parameters α and β of the Extreme I distribution was found as

$$\alpha = 0{,}5492 + 1{,}9568\ \beta$$

with a correlation coefficient of 0,967.

These two correlations enabled the estimation of the deepsea design wave heights along the entire coastline between Oranjemund and Port Elizabeth.

The reason for the strong dependence of the design wave heights on the latitude is undoubtedly the relatively constant path of the cold fronts from west to east past the southern tip of the African continent. Wave heights along this entire coastline are generated by these cold fronts and wave heights reduce with increasing distance from the paths of these cold fronts.

(xi) The fact that such a clear relationship between design wave heights and the latitude of the stations could be established when only using a few years of available records at each station lends further credibility to the relatively narrow confidence limits established in (viii) above.

(xii) The wave period distribution along the entire coastline from Oranjemund to Port Elizabeth is virtually identical with a median T_p of 12,5s and an 80 per cent range of 9s to 16s. East of Port Elizabeth wave periods start reducing slightly and this trend continues all the way up the coast to Richards Bay.

(xiii) Spectral analysis reveal that the highest wave energy levels occur at wave periods between 13s and 16s along the entire South African coastline.

(xiv) In general a good understanding of the long term distribution of wave conditions have been gained through this study especially for the coast between Oranjemund and Port Elizabeth. Limited data are available for the rest of the coastline but the data that are available in general conforms to the expected patterns. The occasional occurrence of cyclones along the northern Natal coast invalidates the use of the normal techniques for design wave estimation for this area.

17. RECOMMENDATIONS

17.1 WAVE RECORDING

(i) There is a tendency for the Waveriders to malfunction during storms, most probably due to a deterioration of the radio link between the buoy and the Waverider. Means to elminate his tendency should be sought.

(ii) The duration of the recording of 1024s used at present is too short. It leads to inaccurate estimate of H_{mo} and poor resolution of the periodicity of the lower frequency component of the waves. An increase in recording length to 2048s is recommended.

(iii) A recording interval of six hours is too large especially during high wave events. It is recommended that continuous records should be collected during major events. This will enable a proper study of the stationarity and other short term characteristics of the waves such as groupiness, short term height distributions, etc. If continuous recording commences when the 1 per cent wave height at a site is exceeded, it will only add approximately 130 forty minute records per year to the data. For the Southern Cape coast it will require continuous recording once a H_{mo} of 6m is exceeded.

(iv) The addition of a deepsea wave recorder off the Transkei and Natal coasts will ensure that the insight into the wave patterns along this part of coast will be drastically improved. It is recommended that a Waverider should be installed off, say, Port St Johns and Durban, for a minimum of, say, two years while the Cape Recife and Slangkop Waveriders are still operational.

(v) Directional wave recording is required for many applications. The establishment of one deepsea directional system for developing methodology and to gain experience in this type of recording and analysis is recommended.

17.2 WAVE ANALYSIS

(i) In general the analysis and data qualification routines of "Waves" are found to be satisfactory. A tendency for the program to accept what is seemingly random noise as a valid low wave height (H_{mo} < 1m) was noticed. A routine to look critically at very low waves may help to eliminate this tendency. It is, however, recommended that the quality routines of "Waves" should not be relaxed to accept more data but that the recording system be improved at stations where high rejection rates occur.

(ii) No analysis of groupiness or the short term distribution of individual wave heights and periods are included in the standard "Waves" program. Addition of routines to look at these aspects will lead to a better understanding of the short term behaviour of the waves.

17.3 DESIGN WAVE ESTIMATION

(i) It is recommended that the method of moments should be used to estimate the parameters of the Extreme I distribution, using all six hourly records in the process. This should replace the present graphical fit of an Exponential distribution to the highest 50 per cent of the data. The recommended procedure is much more robust and is not nearly as sensitive to the loss of the records during storms, the occurrence of particularly extreme events that may have been recorded, or inaccuracies in the height estimates. It further fits the data sets over the full range of wave heights.

(ii) The use of recurrence interval or return period in the design specifications should be avoided. This should be replaced by a specification of a risk of exceedance during the design life of the structure.

17.4 AREAS FOR RESEARCH

(i) A concentrated research effort into the short term characteristics of extreme events is required. Data available at present is not entirely suitable for this purpose but if the recommendations about recording made in Section 17.1 (ii) and (iii) above are accepted, suitable data will soon be forthcoming.

(ii) The influence of the cyclones along the Natal north coast on the design wave heights should be investigated. Several approaches to the problem of design wave estimation in cyclone prone areas have been published and a proper literature search will provide guidance as to possible procedures to be followed.

(iii) A superficial look at VOS data recorded during major wave events along the Cape west and south coast left the author with the impression that the heights and periods recorded by these vessels are totally useless for design wave height determination. This impression is strengthened by the work of Liang (1985) which showed the unreliability of these records. The consistency in recorded wave heights between Slangkop and Cape Recife affords an ideal opportunity to do a proper check on the VOS data in this area.

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APPENDIX A

The "WAVES" Program Printout

The program is designed to print all the information of one record on a single printer page. From the top down, the page is divided into five areas. The areas are described, in order, in points 1 to 5:

1. The header area

   The first three lines of the printout contain the information necessary to identify the record (for example, degrees latitude and longitude) and information about how the sampling took place. The header area also gives the category assigned to the record.

2. Time domain parameter area

   The values of all the time domain parameters calculated are given in this area. As on the whole printout, the values are given separately for the two halves that the record is divided into.

3. Frequency domain parameter area

   The values of the frequency domain parametes calculated are given in this area.

4. Data qualification area

   The severity codes for the tests performed on the record are given in this area. A severity code of 0 means that all the requirements of the test were met. A code of 3 means that the record badly fails the test.

5. The spectral information area

   In this area, the last on the page, the power spectral density is given per frequency for each half of the record. The mean values for the two halves are also given. To facilitate interpreting the values, a small printer plot is provided.

APPENDIX A (continued)

```
WAVE ANALYSIS OF : SLANGKOP          (1ST:GOOD 2ND:GOOD) CATEGORY: GOOD
DATE: 1970 10  5. TIME: 1200.
                                                                                    PROGRAMME REVISION: 1
RECORDER:WAVE RIDER  :DEPTH:   0M; 34DEG  3.1MIN S 18DEG  3.5MIN E;WATER DEPTH: 197M;SAMPLING INTERVAL:0.50S;SAMPLES/HALF:1024
..................................................................................................................................
TIME DOMAIN   HALF   P2  P3  P4  >P4 ERR CON   G(MAX)    CORR     MEAN   VARIANCE    HS       H1       TZ    SKEWNESS KURTOSIS
***********   ----   --- --- --- --- --- ---  --------  -------  -------  --------  -------  -------  ------  -------- --------
PARAMETERS:    1.    10   0   0   0   0   0    2.067    0.002    10.199    0.524    2.896    4.260    6.649   -0.062    2.722
***********
               2.     0   0   0   0   0   0    2.094   -0.010    10.203    0.554    2.976    4.280    5.333    0.053    2.945
..................................................................................................................................
FREQ DOMAIN   HALF     TP       HMO      TH02     TH24     TH01      E        QP     LF.DET   HF.DET
***********   ----   -------  -------  -------  -------  -------  -------  -------  -------  -------
PARAMETERS:    1.    11.907    2.809    5.600    2.455    6.864    0.899    2.160    0.023    0.004
***********
               2.    10.667    2.910    5.369    2.616    6.332    0.873    1.600    0.032    0.003
..................................................................................................................................
DATA QUALI-   HALF  FLATHEAD ERRATICS  CONSEC   MEAN#0   TREND   NON-NORM  LF.DET      CORR1#CORR2 MEAN1#MEAN2  VAR1#VAR2
***********   ----  -------- --------  -------  -------  -------  -------  -------     ----------- -----------  ---------
FICATION:      1.      0        0         0        1        0        0        0
*********
               2.      0        0         0        1        0        0        0              0           0            0
..................................................................................................................................
SPECTRUM    NO    FREQUENCY:  PERIOD        SPECTRAL DENSITY(M*M/HZ)                        MEAN SPECTRAL DENSITY (% OF  8.964)
---------            (HZ)       (S)       1ST HALF    2ND HALF     MEAN              0        20        40        60        80       100
                                                                                     |---------|---------|---------|---------|---------|
             1      0.006    170.667      0.003       0.003       0.003              |
             2      0.016     64.000      0.000       0.005       0.007              |
             3      0.025     39.385      0.005       0.007       0.006              |
             4      0.035     28.444      0.007       0.016       0.012              |
             5      0.045     22.261      0.005       0.000       0.007              |
             6      0.055     18.286      0.037       0.009       0.023              |
             7      0.064     15.515      0.283       0.254       0.269              |*
             8      0.074     13.474      2.147       1.097       1.622              |*********
             9      0.084     11.907     13.536       4.391       8.964             F|**************************************************
            10      0.094     10.667      7.081       8.537       7.809             R|*******************************************
            11      0.104      9.660      4.110       5.509       4.849             E|***************************
            12      0.113      8.828      2.460       4.435       3.448             Q|*******************
            13      0.123      8.127      1.992       5.387       3.689             U|********************
            14      0.133      7.529      3.032       4.220       3.626             E|********************
            15      0.143      7.014      1.549       1.703       1.666             N|*********
            16      0.152      6.564      1.353       2.411       1.082             C|**********
            17      0.162      6.169      0.826       0.538       0.682             Y|***
            18      0.172      5.818      1.346       0.929       1.137              |******
            19      0.182      5.585      0.559       0.903       0.731              |****
            20      0.191      5.224      0.315       1.591       0.953              |*****
            21      0.201      4.971      0.547       1.468       1.008              |*****
            22      0.211      4.741      0.986       0.874       0.930              |*****
            23      0.221      4.531      0.855       0.658       0.757              |****
            24      0.230      4.339      0.870       0.963       0.917              |*****
            25      0.240      4.163      0.378       0.857       0.617              |***
            26      0.250      4.000      0.518       0.711       0.615              |***
            27      0.260      3.850      1.071       0.838       0.955              |*****
            28      0.270      3.710      0.335       0.284       0.309              |*
            29      0.279      3.580      0.288       1.007       0.643              |***
            30      0.289      3.459      0.367       0.370       0.368              |**
            RESOLUTION (DELTA F IN HZ): RAW =  0.00195     SMOOTHED OVER  5 LINES =  0.00977
..................................................................................................................................
FINAL SUMMARY (    MEAN ) :  HS=  2.936 M    HMO=  2.859 M    TZ=  5.991 S   TH02=  5.489 S    TP= 11.907 S
**************************
```

APPENDIX B

Assigning severity counts to qualification parameters:

Parameter	Severity count	Test	Test values
FLATHEAD	0	"P2" <T1 and "P3" + "P4" + ">P4" = 0 or "P2"<T2 and "P4" <1	T1=86-7,5"Hs" T2=46,2-3,45"Hs" + 5,24"P3"- 0,803"P3""P4"
	2	All other cases	
	3	">P4" > T3	T3=16,56-3,18"Hs"
ERRATICS	0	"ERR" < 5	
	1	5 < "ERR" < 10	
	3	"ERR" > 10	
CONSEC	0	"CON" = 0	
	3	"CON" > 0	
TREND	0	0,0 <|"CORR"|<0,062	P = 0,01 and P=0,5 correlation test with 1 024 d.f.
	1	0,062 <|"CORR"| <0,081	
	3	|"CORR"|>0,081	
NON-NORM	0	0,0 <|"SKEWNESS"| <0,26 2,3 <|"KURTOSIS"| <3,85	
	1	0,26<|"SKEWNESS"| <0,3 2,0 <|"KURTOSIS"|<2,3 or 3,85<|"KURTOSIS"| <5,0	
	2	0,3<|"SKEWNESS"| <0,4	
	3	|"SKEWNESS"|>0,4 0,0 <|"KURTOSIS"|<2,0 or 5,0<|"KURTOSIS"|<∞	
LF.DET	0	"LF.DET"<T14	T14=0,2+0,17"Hs"
	2	T14 <"LF.DET" <T15	
	3	"LF.DET">T15	T15=0,4+0,2"Hs"

APPENDIX C

AGULHAS BANK DATA

TABLE C1 : MAXIMUM H_{mo} RECORDED PER WEEK

MONTH \ YEAR	1978	1979	1980	1981	1982	1983	1984	1985	1986
JANUARY		3.5 4.9 5.2 3.3	5.0 3.8 2.5 4.0	4.1 3.5 3.6 4.9	5.5 4.0 4.4 4.6	4.0 4.0 3.8 3.4	3.1 3.6 2.7 4.9	5.1 3.0 2.9 3.7	3.0 4.6 3.0 3.8
FEBRUARY		2.5 4.2 3.6 3.1	3.1 4.1	4.8 4.2 3.6 4.0	3.0 4.0 3.9 3.9	7.8 4.3 2.8 4.0	5.4 3.3 3.1 3.9	2.0 3.1 3.0	3.4 4.0 2.9 3.6
MARCH		4.9 4.5 4.2 3.2	2.3 4.1 4.0 2.0	3.9 2.1 3.8 4.3	3.6 5.1 4.7 3.6	3.5 4.5 5.0 3.6	6.4 3.8 3.0 4.1	5.5 3.5	4.4 2.3 5.0 6.9
APRIL		2.9 5.2 3.4 2.9	3.9 5.9 3.6 4.1	4.2 5.6 3.5 5.2	3.9 4.1 5.1 5.1	3.8 3.0 4.3 3.1	4.9 5.8 3.6 5.6	4.1 2.9 4 1	3.1 4.1 4.6 3.5
MAY		6.1 4.1 4.5 5.0	4.0 4.3 5.3 5.8	4.9 4.3 3.0 4.6	2.3 4.3 6.0 3.5	3.7 6.6 6.3 3.6	6.1 10.8 4.0	4.7 2.8 2.5 4.6	3.3 7.0 5.1 3.4
JUNE	5.5 5.2	3.1 5.8 3.8 8.1	3.9 3.6 4.9 6.3	6.2 4.1 4.6 5.3	3.0 5.9 5.0	4.1 5.2 2.8 6.5	4.2 4.2 4.2	4.3 5.2 7.1 4.1	
JULY	4.6 4.2	4.8 3.0 4.2 6.0	3.0 4.8 4.0 5.0	5.0 5.1 6.5 5.3	4.1 5.3 4.7 5.3	5.1 5.1 5.7 4.8	7.2 6.3 4.8 5.2	4.3 4.4 4.0	
AUGUST	5.1 5.2 4.5 5.7	6.3 4.0 3.8 4.0	3.2 5.7 3.9 6.9	5.7 5.3 3.8 5.6	6.1 4.2 4.7	6.8 3.1 4.8 4.8	4.6 5.1 5.7	5.9 4.6 3.4 5.1	
SEPTEMBER	8.7 4.7 5.6	3.5 4.3 7.4 7.0	6.1 5.1 6.0 4.2	3.8 3.8 5.6 4.1	4.6 4.5 5.0 3.2	4.7 5.4 5.2 6.3	6.9 4.9 3.9 5.1	4.1 5.2 4.3 4.5	
OCTOBER	4.3 5.0 4.8 4.8	7.9 6.0	6.2 3.4 3.7 3.1	6.2 3.7 3.8 4.7	6.8 3.6 4.3	5.7 3.3 4.5 4.4	6.1 3.6 4.0	4.7 3.8 3.2 4.0	
NOVEMBER	4.0 5.8 4.0 4.2	4.6 3.8	4.0 5.1 4.0 4.4	3.8 3.9 4.7 4.9	2.9 5.1 4.7 2.6	2.9 4.1 3.0 3.7	4.0 3.2 5.2	3.1 4.6 3.4 2.4	
DECEMBER	3.9 3.0 8.7 4.0	4.2 3.5 3.0 3.0	3.4 3.6 5.5 4.6	3.4 3.3 3.6 3.0	4.2 3.9 4.8 3.6	3.6 2.7 4.0 3.0	5.5 3.1 5.3	4.1 4.4 3.1 5.6	
NUMBER OF WEEKS		44	46	48	45	48	42	43	
MEAN		4.4568	4.3543	4.3938	4.3711	4.3833	4.7714	4.0605	
STANDARD DEVIATION		1.3967	1.1153	0.9297	0.9507	1.2061	1.4816	1.0365	

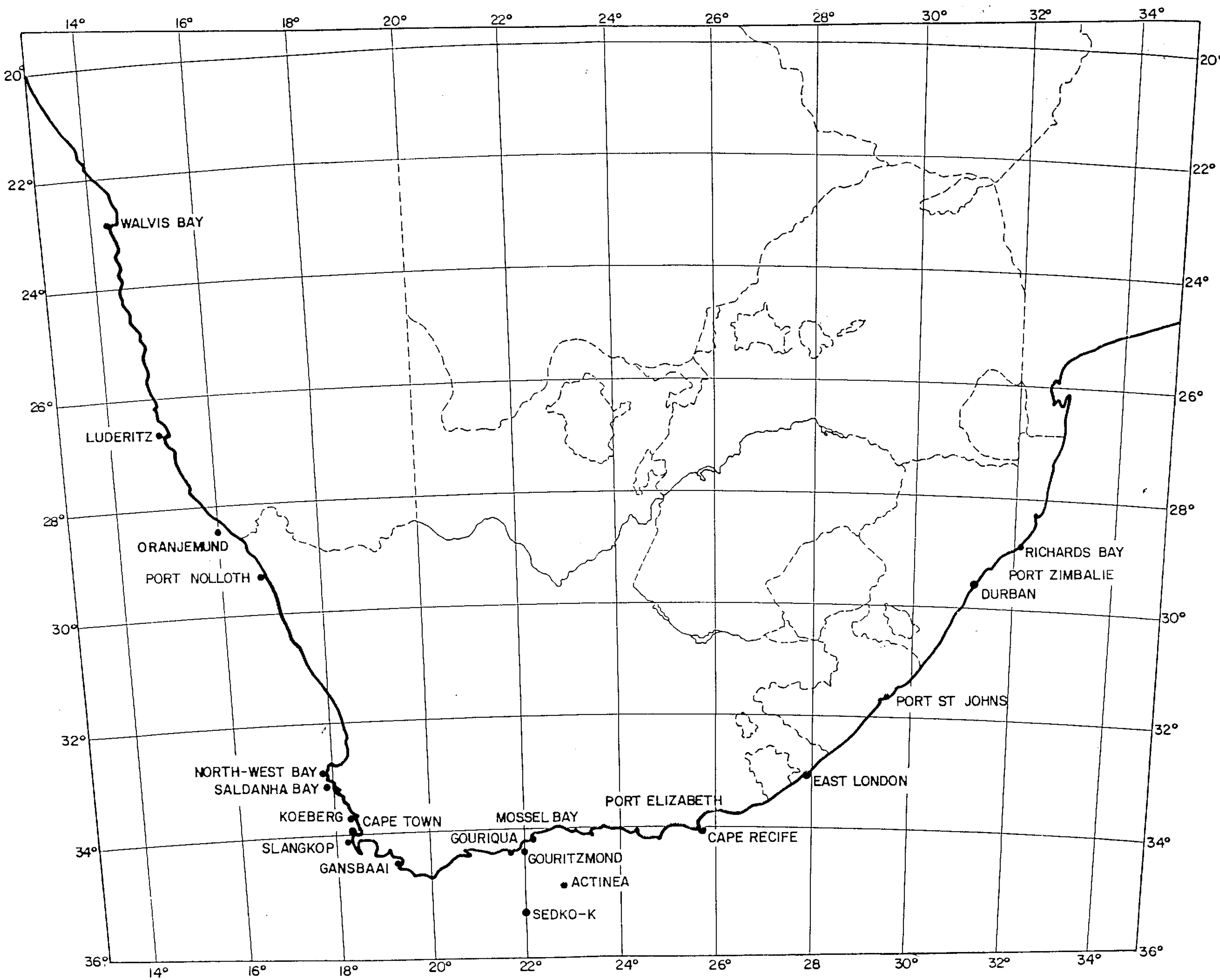
TABLE C2 : MAXIMUM H_{mo} RECORDED PER MONTH AND PER YEAR

MONTH \ YEAR	78/79	79/80	80/81	81/82	82/83	83/84	84/85	85/86	Maximum in month
June	5,47	8,02	6,39	6,29	5,87	6,51	4,32	7,10	8,02
July	4,25	6,03	4,96	6,47	5,36	5,62	7,22	4,42	7,22
August	5,66	6,37	6,91	5,59	6,14	6,63	5,67	5,91	6,91
September	8,67	7,42	6,15	5,56	5,04	6,30	7,01	5,31	8,67
October	4,97	7,79	6,21	6,27	6,72	5,56	6,06	4,65	7,79
November	5,76	4,76	5,14	4,80	5,19	4,20	5,26	4,61	5,76
December	8,60	4,27	5,53	3,62	4,71	4,06	5,44	5,56	8,60
January	5,31	4,95	4,83	5,42	3,97	4,87	5,19	4,51	5,42
February	4,38	4,14	4,72	3,91	7,58	5,39	3,13	4,01	7,58
March	4,48	4,85	4,30	5,09	4,95	6,35	5,44	6,93	6,93
April	5,24	5,84	5,58	5,28	4,34	5,73	4,24	4,56	5,84
May	6,15	5,72	4,83	6,04	6,55	10,80	4,57	7,03	10,80
Maximum in year	8,67	8,02	6,91	6,47	7,58	10,80	7,22	7,10	

TABLE C3 : PERCENTAGE OCCURRENCE AND DISTRIBUTION OF WAVE HEIGHT TO WAVE PERIOD

WAVERIDER AT : AGULHAS BANK
REMARK : ANNUAL
PERIOD : 1978 6 1 TO 1986 6 1
NUMBER OF RECORDS: 10537

WAVE HEIGHT	< TP(PEAK ENERGY PERIOD IN SECONDS.) >																		
HMO(M)	5.22	5.22	5.51	5.82	6.17	6.56	7.01	7.53	8.13	8.83	9.66	10.67	11.91	13.47	15.52	18.29	22.26	22.26	SUM
> .0- .5	0.02			0.01							0.01	0.03	0.10	0.05	0.16		0.02		.40
> .5- 1.0	0.04				0.04			0.01	0.04	0.05	0.10	0.13	0.19	0.25	0.10	0.07	0.01	0.03	1.05
> 1.0- 1.5	0.22	0.02	0.02	0.04	0.04	0.03	0.06	0.10	0.11	0.55	1.28	1.99	1.36	0.60	0.12	0.07		0.01	6.61
> 1.5- 2.0	0.14	0.03	0.06	0.09	0.16	0.15	0.27	0.31	0.56	1.01	2.41	5.18	5.37	2.70	0.79	0.17	0.02		19.42
> 2.0- 2.5	0.03	0.01	0.05	0.14	0.17	0.29	0.47	0.52	0.57	0.98	2.18	5.20	7.56	4.56	1.22	0.21	0.02		24.19
> 2.5- 3.0		0.01	0.01	0.05	0.09	0.16	0.36	0.47	0.66	0.72	1.28	3.04	5.91	4.61	1.45	0.21	0.01		19.05
> 3.0- 3.5					0.02	0.03	0.24	0.32	0.31	0.57	0.75	1.55	3.69	3.46	1.21	0.14	0.01		12.31
> 3.5- 4.0								0.09	0.18	0.37	0.44	0.80	1.86	2.53	1.00	0.13	0.01		7.40
> 4.0- 4.5								0.03	0.09	0.16	0.28	0.31	1.21	1.42	0.72	0.08	0.02		4.34
> 4.5- 5.0									0.02	0.06	0.14	0.21	0.58	0.92	0.43	0.05			2.40
> 5.0- 5.5									0.01	0.02	0.07	0.09	0.38	0.53	0.27	0.06			1.41
> 5.5- 6.0											0.02	0.05	0.09	0.28	0.23	0.02			.68
> 6.0- 6.5												0.03	0.06	0.21	0.10	0.01			.41
> 6.5- 7.0													0.02	0.05	0.04	0.02			.12
> 7.0- 7.5												0.01		0.06	0.03	0.02			.11
> 7.5- 8.0															0.02				.02
> 8.0- 8.5														0.01	0.02				.03
> 8.5- 9.0														0.01		0.01			.02
> 9.0- 9.5												0.01							.01
> 9.5-10.0																			.00
>10.0-10.5																			.00
>10.5															0.01				.01
SUM:	.45	.07	.13	.32	.51	.66	1.39	1.86	2.56	4.48	8.97	18.62	28.39	22.26	7.92	1.25	.11	.04	100


WALVIS BAY
LUDERITZ
ORANJEMUND
PORT NOLLOTH
NORTH-WEST BAY
SALDANHA BAY
KOEBERG
CAPE TOWN
SLANGKOP
GANSBAAI
GOURIQUA
MOSSEL BAY
GOURITZMOND
ACTINEA
SEDKO-K
PORT ELIZABETH
CAPE RECIFE
EAST LONDON
PORT ST JOHNS
DURBAN
PORT ZIMBALIE
RICHARDS BAY