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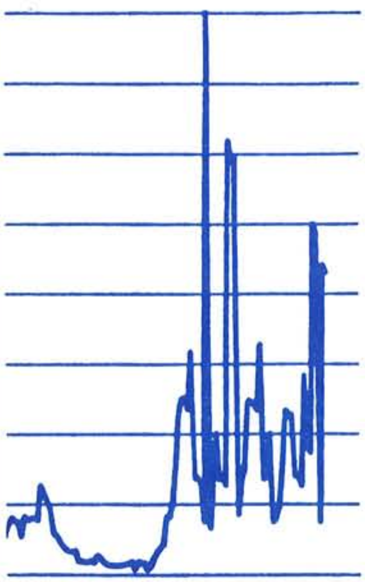

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DURATION OF EXTREME WAVE CONDITIONS

by

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Statistical trends of the duration of extreme wave conditions, as characterized by hindcast wave information, are investigated at five sites along the coastline of the United States (three on the Atlantic coast and two on the Pacific coast). A review of pertinent statistical concepts and water wave characterization conventions and terminology is followed by a description of the Wave Information Studies Program of the US Army Engineer Waterways Experiment Station, Vicksburg, Miss. The database of hindcast wave information in shallow water created by this program is applied to develop a method of identification of extreme events and definition of their duration, based on exceedance of a threshold for zero moment wave heights. The number of events identified is found to be proportional to the percent exceedance of the specified threshold, regardless of geographical location. The Extremal Type I distribution is found to be superior to the Weibull distribution as a model for both distribution of durations and peak zero moment wave

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heights of extreme events identified. A regression analysis of duration with various parameters representing peak wave conditions reveals only a weak linear relation with peak zero moment wave height and little evidence of a linear relation with any other parameter investigated. The assumption of independence of duration from peak wave conditions is proposed as an expedient method for estimating durations above a specified threshold, given a peak wave condition.

PREFACE

Work leading to preparation of this manuscript was conducted as part of the "Develop Functional and Structural Design Criteria" work unit of the Coastal Structures Evaluation and Design research and development program of the US Army Corps of Engineers. Authorization from the Office, Chief of Engineers, to publish this report is gratefully acknowledged. This report was originally submitted by the author to Mississippi State University as partial fulfillment of the requirements for an M.S. degree in Civil Engineering.

This study was conducted and the report written by Mr. Orson P. Smith, Coastal Design Branch (CDB), Wave Dynamics Division (WDD), of the Coastal Engineering Research Center (CERC), US Army Engineer Waterways Experiment Station (WES). The author acknowledges the assistance of Dr. Robert E. Jensen of the Coastal Oceanography Branch, Research Division, CERC, whose technical guidance with regard to application of the Wave Information Studies database of hindcast wave information was appreciated throughout the course of the work. The author also is grateful for the technical guidance of Dr. Michael E. Andrew of the Prototype Measurement and Analysis Branch, Engineering Development Division, CERC, regarding statistical aspects of the work.

Work was performed under the general direction of Dr. Frederick E. Camfield, Chief, CDB; Mr. C. E. Chatham, Chief, WDD; Mr. Charles C. Calhoun, Jr. Assistant Chief, CERC; and Dr. James R. Houston, Chief, CERC.

The Director of WES during the course of the work was COL Allen F. Grum, USA. Commander and Director of WES during publication of this report was COL Dwayne G. Lee, CE. Technical Director was Dr. Robert W. Whalin.

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CHAPTER I: INTRODUCTION

Statement of the Problem

Extreme wave conditions have been the cause of property loss, suffering, injuries, and death since man first approached the sea. Coastal engineers therefore have always attempted to build works that would withstand, with little or no damage, the worst impact of waves from very rare events. The direct effect of sea waves striking coastal structures has long been recognized as a critical phenomenon with respect to structural integrity during a storm at sea. The hydraulic impact of individual waves has traditionally been the specific force used as the basis of structural design criteria; therefore, characteristics of the worst few waves of a hypothetical extreme event have been estimated for application in most design computations. Rubble-mound structures, constructed of layered quarrystone or concrete shapes and built for centuries as wave barriers (breakwaters and jetties) or shore protection (revetments), are usually designed in this fashion.

The limits of functional performance of coastal structures have recently become more critical with respect to overall economic optimization. Public financing of coastal works has been more difficult to arrange than in past decades. The concept of designing a structure to be stable during a very extreme storm, but to be less than 100 percent effective in some extreme events of lesser intensity, has been in the minds of coastal engineers in an effort to conceive affordable harbor or shore protection plans. Life cycle cost also is receiving much more scrutiny, particularly with respect to expensive mobilization and challenging construction techniques required for repairs at many coastal projects. The bulwarks of extreme conservatism in coastal engineering design practice are beginning to buckle under pressure for more precise estimates of structural integrity and functional performance. These estimates may someday approach the precision of those now required for design of buildings and bridges.

One critical question in many new optimized designs is "What is the effect of duration of exposure?" Sandy beaches commonly change their shapes to a more stable configuration, given sufficient exposure to severe wave conditions, in theory approaching a new equilibrium (Bruun 1954). Some radical new rubble-mound concepts attempt to emulate this effect (Delft Hydraulics

Laboratory 1985). Laboratory experiments which simulate natural irregular waves also have shown some duration effects on rubble mounds of more traditional design (Graveson et al. 1980; Van der Meer and Pilarczyck 1984; and Tenaud et al. 1981). The open literature contains little specific guidance, however, for researchers or designers to estimate the duration of a given intensity of extreme wave conditions.

Purpose and Objectives

The purpose of this work is to investigate the duration of extreme wave conditions estimated from hindcast wave data, with a view toward developing a means to characterize the variation of these durations for use in design of coastal structures. Hindcast wave data, which are discussed later in more detail, are one of the most valuable tools of coastal engineers, primarily because weather data on which they are based typically exist for much longer periods of record than other wave information sources. The 20-year (1956-1975) Wave Information Studies (WIS) database of hindcast wave data prepared and maintained by the US Army Engineer Waterways Experiment Station (WES) (Brooks and Corson 1984) is a key source of wave information in many US Army Corps of Engineers projects since it now extends along most of the coastline of the United States.

The specific objectives of this study were to (1) review existing literature regarding the duration of extreme wave conditions and related topics; (2) formulate a practical means of identifying individual events of extreme wave conditions, relying on the intensity of wave conditions as represented in the WIS database and associated publications; (3) address the probability distribution of extreme event durations by fitting selected distribution functions to representative data; and (4) address the possible relation of an extreme event's duration to the peak conditions during the extreme event by regression analysis.

Organization

This report presents reviews of pertinent statistical concepts and techniques, considerations regarding the characterization of wave conditions, and the specific nature of WIS hindcast data before proceeding to describe the progress toward and conclusion of the four objectives stated above. An overall summary and statement of conclusions then is followed by Appendix A containing figures and tables which were not presented in the main text for the sake of continuity and space conservation. Appendix B includes pertinent wave information transcribed from the WIS database. Appendix C includes a listing of the computer program STRMDIST which was used to identify extreme events, define durations, and fit parameterized distribution functions to both the durations and peak wave heights of extreme events identified. Appendix D includes the command file for the commercial statistical software package SPSS (Nie et al. 1975), which was applied to address the relationship of extreme event duration to peak wave conditions.

CHAPTER II: REVIEW OF PERTINENT STATISTICAL CONCEPTS

Continuous Frequency Distributions

The primary tools of this study are statistical procedures which address the variability of parameters of interest, specifically duration of extreme events at sea and their peak intensity. A brief review of pertinent statistical concepts, which are critical to understanding the methods and conclusions of the analysis, is presented below.

Continuous random variables are variables whose values are measured on a continuous scale, as opposed to their discrete counterparts such as rolling dice or coin flipping. Most natural phenomena of varying intensity as measured by instruments are treated as continuous random variables. The probability that the value of a particular random variable, x, will fall within a certain range can be estimated by application of its probability density function, $f(x)$, which is analogous to a histogram for discrete variables. The following two conditions apply in defining probability density functions:

$$f(x) \geq 0 \quad \text{for all} \quad x \quad \text{within the domain of} \quad f$$

and

$$\int_{-\infty}^{\infty} f(x)\, dx = 1 \tag{1}$$

The probability that x will fall within the range from a to b is given by:

$$P(a \leq x \leq b) = \int_{a}^{b} f(x)\, dx \tag{2}$$

Technically the probability of x taking on a value of exactly a or b is zero, but since physical measurements cannot be infinitely accurate, the interval from a to b can be considered inclusive. A transformation of the probability density function into its corresponding distribution function, $F(x)$, allows more expedient computation of probabilities:

$$F(x) = \int_{-\infty}^{x} f(t)\ dt \tag{3}$$

where $f(t)$ is the probability density function of a dummy variable t .

The value of $F(x)$ varies between 0 and 1. The probability that x will have a value equal to or less than a is $F(a)$. The probability that x will have a value between a and b is $F(b) - F(a)$. The corresponding probability density function is:

$$f(x) = \frac{dF(x)}{dx} \tag{4}$$

It is important to define the domain of f and that this domain include all the values of x of interest. Furthermore, the function f must be integrable within this domain (and F differentiable) for the above definitions to apply (Miller and Freund 1985).

Distribution Parameters

The mean or expected value of x is defined by:

$$\mu = \int_{-\infty}^{\infty} xf(x)\ dx \tag{5}$$

The variance of probability density function is the expected value of the squared deviation from the mean, given by:

$$\sigma^2 = \int_{-\infty}^{\infty} (x - \mu)^2\ f(x)\ dx \tag{6}$$

The variance, σ^2 , and its square root, the standard deviation, σ , are both measures of the spread of the probability density about the mean. The standard deviation is expressed in the same units as x and μ . A small variance or standard deviation implies a strong central tendency while large values imply significant spread or "variance" of x values (Miller and Freund 1985).

The Poisson Distribution

A wide variety of distribution functions have been formulated by researchers and statisticians which have been shown to describe well the behavior of certain random variables which occur in nature. One such function is the Poisson distribution, defined by:

$$f(x) = \frac{\lambda^x e^{-\lambda}}{x!} \qquad \text{for} \quad x = 0, 1, 2, \ldots \tag{7}$$

This is a discrete distribution which has important associations with the continuous distributions that have been applied to describe weather-related variables. Specifically, the Poisson distribution has been applied to describe the number of occurrences of events taking place randomly over continuous intervals of time. The parameter λ is both the mean and the variance of the Poisson distribution. A key assumption behind application of this distribution is that the probability of an occurrence for the type of event in question during a small interval of time must not depend on what happened prior to that time. A random process which fits this criterion is called a Poisson process.

The Exponential Distribution

A continuous distribution which is often associated with the Poisson distribution is the exponential distribution, given by:

$$\begin{aligned} f(x) &= \frac{e^{-x/\beta}}{\beta} \qquad \text{for} \quad x > 0 \quad \text{and} \quad \beta > 0 \\ &= 0 \quad \text{elsewhere} \end{aligned} \tag{8}$$

The corresponding distribution function is:

$$F(x) = 1 - e^{-x/\beta} \tag{9}$$

The mean and standard deviation of a variable represented by an exponential distribution are both β and the variance is β^2. This distribution is often used with Poisson processes to model the waiting time between successive occurrences. If the λ parameter of a Poisson distribution is the average number of occurrences in time T, then the average rate of occurrences per unit time is λ/T. The corresponding exponential distribution parameter is $\beta = T/\lambda$. This relation and the fact that both distributions are fully described by a single parameter make them easy to use in a wide range of applications dealing with the frequency of and waiting time between discrete events.

The Weibull Distribution

Another distribution, which is widely used to model the variation in intensities of natural extremes such as flood elevations and storm intensities, is the Weibull distribution, where:

$$f(x) = \frac{1}{\beta^\alpha}\,\alpha x^{\alpha-1} \exp\left[-\left(\frac{x}{\beta}\right)^\alpha\right] \quad \text{for} \quad x > 0\,, \quad \alpha > 0\,, \quad \beta > 0$$

$$= 0 \quad \text{elsewhere} \tag{10}$$

The corresponding Weibull distrubution function is very similar to the exponential distribution:

$$F(x) = 1 - \exp\left[-\left(\frac{x}{\beta}\right)^\alpha\right] \tag{11}$$

The parameter α is the "shape parameter" which defines the basic shape of the function. The β parameter is the "scale parameter" which determines the degree of spread along the abscissa (Isaacson and MacKensie 1981). The mean and variance of the Weibull distribution are:

$$\mu = \beta\Gamma\left(1 - \frac{1}{\alpha}\right) \tag{12}$$

$$\sigma^2 = \beta^2 \left[\Gamma\left(1 + \frac{2}{\alpha}\right) - \Gamma^2\left(1 + \frac{1}{a}\right)\right] \tag{13}$$

The gamma function is given by:

$$\Gamma(z) = \int_0^\infty x^{z-1}\, e^{-x}\, dx = (z - 1)! \tag{14}$$

The Weibull distribution has two parameters which make it actually a family of functions. A three-parameter form is sometimes used to provide further flexibility in adapting the distribution to certain phenomena, where:

$$F(x) = 1 - \exp\left[- \frac{(x - \varepsilon)}{\beta}\right]^\alpha \qquad \text{for} \quad \varepsilon > 0 \tag{15}$$

The parameter ε is a "location parameter" which locates the position of the probability along the abscissa (x-axis). In the particular case of the Weibull distribution, ε is in effect a lower limit to values of x. The ε parameter is often taken as zero in practice. The Weibull distribution reduces to the exponential distribution when $\alpha = 1$ and $\varepsilon = 0$ (Isaacson and MacKensie 1981).

The Rayleigh Distribution

The Weibull distribution reduces to a Rayleigh distribution when $\alpha = 2$ and $\varepsilon = 0$, a function widely used to model the distribution of wave heights passing a point during a stationary sea state. The term "stationary" refers to the common assumption that, for practical purposes, statistical properties of ocean waves tend to be time invariant during a period of a few minutes to an hour or more. The time for significant changes to occur in a sea state is thus assumed to be substantially longer than the time necessary to measure the form of a few hundred waves passing a fixed point. The Rayleigh distribution, for this purpose, is often expressed in the form:

$$F(H) = 1 - e^{-2(H/H_s)^2} \tag{16}$$

where H is an individual wave height in a sea state and H_s is the "significant wave height," also defined as the average of the highest 1/3 waves. This relation has been found to be quite accurate in most conditions at sea, with the exception of waves nearing the point of breaking in shallow water (Massie 1976). The corresponding probability density function, mean, and variance of this form of the Rayleigh distribution are:

$$f(x) = 4\left(\frac{H}{H_s^2}\right) e^{-2(H/H_s)^2} \tag{17}$$

$$\mu = \left(\frac{\pi}{8}\right)^{1/2} H_s = 0.627\ H_s \tag{18}$$

$$\alpha^2 = \left(\frac{1 - \pi}{8}\right) H_s^2 \qquad (\sigma = 0.779\ H_s) \tag{19}$$

The Extremal Type I Distribution

This distribution, sometimes called the "Gumbel" or "Fisher-Tippet Type I" distribution, also is frequently applied to model natural extremes such as storm intensities (Gumbel 1958). The probability density and distribution functions have the following forms:

$$f(x) = \frac{e^{-e^{-[(x-\varepsilon)/\phi]}} \quad e^{-[(x-\varepsilon)/\phi]}}{\beta} \qquad \text{for} \quad \begin{array}{l} -\infty < x < \infty \\ -\infty < \varepsilon < \infty \\ \beta > 0 \end{array} \tag{20}$$

$$F(x) = e^{-e^{-[(x-\varepsilon)/\beta]}} \tag{21}$$

The mean and variance are:

$$\mu = \varepsilon - \gamma\beta \tag{22}$$

$$\sigma^2 = \frac{\pi^2\beta^2}{6} \tag{23}$$

where γ = Euler's constant = 0.5772 . The Extremal Type I distribution is also a two-parameter family of functions, in this case with a shape parameter of $\alpha = 1$ in keeping with the usual practice for application to weather-related phenomena (Isaacson and MacKensie 1981 and Andrew et al. 1985). The ε parameter is again the location parameter and β the scale parameter. The Extremal Type I distribution is not constrained to positive values of x .

Figure 1 illustrates the relative form of the Exponential, Weibull, Rayleigh, and Extremal Type I distributions. The Exponential and Rayleigh

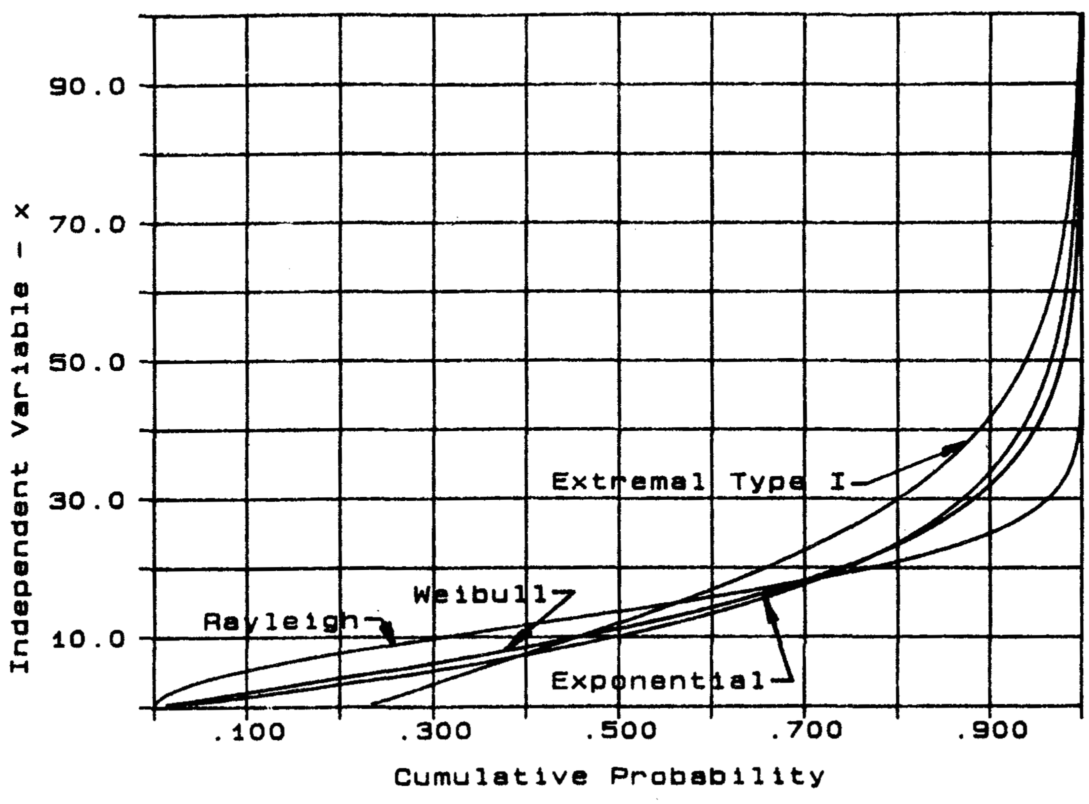

Figure 1. Relative form of four distribution functions

curves shown in Figure 1 have the same mean as the Weibull curve. The Extremal Type I curve of Figure 1 was derived from the same data as the Weibull curve.

Joint Probability

It is often important to describe an event by more than one variable, such as both the duration and peak intensity, in which case the joint probability density must be evaluated. The probability that variables describing the event fall within specified ranges is determined from the joint probability density in a similar manner as with single variable probability density functions:

$$P(a_1 < x_1 < b_1 \, , \, a_1 < x_2 < b_2 \, , \ldots , \, a_n < x_n < b_n)$$

$$= \int_{a_1}^{b_1} \int_{a_2}^{b_2} \cdots \int_{a_n}^{b_n} f(x_1 \, , \, x_2 \, , \ldots , \, x_n) \, dx_1 \, dx_2 \ldots dx_n \qquad (24)$$

$$\text{when} \quad f(x_1 \, , \, x_2 \, , \ldots , \, x_n) \geq 0$$

and

$$\int_{-\infty}^{\infty} \int_{-\infty}^{\infty} \cdots \int_{-\infty}^{\infty} f(x_1 \, , \, x_2 \, , \ldots , \, x_n) \, dx_1 \, dx_2 \ldots dx_n = 1 \qquad (25)$$

A joint distribution function can be defined also:

$$F(x_1 \, , \, x_2 \, , \ldots , \, x_n) = \int_{-\infty}^{x_1} \int_{-\infty}^{x_2} \int_{-\infty}^{x_n} f(t_1 \, , \, t_2 \, , \ldots , \, t_n) \, dt_1 \, dt_2 \ldots dt_n \qquad (26)$$

The marginal probability density of variable x_i is determined by integrating the joint probability density function over the entire domain of all variables except x_i :

$$f(x_i) = \int_{-\infty}^{\infty} \ldots \int_{-\infty}^{\infty} f(x_1 \, , \, x_2 \, , \ldots , \, x_n) \, dx_1 \ldots dx_{i-1} \, dx_{x+1} \ldots dx_n \qquad (27)$$

An important feature of joint probabilities is that if the random variables involved are independent, then their joint distribution function is the product of their marginal distribution functions, such that:

$$F(x_1 , x_2 , \ldots , x_n) = F(x_1) F(x_2) \ldots F(x_n) \tag{28}$$

Another important concept of joint probabilities is conditional probability density, defined in the case of two random variables as the conditional probability density of the first, x_1, given that the second takes on a specified value, x_2, or:

$$g_1(x_1 \mid x_2) = \frac{f(x_1 , x_2)}{f(x_2)} , \text{ if } f(x_2) = 0 \tag{29}$$

Conditional distribution functions, such as $F(x_1 \mid x_2)$, also can be defined, expressing the cumulative probability density in a manner analogous to single variable density functions. Conditional probability densities or distribution functions do not require independence for their definition.

Concepts Related to Evaluation of Risk

A traditional measure of risk of encountering an event of a specified intensity x, such as a critical flood elevation, wind velocity, or wave height, is the return period, $RT(x)$. This is defined in practical terms as the average waiting period between exceedances of x. The return period for variables whose rate of occurrence is independent of their intensity (i.e., the number of occurrences per unit time is a Poisson process with a mean λ) is given by (Borgman and Resio 1982):

$$RT(x) = \frac{1}{\{\lambda[1 - F(x)]\}} \tag{30}$$

The nonencounter probability, $NE(x)$, is defined as the probability that, during a specific time interval L, the largest intensity encountered will be less than or equal to x. This can be expressed in terms of the distribution function $F(x)$ for the case of a Poisson process as (Borgman and Resio 1982):

$$NE(x) = e^{-\lambda L[1-F(x)]} \tag{31}$$

Expressed in terms of the return period:

$$NE(x) = e^{-L/RT(x)} \tag{32}$$

This last relation demonstrates the danger of misinterpreting the return period as a frequency of occurrence for events of intensity x. When $L = RT(x)$, then $NE(x) = 0.37$. In other words, there is a 63 percent probability of encountering an event of intensity x during the time interval L. The term "risk" is defined as the probability that an event of intensity x or greater will occur at least once in the time interval L, which is $1 - NE(x)$.

Another concept important in risk and optimization analyses is that of expectation, $E\{x\}$. This has actually already been defined as the mean of $f(x)$:

$$E\{x\} = \mu = \int_{-\infty}^{\infty} x f(x)\ dx \tag{33}$$

One useful feature of the expectation as a long-term average of the values of x is that the expectation of a function of x, $g(x)$ can be defined by:

$$E[g(x)] = \int_{-\infty}^{\infty} g(x)\ f(x)\ dx \tag{34}$$

Another feature with respect to Poisson processes worth noting regards the reference time period for risk criteria, such as estimation of the average annual value of some variable. Relation of the Poisson parameter λ to expectations of functions of the random variable x (the outcome of a Poisson process, where the number of occurrences per unit time is independent of the value of x) is easiest demonstrated by an example. Assume that in 1 year k

extreme events occur, where k is a Poisson variable. Intensities of extreme events are represented by significant wave heights, H_{si} $(i = 1, 2, 3, \ldots k)$. Damage to a structure caused by each extreme event is assumed to be a function of H_s, $D(H_s)$. Total damage in the year's time is:

$$\frac{D}{yr} = \sum_{i=1}^{k} D(H_{si}) \tag{35}$$

Since k and H_{si} are independent, then the expectation with respect to H_s is:

$$E\left(\frac{D}{yr}\right) = \sum_{i=1}^{k} E[D(H_{si})] \tag{36}$$

Since H_{si} values are independent identically distributed random variables, they all have the same expectation, and:

$$E\left(\frac{D}{yr}\right) = E(k)\ E[D(H_s)] \tag{37}$$

Taking the expectation of k to be the average number of extreme events per year (= the Poisson parameter, λ), the long-term average annual storm damage is:

$$E\left(\frac{D}{yr}\right) = \lambda E[D(H_s)] = \lambda \int_{-\infty}^{\infty} D(H_s) f\ (H_s)\ dH_s \tag{38}$$

This relation is critical in optimization of first costs against estimates of long-term maintenance costs.

Regression by the Method of Least Squares

An important part of many research efforts is the estimation of distribution parameters from measured data by regression using the method of least squares. Assumed linear relationships between an independent variable x and a dependent variable y of the form:

$$y = \alpha + \beta x \tag{39}$$

can be tested against a set of x , y data and the differences, ε , between the estimated y and the predicted value measured. These differences can be due to measurement errors or inadequacies in the assumed relationship, such as neglect of other independent variables which also affect the value of y . The method of least squares allows the parameters α and β to be estimated by constants a and b such that resulting differences in the predicted versus measured y values are a minumum. Since these differences, called residuals, could be both positive and negative and therefore have a tendency to offset each other, the square of the differences is minimized instead. Many nonlinear relationships can be transformed into a linear form to take advantage of this technique.

The accuracy or reliability of least squares estimates of the true linear parameters α and β can be expressed in a number of ways. All possible true y values are assumed to be independently normally distributed with means $\alpha + \beta x$ and the common variance σ^2 . Measured values then can be written as:

$$y_i = \alpha + \beta x_i + \varepsilon_i \tag{40}$$

where ε_i represents independent normally distributed random variables with zero means and a common variance σ^2 . This variance for "n" y values can be estimated in terms of the residuals as:

$$s_e^2 = \frac{1}{n - 2} \sum_{i=1}^{n} [y_i - (a + bx_i)]^2 \tag{41}$$

where s_e is the standard error of estimate. The standard error is in units of y and represents the limit within which approximately 68 percent of the absolute values of all errors will fall. Another quantitative measure of variance is the sum of the square residuals, or $(n - 2)\, s_e^2$.

The proportion of the variation of y values which can be attributed to the assumed relationship with x can be estimated as the ratio of the sum of squared residuals, $y - \hat{y}$, to the sum of squared deviations of y from the measured mean, $\bar{y}$, subtracted from 1, the square root of which is known as the nonlinear correlation coefficient, r :

$$r = \sqrt{1 - \frac{\Sigma(y - \hat{y})^2}{\Sigma(y - \bar{y})^2}} \tag{42}$$

The above relation has the advantage over other correlation formulas that it is not restricted to linear relationships, although it is more tedious to compute.

Confidence that can be placed on predictions made with an equation developed by the least squares method can be estimated by various methods (Miller and Freund 1985, Isaacson and MacKensie 1981). The upper limit of confidence in estimates applied as design criteria always should be addressed by engineers as an integral part of the design process, particularly if predictions are extrapolated beyond the range of measured data. Techniques for estimating statistical confidence are not discussed here in detail since this project does not directly involve extrapolation. It should be noted, however, that obtaining a large sample is very important in improving statistical confidence. LeMehaute and Wang (1984 and 1985) have made special note of the sensitive effect on confidence of wave statistics attributable to the number of years of record and frequency of recordings. Neglect of statistical confidence inherent in formulation of structural design criteria can lead to inadequate safety and higher than anticipated maintenance costs for structures involved. The 20 years of hindcast wave data at 3-hr intervals available from the WIS program are valuable in this regard.

CHAPTER III: CHARACTERIZATION OF WAVE CONDITIONS

Basic Sinusoidal Concepts

An understanding of the basic theory and terminology of water wave mechanics is necessary for interpretation of hindcast wave information and any analytical application of this information. Water surface waves are most easily described as wave forms of sinusoidal shape. Certain key terms with reference to this simplified concept of water waves, as illustrated in Figure 2, include:

1. Wave height, H - the vertical distance between a consecutive trough and crest

2. Wave length, L - the horizontal distance between two consecutive crests (or troughs)

3. Wave period, T - visualizing the wave form as travelling horizontally, the time for two consecutive crests (or troughs) to pass a fixed point, usually in seconds

4. Wave frequency, f - nominally, the rate at which consecutive crests (or troughs) pass a fixed point (= 1/T), in hertz (cycles per second)

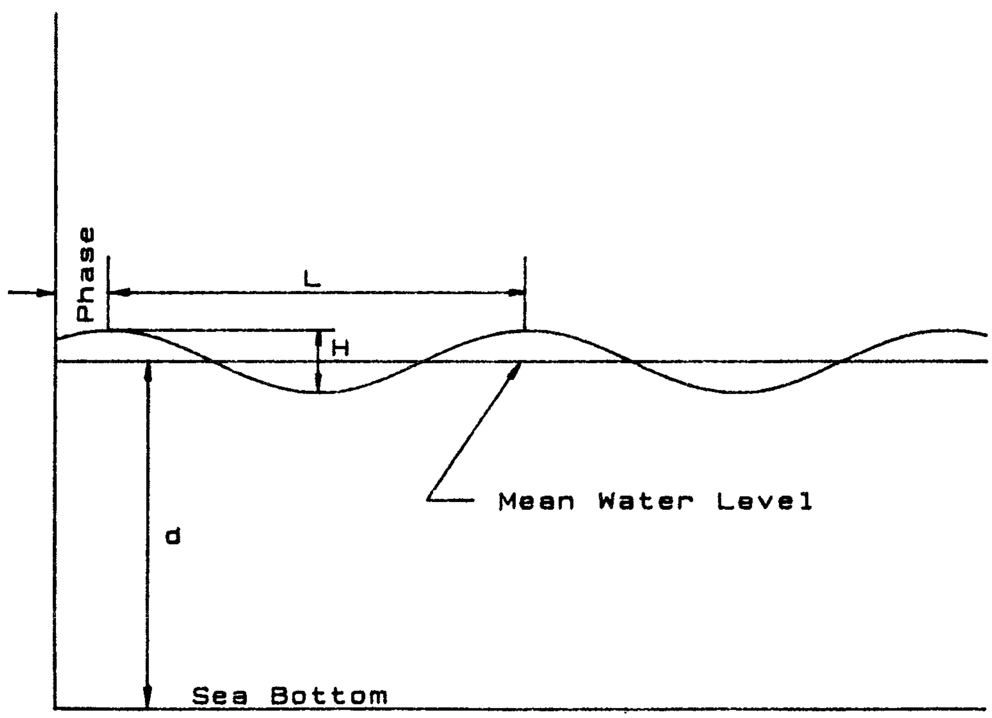

Figure 2. A sinusoidal wave

5. Radial frequency, ω - the radial equivalent of frequency ($\omega = 2\pi/T$), also in hertz

6. Wave number, k - the radial equivalent of wave length ($= 2\pi/L$)

7. Phase, ϕ - the radial equivalent of the horizontal displacement, x', of a wave crest from the origin of the reference axis at time, $t = 0$ ($= 2\pi/x'$)

The basic equation which defines the wave profile in these terms is:

$$\eta(x, t) = \frac{H}{2} \cos (kx - \theta t + \phi) \tag{43}$$

where $\eta(x, t)$ is the instanteous position of the water surface. Consideration of the sum of potential and kinetic energy inherent in a travelling wave of this form (per unit surface area) can be estimated by:

$$E = \frac{\rho g H^2}{8} \tag{44}$$

where ρ is the mass density of the seawater. This total energy is notably a function only of the wave height squared (Dean and Dalrymple 1984).

A consideration of surface, bottom, and transverse boundary conditions, with simplifications which eliminate all but first-order differential terms, yields the mathematical equation, known as the dispersion relation, which predicts effects of depth on wave length:

$$\omega^2 = gk \tanh (kd) \tag{45}$$

where g is the acceleration due to gravity and $\tanh$ is the hyperbolic tangent. A feature of sinusoidal waves which is consistent with this relation is that deepwater wave length, $L_o = (g/2\pi) T^2 = 5.12T^2$ ft or $1.56T^2$ m. The speed at which a wave crest travels, the phase velocity, C, in deep water $= L_o/T = 5.12T$ ft/sec or $1.56T$ m/sec. The change that occurs in shallower water is that wavelength shortens and phase velocity, $C = L/T$, increases. The wave height also is affected, first slightly decreasing, then increasing as the water grows more shallow. The overall tendency of water waves to change form as depths decrease is known as shoaling. The change in wave height due to shoaling is governed by:

$$K_s = \frac{H}{H_o} = \left(\frac{C_o}{2C_g}\right)^{1/2} \tag{46}$$

where H and H_o are shoaled and deepwater wave heights and K_s is the shoaling coefficient. The variable C_g is the shoaled group velocity, the speed at which groups of waves travel which is also the speed at which wave energy approaches shore:

$$C_g = \frac{C}{2}\left[1 + \frac{2kd}{\sinh\ (2kd)}\right] \tag{47}$$

where C is the shoaled phase velocity (= L/T) and sinh is the hyperbolic sin function (Dean and Dalrymple 1984).

The wave form becomes steeper in decreasing depths, ultimately reaching an unstable state when breaking occurs. The point at which breaking actually occurs is not fully understood at this time, but, based on the theory of solitary waves, generally occurs at the point where the wave height, $H = 0.78d$. Some field data tend to show that most locally wind-generated waves (i.e. "seas") break in deeper water, with breaking heights on the order of $0.6d$ to $0.7d$. Very long waves not locally generated (i.e. "swell") may not break until they are in very shallow water, however, since they may form surging breakers analogous to hydraulic phenomena known as "bores" or "hydraulic jumps."

The discussion above is meant to point out that there are practical limits to wave heights at most coastal sites due to breaking, but that these limits are as yet difficult to reliably define in practice. Furthermore, simplifications inherent in first-order sinusoidal theory are not sufficiently accurate for engineering purposes in many shallow-water situations and predictions made with a higher order wave theory must be applied.

Shoaling occurs only as a function of depth, but refraction also affects the wave form as a function of wave direction with respect to depth contours of the sea bottom. Refraction of water waves is analogous to refraction in classical physics of a ray of light passing through a pane of glass at an angle. The most frequently observed effect of water wave refraction is for waves approaching the coast at an angle to bend around as their crests tend to become parallel to the shoreline in shallow water. Snell's Law is usually applied to describe the change in angle of water waves by refraction in much the same way as it is in optics, commonly stated as:

$$\frac{\sin \theta}{C} = \frac{\sin \theta_o}{C_o} \tag{48}$$

where C and C_o are the refracted and deepwater phase velocities (= L/T and L_o/T) and θ and θ_o are the refracted and deepwater angles of wave crests with the bottom contours. Snell's Law assumes straight and parallel contours between deep water and the depth at which the above relation is applied. The relation can be applied in increments of incident versus refracted angles and thus applied to gently curving contours. Refraction usually (except in cases of convergence at convex contours) causes a reduction in wave height, which is superimposed on the effect of shoaling, according to the ratio:

$$K_r = \frac{H}{H_o} = \left(\frac{\cos \theta_o}{\cos \theta}\right)^{1/2} \tag{49}$$

where H and H_o are the refracted and deepwater wave heights and K_r is the refraction coefficient (Dean and Dalrymple 1984).

Wave diffraction describes the effect which a partial barrier has on wave heights beyond the barrier. It is the process which allows wave energy to leak sideways behind an obstruction or laterally from an area of high energy to an adjacent area of lower energy. The head of a breakwater, for example, will cause waves to diffract behind the breakwater into its geometric shadow, even though it may prevent any other form of wave transmission. Larger scale landforms and submerged formations can cause a degree of wave diffraction. Precise predictions of the effects of diffraction are more complicated than for shoaling and refraction, but the combined effects of these three forms of wave transformation are important in explaining observed behavior of water waves in many practical situations. The complexity of diffraction often requires the use of physical scale models to ensure with confidence satisfactory performance of protective structures such as breakwaters enclosing a port or harbor area.

Irregular Waves

The fact that real ocean waves typically appear chaotic with little regular form was mentioned previously. An explanation of this reality is that wave groups from many different sources with different heights, periods, phases, and directions are interacting in the small area we observe with the resulting superpositions appearing as chaos. Figure 3 illustrates a

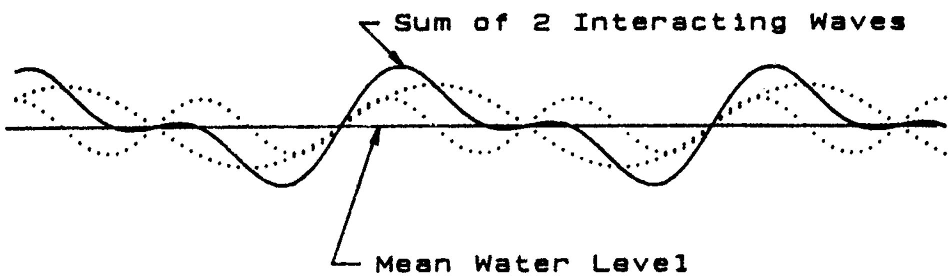

Figure 3. Interaction of sinusoidal waves

hypothetical point in time when two sinusoidal wave groups interact, one with 50 percent greater height and period and a $\pi/4$ phase difference. The waves would appear criss-crossed when viewed from above if their directions were not parallel.

Actually, winds that create the waves generate a range of heights and periods. Since phase velocity varies with period, longer period waves travel faster and soon leave shorter period waves behind. Swell, as previously defined, refers to waves which have completely left the area in which they were generated. These waves typically have periods greater than about 9 or 10 sec, but a clear distinction does not exist. Waves which are still within the influence of the generating wind system are called "seas" and typically are dominated by shorter period waves (less than 9 sec).

The distribution of individual wave heights in a stationary sea state has been found in most cases to follow a Rayleigh distribution, as discussed in the previous paragraphs on statistical concepts. Stationarity technically is the condition during which all moments (including the mean and variance) are time invariant (Bendat and Piersol 1971). A small sample thus can be analyzed and taken to represent the entire period during which conditions remain stationary. Waves at sea are assumed by most investigators to be weakly stationary for periods of about 3 hr, occasionally for as much as 6 hr, but seldom longer. This is more of a tradition related to the practicalities of collecting wave data than a precisely defined interval. The parameters derived from an instanteous measurement (such as the case of synoptic hindcasting) or from a 20-min recording of the water surface elevations are therefore typically taken to represent a much longer period during which conditions do not change. This, of course, is not really true, but as long as the changes are not drastic and are generally within the confidence limits of the statistical parameters of interest, this practice is acceptable.

Wave periods do not lend themselves as readily as do wave heights to representation by a standard statistical distribution such as the Raleigh distribution. Bretschneider (1959), however, found that the distribution of squared wave periods, T^2 , for seas followed a Rayleigh distribution. Other investigators have applied a variety of standard distributions, and specialized empirical distributions also have been developed.

The practice of coastal engineers in the last 10 years has largely shifted from considerations of wave period exclusively in the time domain to frequency domain considerations. Decomposition of a time series of water surface elevations into a set of incremental sinusoids, each represented by an amplitude (= $H/2$) and a frequency (= $1/T$), can be accomplished by transformation of the time series into its equivalent Fourier series. Wave conditions thus can be represented by the distribution of wave energy (proportional to amplitude squared per Equation 44) as a function of frequency, or a wave spectrum.

Figure 4 illustrates a wave spectrum with two "peaks," one representing swell-type waves and the other representing coexistent seas. The inverse frequency of the dominant peak is in practice usually taken as the peak period, which is generally assumed as the most probable period in the sea state. This is a "one-dimensional" spectrum which does not account for the direction of wave energy propagation. More complex procedures have been developed to express the distribution of wave energy as a function of both frequency and direction. The most common practice is to treat the directional spread of wave energy to be independent of the distribution of energy by frequency. This

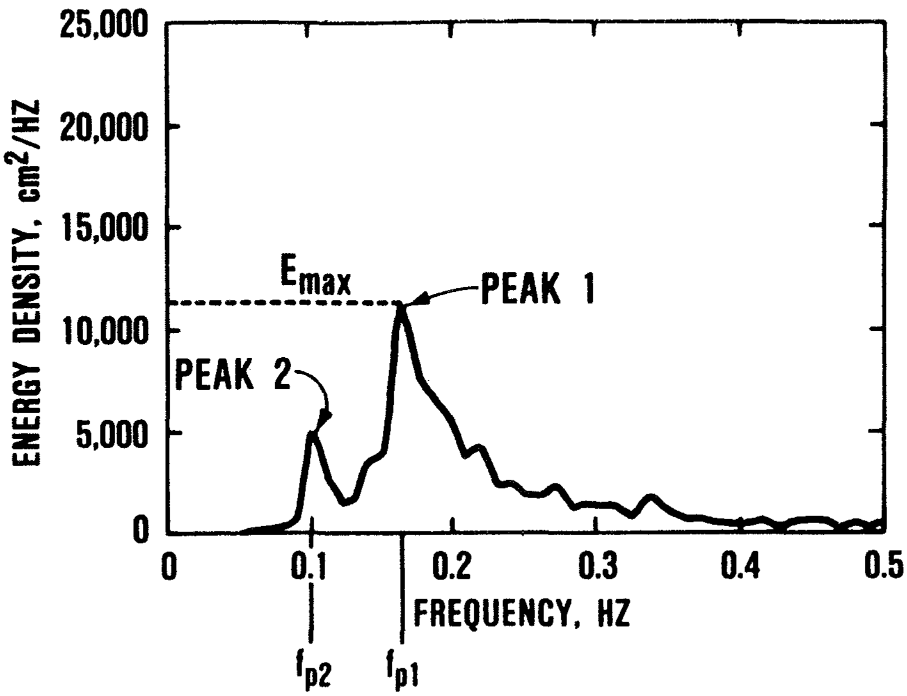

Figure 4. An example of a double-peaked energy density spectrum

allows application of a spreading function $\Theta(\theta)$ which, when multiplied by the one-dimensional spectrum $S(f)$, yields the two-dimensional spectrum $S(f, \theta)$:

$$S(f, \theta) = S(f)\ \Theta(\theta) \tag{50}$$

The form of a spectrum is quite sensitive to the analytical procedures applied, particularly "smoothing" performed to improve statistical confidence at the cost of resolution. Most spectral analysis procedures actually deal with discrete frequencies (= $2\pi/T$ of the individual sinusoids) which, when averaged over equal intervals, yield a smoother looking plot with more narrow confidence bands. A jagged looking spectrum will have wider confidence limits than a smoothed spectrum computed from the same data.

Integration of a wave spectrum which has been computed as energy per frequency band, $E/\Delta f$ (e.g. m^2/Hz), versus frequency yields the total energy of the sea state. This relates directly to actual variance of the water surface elevations such that:

$$\sigma_{ws}^2 = \int S(f)\ df \qquad (51)$$

where σ_{ws}^2 is the variance of the water surface elevations and $S(f)$ is the computed energy density spectrum. Spectra in this form are often taken as continuous functions since it is reasonable to expect wave energy to be generated in continuous frequencies.

A parameter in units of wave height which has been used to represent the range of wave heights in a sea state is the zero moment wave height, $H_{mo} = 4\sigma_{ws}$. The "zero moment" title comes from integration of $f^n S(f)$ with respect to f where n, the power of f in the integral, is zero as with Equation 51. This wave height has been found to be very close to the significant wave height, H_s, of Rayleigh distributed seas in deep water. H_s typically departs from H_{mo} in shallow water (Thompson and Vincent 1983). The zero moment wave heights corresponding to two interacting wave groups of double-peaked energy density spectra, as illustrated in Figure 4, can be estimated by splitting the spectrum between peaks and integrating each side separately. There is no widely accepted way to estimate the parameters of multiple wave groups from their combined spectrum, but this method gives an indication of their relative intensity as potential structural design criteria.

A number of parameterized spectra have been developed in the effort to relate wave conditions to winds and geographical factors which constrain generation of waves at sea. These parametric spectral forms nearly all apply to waves in the generation phase, i.e. seas, not swell. The four most important factors in wave generation are wind velocity (and resultant stress) over water, duration of that velocity, fetch (distance over water which the wind blows), and water depth. Depth limitations on wave spectra are the most recent effects to be reliably defined in combination with other primary constraints. Other factors which also can be significant are preexisting waves (wave-wave interaction) and the presence of strong currents (wave-current interaction). Waves generated by winds of a given velocity in water of a given depth thus are either duration limited, fetch limited, or fully developed and may be affected by waves coming into the generation area from a distant source and strong currents. Virtually all parametric spectral shapes have the "tail" of the spectrum, the portion to the right of the peak, proportional to f^{-5}, following the work of Phillips (1977). An advanced form, as an example, is the TMA spectrum (Hughes 1984), which includes the depth limitation:

$$S(f, d) = \alpha g^2 f^{-5} (2\pi)^{-4} \phi(2\pi f, d) e^{-5/4(f/f_p)^{-4}} \gamma^{\exp -(f/f_p - 1)^2/2\sigma_*^2} \quad (52)$$

where $\phi(2\pi f, d)$ is a function of depth (d), k (the wave number, $2\pi/L$), and ω (the radial frequency, $2\pi/T$) allowing portions of the spectrum to be transformed by linear wave theory. The term α is the Phillips equilibrium constant, which has recently been taken to be a function of depth, wind speed, and peak frequency, f_p. The γ term is the "shape parameter" which is a function of wind speed and fetch. The σ_* term is an empirical factor affecting shape of the spectrum on either side of the peak. This form applies to fully developed or "saturated" seas in decreasing depths. Figure 5 illustrates the effect of changing depth on TMA spectral shape. The deepwater predecessor of the TMA spectrum, the JONSWAP spectrum, now is widely used to predict both fetch and duration limited wave growth in deep water (Vincent 1984).

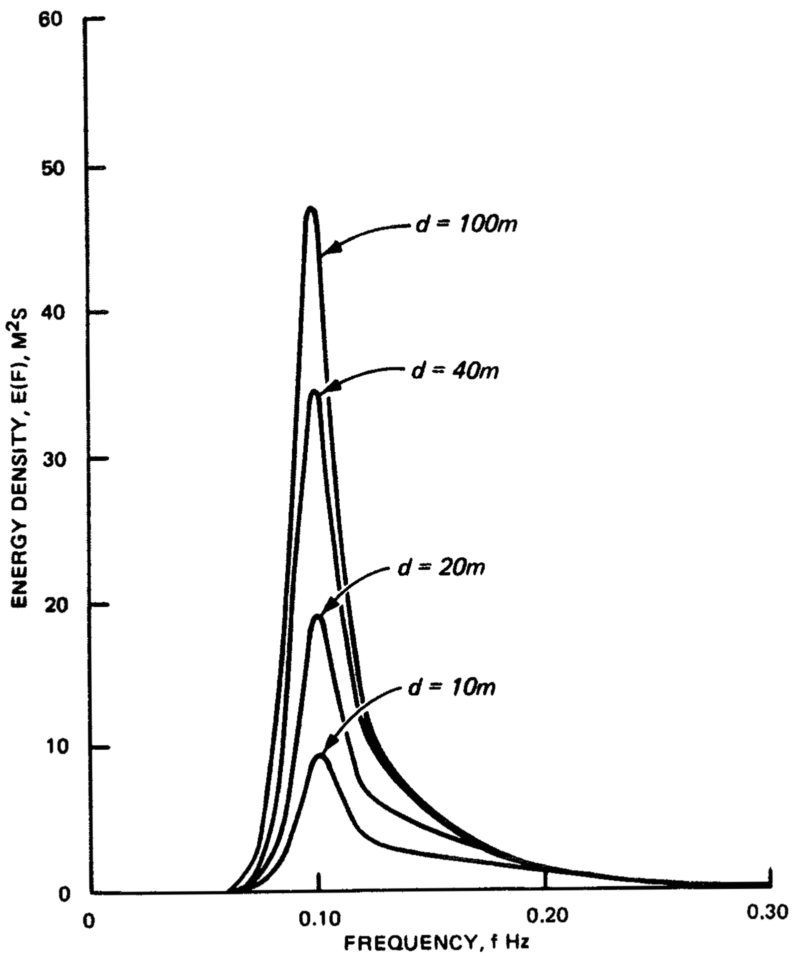

Figure 5. The TMA spectrum

CHAPTER IV: WAVE INFORMATION STUDIES HINDCAST DATABASE

General Background of Phases I and II

The WIS program of the US Army Engineer Waterways Experiment Station began in 1976 with the goal of providing a long-term (20-year) hindcast of wave information for use in development of design criteria for coastal projects. The term "hindcast" refers to the technique of simulating historical wind and wave generation from pressure data available from surface weather charts. The basic raw data for hindcasting thus are instanteous pressure recordings which meteorologists have applied to produce pressure fields delineated by isobars and other notation common to surface weather charts. These "highs," "lows," "fronts," "troughs," and "ridges" are then applied to simulate the effect of corresponding wind fields on the surface of the ocean.

The WIS program first transcribed into digital form pressures from surface weather charts from 1956-1975 for the North Atlantic, Gulf of Mexico, and North Pacific, with as much checking for accuracy and consistency as the basic data allowed (Corson, Resio, and Vincent 1980). This information was available at 6-hr intervals. Winds which would have existed with each consecutive distribution of pressures next were simulated by a series of numerical models assuming quasigeostrophic flows and a planetary boundary layer which yielded surface level (19.5-m elevation) wind fields. These wind fields were in turn adjusted with observations of actual wind velocities, wherever possible (Resio, Vincent, and Corson 1982).

Given the database of surface level winds created by the steps above, basin geometry and grid were defined for numerical simulation of deepwater wave generation. Figures A-1 and A-2 illustrate deepwater (Phase I) grids for the North Atlantic and North Pacific Oceans. Execution of a deepwater numerical model of wave generation, which took into account fetch, duration, directional spreading effects, and wave-wave interaction, produced a database of two-dimensional spectra and related parameters at intersections of the grid lines. Detailed wave information was retained only at intersections marked with dots and published in written form (Corson et al. 1981 and Ragsdale 1983) for the numbered sites.

Phase II of the WIS program performed simulations at 3-hr intervals of wave generation in a manner similar to Phase I (deep water), but at a finer scale and in transitional depths of the continental shelf. Figures A-3 and A-4 show the Atlantic and Pacific Phase II grids and stations where wave information has been published (Corson et al. 1982 and Ragsdale 1983). In addition to Phase I factors, Phase II simulations took into account the sheltering effect of large-scale land masses, refraction, and shoaling. The Phase I wave information served as a boundary condition at the seaward limit of the Phase II grid.

Neither Phase I nor Phase II distinguished seas and swell, but rather dealt with individual discrete frequency bands over the entire two-dimensional spectrum at any point. Phase III decomposed this spectrum into seas and swell, treating seas as two-dimensional spectra and swell as monochromatic, unidirectional wave groups. The definition of swell as waves which have travelled beyond the area in which they were generated was applied. This approach economized computations by taking advantage of the fact that swell typically has its energy highly concentrated in a narrow band of frequencies, which is close to a monochromatic condition. Wave parameters computed and recorded in the Phase III database included zero moment wave height, peak period, and dominant direction of propagation. Monochromatic equivalents were recorded in the case of swell and combined wave heights were recorded as:

$$H_{combined} = \sqrt{H_{sea}^2 + H_{swell}^2} \qquad (53)$$

Period and direction recorded in the "combined" category corresponded to the peak period and dominant direction of either the sea or swell, whichever had the higher zero moment wave height (Brooks and Corson 1984). The Phase III approach is most valid for coasts with straight and parallel contours and is less precise in more complex bathymetry.

Phase III Shallow-Water Wave Information

Phase III efforts of the WIS program were directed at providing wave information suitable as design criteria for a great many coastal endeavors in a depth of 10 m at 10-mile (16.1-km) intervals along the Atlantic (Jensen 1983a) and Pacific (Ragsdale 1983) coasts of the continental United States. This task dealt with transformation of wave conditions from Phase II stations to

166 Atlantic and 134 Pacific Phase III stations. Figure A-5 illustrates a section of the Atlantic Phase III stationing system and adjacent Phase II stations. The magnitude of data processing requirements and complexity of the coast at this finer scale led to procedures for estimating wave conditions in shallow water (10 m depth) described briefly below.

A spectral (frequency domain) approach to wave transformation was sought to reduce computational time required to simulate wave transformation in the time domain. A parameterized spectrum was necessary for this, but one as complex as the TMA spectrum, or the most refined spectral forms available at the time of the Phase III procedure formulation, would have provided an unmanageable computational burden. The one-dimensional parameterized spectrum chosen for Phase III simulations had the following form:

$$S(f) = \alpha g^2 f^{-5} (2\pi)^{-4} \qquad \text{for} \quad f \geq f_p \tag{54}$$

$$S(f) = \alpha g^2 f_p^{-5} (2\pi)^{-4} \exp\left[1 - \left(\frac{f}{f_p}\right)^{-4}\right] \qquad \text{for} \quad f < f_p \tag{55}$$

which applied the well-accepted f^{-5} right-hand tail, but limited free parameter determination to only two variables, α and f_p (Kitaigordskii 1962).

A spreading function, assumed to be independent of the one-dimensional spectral form, was defined as:

$$\Theta(\theta) = \frac{8}{3\pi} \cos^4 (\theta - \theta') \tag{56}$$

where θ' is the predominant direction of propagation. Thus, the two-dimensional form was:

$$S(f, \theta) = S(f)\Theta(\theta) \tag{57}$$

Within each 10-mile (16.1-km) interval defined as Phase III stations along the coast, bottom contours were assumed to be straight and parallel. A specific orientation was assigned to each interval such that departure of this assumption from the true situation was minimized. The processes of refraction and shoaling, as defined by Snell's Law and sinusoidal theory, were applied to increments of frequency and direction of the directional distribution defined by $S(f, \theta)$. Wave energy propagating seaward was ignored.

The geometric relationship between a Phase III station and adjacent Phase II stations from which the model derived its input was the most important consideration in addressing sheltering in Phase III. Basically, the geometric shadow of a landform to wave energy from a specific direction was considered as absolute, i.e., no energy was propagated into the shadow area. This is a gross simplification, but it made the simulation of sheltering effects practical for Phase III. Discrete combinations of frequency and direction were considered incrementally with respect to sheltering, as they were with refraction and shoaling.

The problem of wave-wave interaction and the losses it can cause, evidenced by white caps and other signs of turbulent energy dissipation, was addressed by definition of another spectral form for shallow water. Principles of similarity were applied to derive a form consistent with Phase I and II deepwater considerations, which predicted the spectrum in shallow water:

$$S(f) = \alpha g h (8\pi)^{-2} f^{-3} \qquad \text{for} \quad f \geq f_p \tag{58}$$

This relation is consistent with the visualization that energy losses due to wave-wave interaction tend to occur at high frequencies, while energy at lower frequencies is conserved. A further application of equilibrium principles allowed derivation of an integrated form of this equation which describes the dependency of sea wave heights on depth:

$$(H_{seas})_{max} = \frac{(\alpha g d)^{1/2}}{\pi f_c} \tag{59}$$

where $f_c = 0.9 f_p$ is a energy cutoff frequency (lower integration limit) and $(H_{seas})_{max}$ is the upper limit of seas wave heights. Surf zone breaking was treated differently for swell, however, in the manner of estimating breaker heights for monochromatic waves. A breaking coefficient of 0.6 was applied, which is consistent with recent measurements of breaking waves by the WES (Jensen, Robert E., verbal communication, February 1986):

$$H_{max} = 0.6d \tag{60}$$

Extensive comparisons have been made between the limited measured wave data available and WIS wave information, generally with acceptable results (Corson and Resio 1981). The reduction of measurements made by wave gages also involves compounded assumptions, and discrepancies between wave information based on gage data and Phase III wave information could not always be resolved. More accurate techniques are available for site-specific simulation of the transformation of waves into shallow water. These methods unfortunately were too complex to apply systematically on the scale of the WIS Phase III endeavor, though improvements are under consideration. The presently available end product of Phase III is, however, an excellent tool for coastal engineers to use in the planning and preliminary design stages of coastal projects for development of design criteria. More complex and expensive numerical simulations and physical scale models can be performed in the detailed design phase after the economic feasibility and financeability of the project has been ensured. Even in the final stage, some basis of experiment design and cross-check on other sources of wave information is necessary. The 20-year period of record for the WIS database can rarely be exceeded by other reliable sources. The WIS wave information provides, therefore, a vast improvement to the confidence of each design effort to which it is applied.

CHAPTER V: LITERATURE REVIEW OF STORM DURATION STUDIES

Recent Literature on the Duration of Sea States

Table 1 presents mean durations for various weather types in the British Isles which were excerpted from Barry and Perry (1973). The weather type

Table 1

Mean Durations of Weather Types in the British Isles

	Mean Duration (days)			
	January		July	
Weather Type	1910-1930	1948-1968	1910-1930	1948-1968
Westerly	4.1	1.7	2.6	2.7
Northerly	1.5	1.8	2.0	1.8
Easterly	1.9	2.6	2.0	1.8
Southerly	2.0	1.8	1.7	1.3
Cyclonic	1.4	1.3	1.7	1.9
Anticyclonic	2.2	1.9	2.5	2.2

identified as "cyclonic" is assumed to meet the standard definition of winds circulating around a low pressure area (Lester 1973), corresponding to the extratropical cyclonic events which are simulated in the WIS program. This type of weather is noted to have a mean duration of 1 to 2 days in Great Britain, with some seasonal variation. Statistics of this type would surely vary from region to region, but the order of magnitude in hours, say less than 100 but more than 10, can serve in this investigation of storm characteristics as a rough first measure of a reasonable mean duration. The untrained intuition of any regular viewer of television weather reports would likely agree with this typical range.

Surprisingly little material was available in the coastal engineering and oceanographic literature which dealt directly with the duration of extreme events at sea or of extreme wave conditions. Occasional references were made to a 3-hr period of wave height stationarity assumed for practical purposes in measurement programs (e.g., Agerschou et al. 1983 and Massie 1976). The interval between samples of wave measurements is commonly set at 3 hr.

Publications of WIS wave information (Corson et al. 1981 and 1982 and Jensen 1983a) tabulated durations of significant wave heights above selected thresholds, but did not discuss trends or other implications inherent in this information.

North Sea Investigations of Houmb and Vik

The most rigorous work to date has been a series of studies by two Norwegian investigators (Houmb 1971, Houmb and Vik 1975, Vik and Houmb 1976, and Houmb and Vik 1977). Other authors have reviewed this work (e.g. Battjes 1977, PIANC 1979, and Bruun 1985), but no significant advances seem to have been made regarding the characterization of extreme event durations following Houmb and Vik (1977). Their work on the duration of sea states culminated in the findings of the last reference, which will be reviewed in detail in the following paragraphs.

Houmb and Vik (1977) considered both the duration of extreme events, specified as the time during which the significant wave height exceeded a given threshold, and the duration of "calms" between these extremes. The basis of their investigations was wave recordings made at five North Sea sites where depths varied from 80 to 250 m. Three sites involved time series measurements made for 20 min every 3 hr. A fourth site involved 10-min time series measured every 4 hr. The sequences of these measurements were not continuous and varied in total period of record from 3 to 31 months. The fifth site provided observations from a rescue vessel every 3 hr from 1959 to 1974 during October through March only. These observations classified predominant wave heights into classes of 0.5 m.

A theoretical approach toward prediction of variation of storm durations was first proposed by Houmb and Vik (1977) which took the frequency, or marginal probability density, of threshold up-crossings (i.e. $H_s' = dH_s/dt$ was positive) as:

$$f(H_t) = H_s' f(H_t, H_s')\ dH_s' \tag{61}$$

where H_t is the specified threshold and $f(H_t, H_s')$ is the joint probability density of H_s and its time derivation, H_s' . The average duration of extreme events, $t(H_t)$, $(H_s > H_t)$ was derived to be:

$$t(H_t) = \frac{L[1 - F(H_t)]}{f(H_t)L} = \frac{[1 - F(H_t)]}{f(H_t)} \tag{62}$$

where L is the period of interest (say 50 years) and $F(H_t)$ is the cumulative distribution of H_s evaluated at H_t, or the probability that H_s is equal to or less than H_t. The quantity $[1 - F(H_t)]$ is the probability that H_s is greater than H_t. The average number of up-crossings, i.e. the average number of extreme events, during the period L was taken to be $f(H_t)L$, where $f(H_t)$ is the probability density of H_s at H_t given above.

The rate at which H_s changes (from one stationary period to the next) was assumed to be a Poisson process, i.e. H_s' was assumed to be independent of H_s. The joint probability density function $f(H_s, H_s')$ could then be evaluated as:

$$f(H_s, H_s') = f(H_s)f(H_s') \tag{63}$$

The marginal probability density function $f(H_s)$ was assumed to follow a Weibull distribution whose corresponding distribution function had the form:

$$f(H_s) = 1 - \exp\left(-\frac{H_s - H_o}{H_c - H_o}\right)^T \tag{64}$$

and

$$f(H_s) = \frac{T(H_s H_o)^{T-1}}{(H_c - H_o)^T} \exp\left(-\frac{H_s - H_o}{H_c - H_o}\right)^T \tag{65}$$

where H_c, H_o, and T are parameters of the distribution.

The function $f(H_s')$ was assumed to be normally distributed with zero mean for positive values of H_s' (increasing H_s). The data seemed to support this assumption. This gave $f(H_s')$ as:

$$f(H_s') = \frac{1}{2\pi\sigma_h} \exp\left(\frac{-H_s'^2}{2\sigma_h^2}\right) \tag{66}$$

where σ_h is the standard deviation of H_s' which was evaluated from the data. The differences between σ_h values computed for increasing and decreasing H_s were found to be negligible. Furthermore σ_h was not noted to follow a seasonal pattern. This application of the above normal distribution with zero mean gave the advantage of requiring only one parameter, σ_h, to be determined empirically, in addition to those (H_c, H_o, and T) for $F(H_s)$. The resulting function for the mean duration $t(H_t)$ reduced to:

$$t(H_t) = \frac{2\pi(H_c - H_o)^T}{T\sigma_h(H_t - H_o)^{T-1}} \tag{67}$$

The cumulative distribution of measured durations was found to be well represented by a Weibull distribution of the form:

$$F(t) = 1 - \exp\left[-\left(\frac{t}{t_c}\right)^{\alpha}\right] \tag{68}$$

where α is the shape parameter and t_c is the scale parameter. Average durations estimated by the $t(H_t)$ function derived above also compared well with means computed from the set of measured durations. Houmb and Vik (1977) gave examples of how this formulation could be applied in the conduct of offshore oil explorations, as in prediction of duration of operation down time caused by extreme wave conditions.

The formulation of Houmb and Vik (1977) was well defended in terms of conceptual limits of parameters such as H_s' and σ_h. They tested their hypotheses as well as possible with their limited data set, but urged in their conclusions that further investigations be pursued with more comprehensive wave information.

CHAPTER VI: EXTREME EVENT IDENTIFICATION

Choice of Sites

Each Phase III site includes 58,440 records of wave information 3 hr apart from 0000 (midnight) January 1, 1956, to 2400 (midnight) December 31, 1975 (20 years). Four sites were originally chosen for analysis, two on the Atlantic coast and two on the Pacific coast. A third Atlantic site was later chosen when it was discovered the first two had very similar distributions of significant wave heights. The five sites ultimately investigated are listed in Table 2. They were intended to represent a wide geographical spread in

Table 2

WIS Phase III Stations Investigated

Station	Site	Latitude	Longitude
A3061	Atlantic City, New Jersey	39.34° N	74.47° W
A3083	Nagshead, North Carolina	35.94° N	75.61° W
A3142	Daytona Beach, Florida	29.20° N	81.00° W
P3036	Newport, Oregon	43.63° N	124.08° W
P3105	Half-Moon Bay, California	37.45° N	122.45° W

hopes that analysis would reveal any important universal traits or significant geographical differences. Figure 6 shows their relative location along the US coasts. Statistics published by the WIS program (Jensen 1983a, b) for the Atlantic sites are presented in Appendix A. Wave height frequency tables (not yet published by the WIS program) for the two Pacific sites also are presented in Appendix A.

Basic Treatment of WIS Phase III Wave Information

Table A1 illustrates format and unit conventions of the WIS Phase III database. Dates are given as year/month/day and times referenced to the 24-hr clock (i.e., military time). Wave heights, i.e. the zero moment wave heights derived for each 3-hr time step, are reported in centimetres. Wave periods,

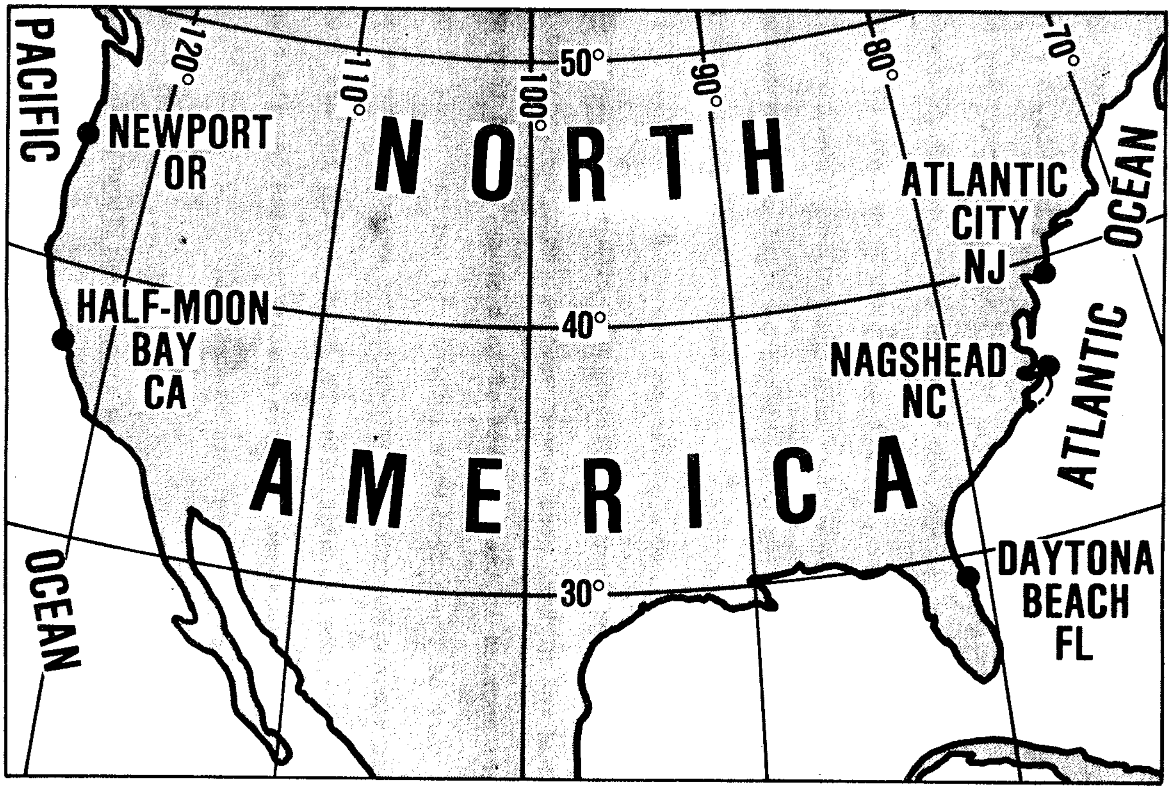

Figure 6. Geographic relation of sites investigated

i.e. the peak periods of the hindcast spectra, are reported to the nearest second. Direction or azimuth is reported in degrees relative to the shoreline, such that 90 deg is a wave direction travelling straight into the straight and parallel contours assumed for each 10-mile (10.1-km) shoreline increment. Combined statistics presented in Table A1 include the geometric average wave height (Equation 53) and the peak period and predominant direction of either seas or swell, whichever had the highest zero moment wave height. Combined statistics were applied in analyses of this study, though they were not actually a part of stored wave information and had to be computed. Mean and maximum duration of exceedance of selected wave heights were reported for the Atlantic sites by Jensen (1983b) and are included in Appendix B. A comparison of those statistics with results from this investigation is made later in this report.

The Problem of Extreme Event Identification

The work of Houmb and Vik (1977) on duration of sea states was apparently performed exclusively with significant wave heights crossing an arbitrary threshold. This implies that the significant (or zero moment) wave height is the most appropriate measure of the extreme events' intensities for applications of duration statistics. Other parameters can be conceived, however, which might be better representatives of the overall intensity or extreme nature of a storm. The most obvious alternate parameter would be peak period, to which refraction, shoaling, and wave breaking are all quite sensitive. Wave length might be another, although wave length at any depth is a function of period. Wave steepness, H/L, is commonly associated with breaker characteristics and forces on coastal structures. If the ratio of zero moment wave height to deepwater wavelength corresponding to the peak period is used, representative wave steepness becomes $2\pi H/gT^2$. The 2π factor is commonly dropped as a part of this dimensionless steepness parameter in favor of H/gT^2.

Wave severity, H^2L, has recently become of interest as a factor closely related to stability of rubble-mound structures (Graveson et al. 1980 and Ahrens 1984). Wave severity can be thought of as the ratio of wave height cubed (the traditional wave parameter for evaluation of rubble-mound stability) to wave steepness, H/L. Again, significant or zero moment wave height and deepwater wave length corresponding to the peak period of the spectrum are used for convenience, yielding $H^2L = 2\pi H^2/gT^2$. It should be noted that the four parameters discussed so far vary the relative influence of wave height and period in the following order: H, T, H/T^2, and H^2/T^2. These parameters also could be used to define extreme event duration as the time during which consecutive parameter values exceed a specified threshold value.

A fifth parameter which might be important with respect to duration of extreme wave conditions is predominant wave direction. This certainly would be true for sites naturally protected in all but one narrow sector. WIS Phase III data did not include any such sites, however, assuming an open coast with sheltering only from major landforms.

Figure 7 illustrates the time series for wave heights during October 1956 at Nagshead, North Carolina. This particular time span was chosen for presentation because it included rapid changes in wave conditions, especially on October 27 and 28, 1956, as indicated by sharp spikes near the end of the wave height time series plot of Figure 7. Table A1 includes Phase III wave information recorded for these 2 days.

Figure A6 shows the time series of peak wave period during this same

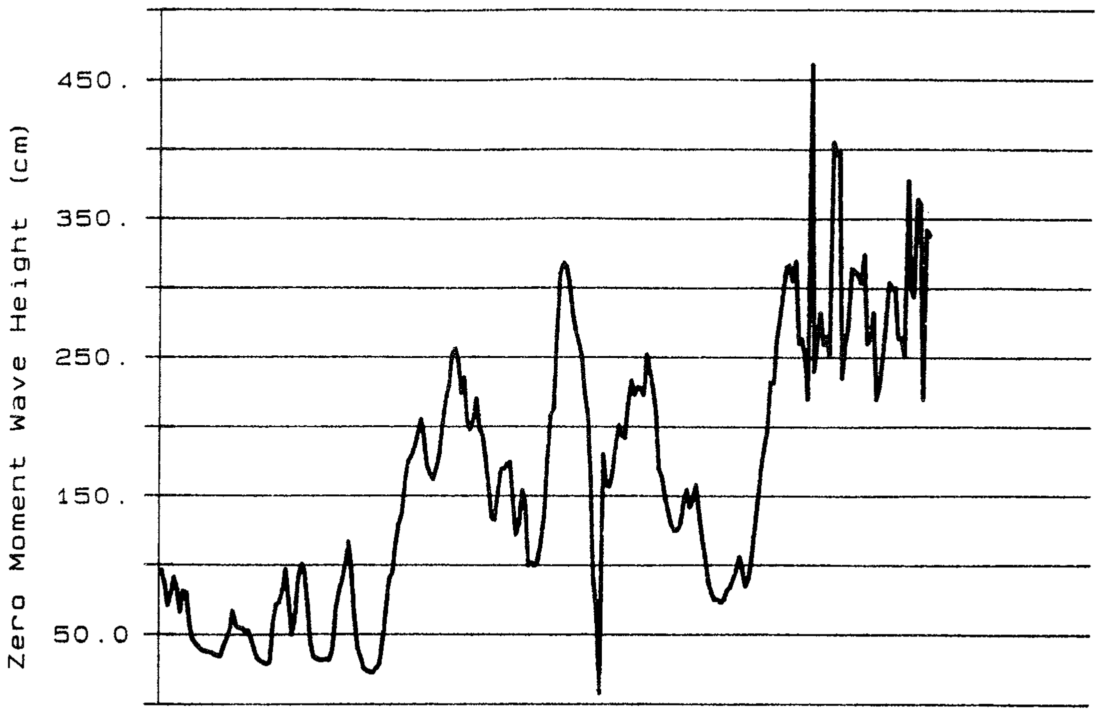

Figure 7. Wave height time series: Nagshead, North Carolina, October 1956

month for Nagshead, North Carolina. The wave period can be seen to vary somewhat out of phase with wave height and to have a tendency to remain constant for significant time spans and then change abruptly. A plot of H/gT^2 for the same period at Nagshead (Figure A7) appears more like the wave period time series than the wave height time series, also tending to vary slowly for significant time spans and change abruptly (the influence of T^2 in the denominator). A plot of wave severity, H^2L (Figure A8), for the same time period dramatically delineates extremes of the wave height time series. When plotted in Figure A9 as $(H^2L)^{1/3}$, however, wave severity very closely resembles the wave height time series plot. Wave severity in this form has the same units as wave height and includes the influence of H^2 to balance the influence of T^2 in the denominator. The plot of direction (Figure A10) does not indicate direct relation to the wave height plot and is much more erratic, even in non-extreme periods. Direction can be considered to be practically independent of the intensity of wave conditions since it is controlled almost exclusively by geometric factors.

The convention of previous investigators (Houmb and Vik 1977) to rely solely on variation in zero moment wave height for definition of extreme event durations was maintained in this study. This parameter is most easy to visualize and has a long tradition as the critical measure of intensity of extreme events at sea. Variations of H^2L and $(H^2L)^{1/3}$ show promise, but the large units of H^2L make results of computations rather abstract and the variation of $(H^2L)^{1/3}$ seemed quite close to that of H. Relationships of individual extreme event durations (measured by variation of H) to peak conditions measured by all parameters discussed above were investigated, however, and the results of that analysis are reported later in this report.

An investigation of actual weather conditions on the Atlantic coast in the time frame surrounding October 27-28, 1956, was conducted to better understand what events were actually driving the numerical simulations to produce irregularities in the time series of Figure 7. First, Phase II data input to the Phase III numerical wave transformation were inspected. Table A2 presents Phase II information at Station A2037, at 36.06° N latitude and 74.92° W longitude, approximately 33 nautical miles (61 km) east-southeast of Nagshead in about 240 ft (73 m) of water. The intermittent appearance and disappearance of swell can be seen to follow a similar pattern in the Phase III site of interest (Station A3083) and the Phase II site directly offshore (Station A2037). Wave heights in deeper water are higher, lacking the depth limitations inherent in Phase III simulations. Wave periods of both sites are identical, unaffected by the wave transformation processes simulated in Phase III. The direction convention in Phase II is different, indicating the direction from which waves are travelling toward the center of the compass rose. Phase II data do not include anything significantly revealing about the irregularities of interest, basically showing the same patterns in this case.

The nearest Phase I site offshore of Nagshead was Station A1005 at 35.4° N latitude and 72.3° W longitude, located in deep water approximately 163 nautical miles (302 km) east-southeast of Nagshead, North Carolina. Table A3 shows Phase I information recorded for October 26-28, 1956. There is only one record which included swell; that record did not dominate the combined wave height, which appears to be steadily decreasing at that time. It is important to recognize that a significant travel time would be involved between this Phase I site and Stations A2037 or A3083 (approximately 8 and 10 hr, respectively, for waves of 11-sec period), so the conditions at a given date and time should be "out of phase" by three to four records.

An inspection of surface weather charts during the later part of October

1956 for North America and the north Atlantic Ocean was made to identify synoptic weather systems which may have dominated Phase I and Phase II information. An explanation was sought for the sudden appearance and disappearance of swell in the data, as well as an explanation of differences between Phase I and Phase II wave information. Figure 8 illustrates recorded weather patterns of October 26-28, 1956, showing the presence of a generally stationary, weakly defined, low pressure system of fluctuating intensity offshore of Cape Hatteras. This location is close to Station A1005; thus, the basic definition of swell as waves which have left their area of generation could explain the lack of swell in Phase I data. The wave field at this point would have been under the influence of cyclonic winds of the low pressure system and thus only seas would have existed, as defined by WIS conventions. The relative position of Stations A2037 and A3083 in combination with the fluctuating intensity of the low pressure system appears to have caused swell either to come from too far south to affect Nagshead or to exist only as seas, except for the spikes of Figure 7. This set of circumstances is probably exceptional, but an understanding of the real weather patterns driving numerical simulations of the WIS program in this instance may help explain trends of duration revealed by further analysis of WIS data.

Analytical Procedure and Results

A FORTRAN computer program was written which read the 58,440 records stored for each Phase III site and maintained a record of the number of consecutive records, each of which had a combined wave height above a specified wave height threshold, H1 . Subsequent use of the term "extreme event" refers to events defined in this manner. The number of extreme events was counted and statistics including the maximum, minimum, mean, and standard deviation durations were computed. Peak conditions of each extreme event were noted as the highest combined wave height in a consecutive series above the threshold, and the period and direction of sea or swell, whichever had the highest incremental wave height. Maximum, minimim, mean and standard deviation wave heights also were computed. Each data set included 20 years ot record, so the number of extreme events per year (the Poisson lambda parameter) was computed as the total number of events divided by 20.

Initial runs of this extreme event identification program resulted in a

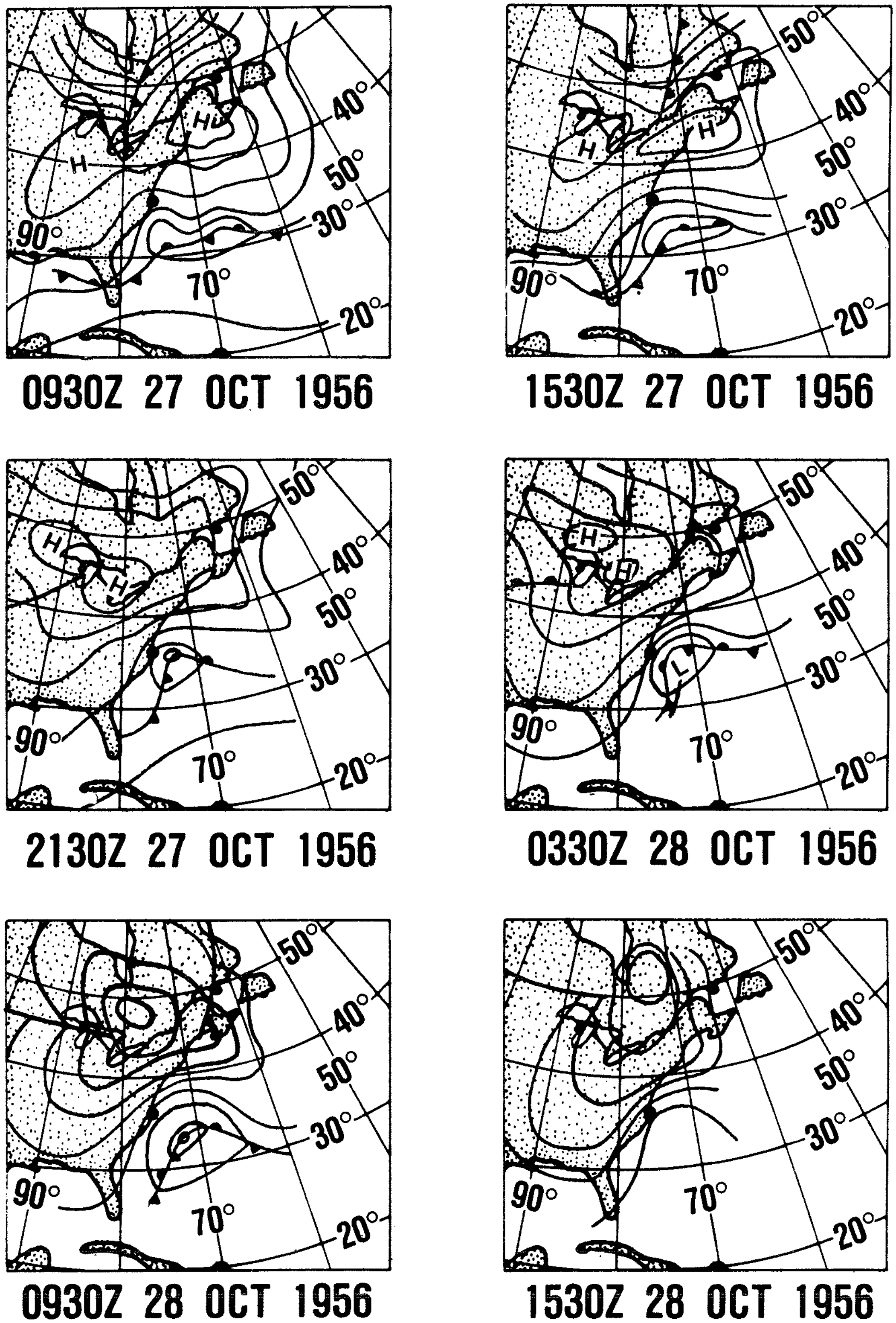

Figure 8. Eastern US surface weather patterns: October 1956

surprisingly large number of extreme events, consistently on the order of 30 to 40 percent of all extreme events identified, to be only 3 hr in duration, i.e., only one record above the threshold. The actual duration could be anywhere from 0 to 6 hr for a single record above the threshold, but an average value of 3 hr was consistently assumed in such cases. Variation of the threshold had little effect on the percentage, although the total number of extreme events was of course affected. Review of climatology considerations inherent in WIS simulations (Resio and Hayden 1973; and Corson, Resio, and Vincent 1980) did not uncover a rationale for excluding a priori durations that short. In fact, a duration of 3 hr is either implicitly or explicitly assumed for peak conditions in many wave forecasts, designs, and research efforts relating to the tradition of sampling wave gages at this interval. Average low pressure systems which would generate extreme wave conditions are known to typically last much longer, however, as in the case of the system illustrated in Figure 8.

In view of this last fact and the example of late October 1956 at Nagshead, the program was adjusted to ignore a lapse below the threshold of only one record (i.e., 6 hr) between consecutive extreme events, as identified previously. This adjustment lowered the number of extreme events of only 3 hr duration (one record above the threshold) only slightly, but a neglect of longer lapses or other adjustments to the identification procedure could not be rationalized. Tables A4-A8 give duration results, following the procedures described above, for the five sites at all thresholds investigated.

Mean and maximum durations for the three Atlantic sites are virtually identical to those reported by Jensen (1983a), with the occasional exception caused by combination of two events separated by only one record with H below the threshold. The mean duration was slightly higher in these few cases.

The percent occurrence of wave heights (percent records $H > Hl$) was of special interest since this statistic for a range of Hl levels is now or will be published and readily available for all WIS stations of all three phases. It was hoped this nondimensional parameter could be used as a tool for choosing threshold levels for duration computations which would preclude many of the iterations which otherwise might be necessary. The number of extreme events per year was also of special interest since this parameter is so important in extremal statistics and expectations.

Figure 9 shows the mean and standard deviation of duration plotted against percent occurrence (actually exceedance) of wave heights above the

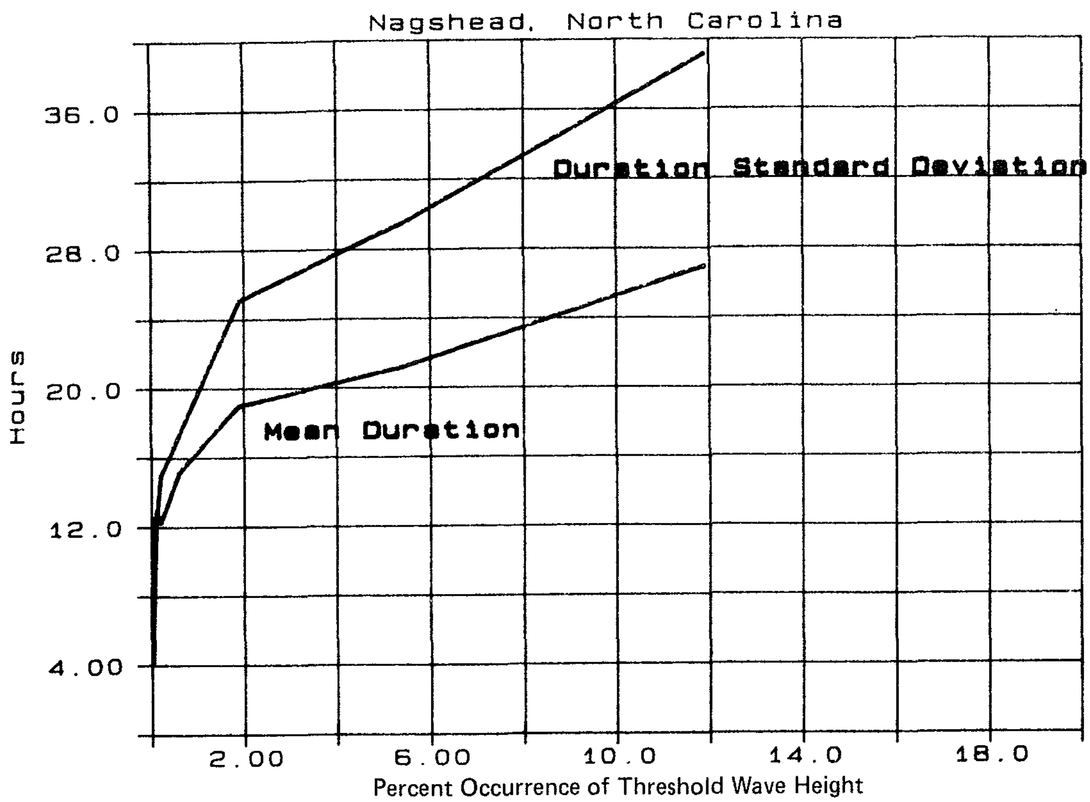

Figure 9. Mean duration and standard deviation versus percent occurrence of wave height threshold, Nagshead, North Carolina

specified threshold for Nagshead. Figures A11-A14 show mean and standard deviation durations for the other four sites plotted against percent occurrence of wave heights above the threshold. Figure 10 shows the nearly linear relationship of the number of extreme events per year with percent occurrence of wave heights above a specified threshold for Nagshead. A similar trend is evident for higher wave with similar percent occurrence at Newport, Oregon, as shown in Figure A15. These plots in themselves do not indicate an outstanding range of percent occurrence as a choice for definition of extreme events and durations. Some subjective choices can be made since an important purpose of this exercise is to identify extreme events. Clearly, an excessively large number of extreme events per year, say more than 20, will probably include some events that can hardly be regarded as "extremes" in the practical sense. On the other hand, an average number of extreme events per year less than one or two would generally imply exclusion of some events which belong in a

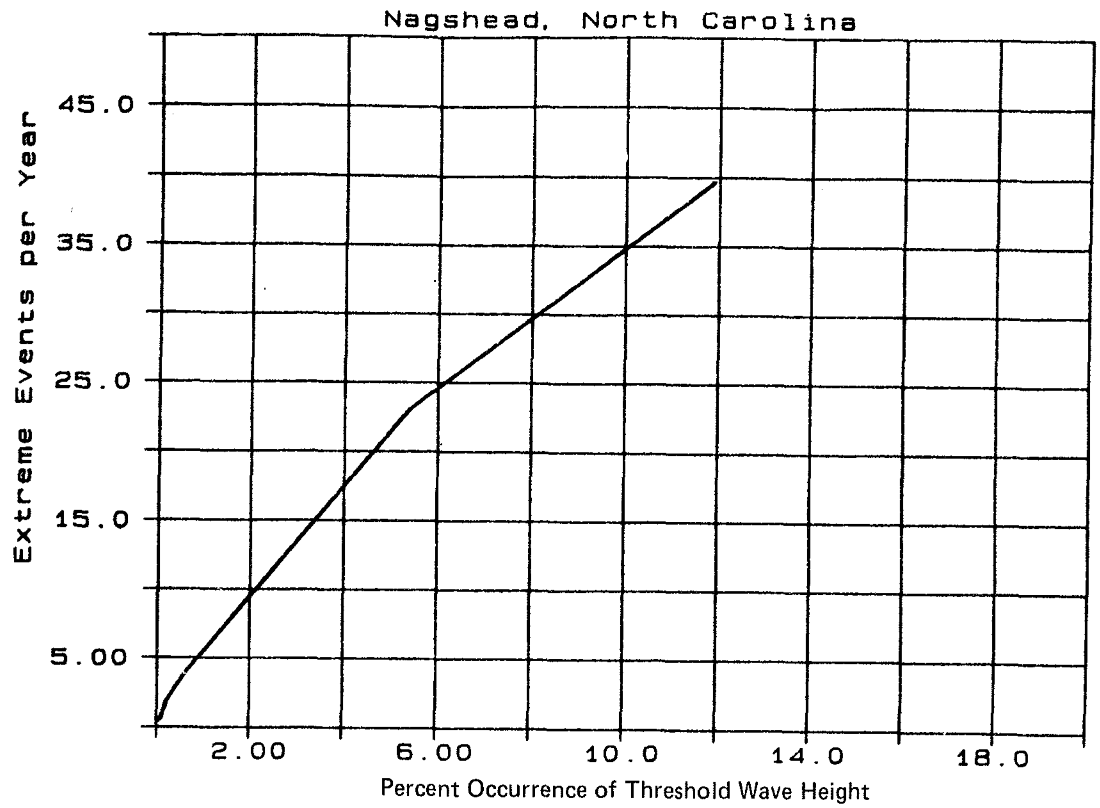

Figure 10. Extreme events per year versus percent occurrence, Nagshead, North Carolina

population of extremes. These considerations are consistent with the author's experience in developing design criteria based on extremal statistics of peak wave height conditions (e.g. Andrew, Smith, and McKee 1985).

A simple linear regression of extreme events per year with percent occurrence of wave heights above the threshold, constrained to pass through the origin, for the 41 cases considered at all five sites indicates that percent occurrence = 0.3λ with a correlation coefficient of 0.97. This relation applies to both the Atlantic and Pacific sites addressed individually, even though the absolute value of wave heights themselves on the Pacific are substantially higher than those on the Atlantic at the same percent occurrence levels. A range in λ of 2 to 20 thus would correspond to a range in percent occurrence of 0.6 to 6.0 percent for the choice of a desirable threshold level, $H1$. The lower limit of this range would guarantee a sample size of at least 40 extreme events, which is generally desirable for most statistical considerations. The choice of a threshold wave height may be made more precisely when some physical tolerance level is at issue, for example the point at which some operation at sea must be temporarily terminated.

The other parameters presented in Tables A4-A8 show interesting trends. The minimum duration was 3 hr in every case except one where only three extreme events were identified. A count of extreme events with a 3-hr duration for the Nagshead cases indicated 32 to 48 percent of extreme events shared the minimum duration. No relation of the number of extreme events with a 3-hr duration to the threshold level was apparent. The maximum duration can be seen to be proportional to percent occurrence of wave heights above the threshold and typically many standard deviations above the mean. The mean duration accordingly also is proportional to percent occurrence of wave heights above the threshold. The standard deviation was rarely less than the mean, but always of the same order of magnitude. A lack of central tendency for durations was noted by Houmb and Vik (1977).

Another scheme of extreme event identification was investigated which actually applied a lower threshold H1 in the same way for determination of duration, but only to extreme events whose peak (combined) wave height was above a second higher threshold, H2 . The most notable effects of the second threshold were to substantially reduce the number of extreme events per year for a given H1 threshold and to reduce the number of extreme events with a 3-hr duration to zero in nearly every case. Variation of H1 with a fixed H2 had little effect on the number of extreme events per year. The central tendency of durations was somewhat stronger in these subsets, with the standard deviation often, but not always, less than the mean. These two parameters consistently retained the same order of magnitude.

Tables A9 and A10 present the parameter values computed for various combinations of H1 and H2 at the Nagshead and Daytona Beach sites. Figures A16 and A17 show variation of the mean and standard deviation durations with percent occurrence of the lower threshold H1 at a upper threshold H2 for peak conditions fixed at 300 cm (0.6 percent) and 300 cm (0.8 percent) for the Newport and Nagshead sites. This scheme of double thresholds for extreme event identification was not pursued further since it was considered more desirable to address trends in peak conditions separately from durations above a specified threshold. An approach which addressed marginal distributions versus conditional distributions was preferred.

CHAPTER VII: DISTRIBUTION OF DURATIONS

Method of Analysis

The cumulative probability of durations derived by the single threshold method described above is estimated by application of a plotting formula commonly applied in analyses of this type (Gumbel 1958, and Isaacson and MacKensie 1981):

$$F(t_i) = \frac{i}{(n + 1)} \qquad i = 1, 2, 3, \ldots, n \tag{69}$$

where $F(t_i)$ is the estimated cumulative probability of the "i^{th}" smallest duration and n is the number of extreme events. Durations are first ordered from smallest to largest for this purpose and the corresponding cumulative probability computed. Other plotting formulae were considered (e.g., Gringorten 1963), but this more commonly used approach is preferable for general application since no additional parameters need be estimated.

Two continuous distributions are considered as models for the cumulative probability of durations because of their common application to peak wave height conditions: the Extremal (Fisher-Tippett) Type I and the Weibull distributions. An existing FORTRAN program (US Army Engineer Waterways Experiment Station 1985), originally designed to fit these distributions to wave height data by the method of least squares, applying the plotting formula convention described above, was adapted to work instead with duration data derived by the extreme event identification program. The extreme event identification program was ultimately combined with the program-estimating distribution parameters and titled STRMDIST, a listing of which is presented in Appendix C. The program STRMDIST, in addition to the extreme event indentification and duration derivation computations already described, computes distribution parameters (ε and β for the Extremal Type I and α and β for the Weibull), estimated (distribution) mean and standard deviation, correlation coefficient, sum of the square residuals, and standard error. These parameters also are computed for peak wave heights of extreme events identified. Tables A11-A15 give results of the STRMDIST analysis for five Phase III sites.

Discussion of the Distribution Analysis

Figures 11 and 12 demonstrate fit of the least squares regression distribution to the data as represented by the plotting formula for one case each

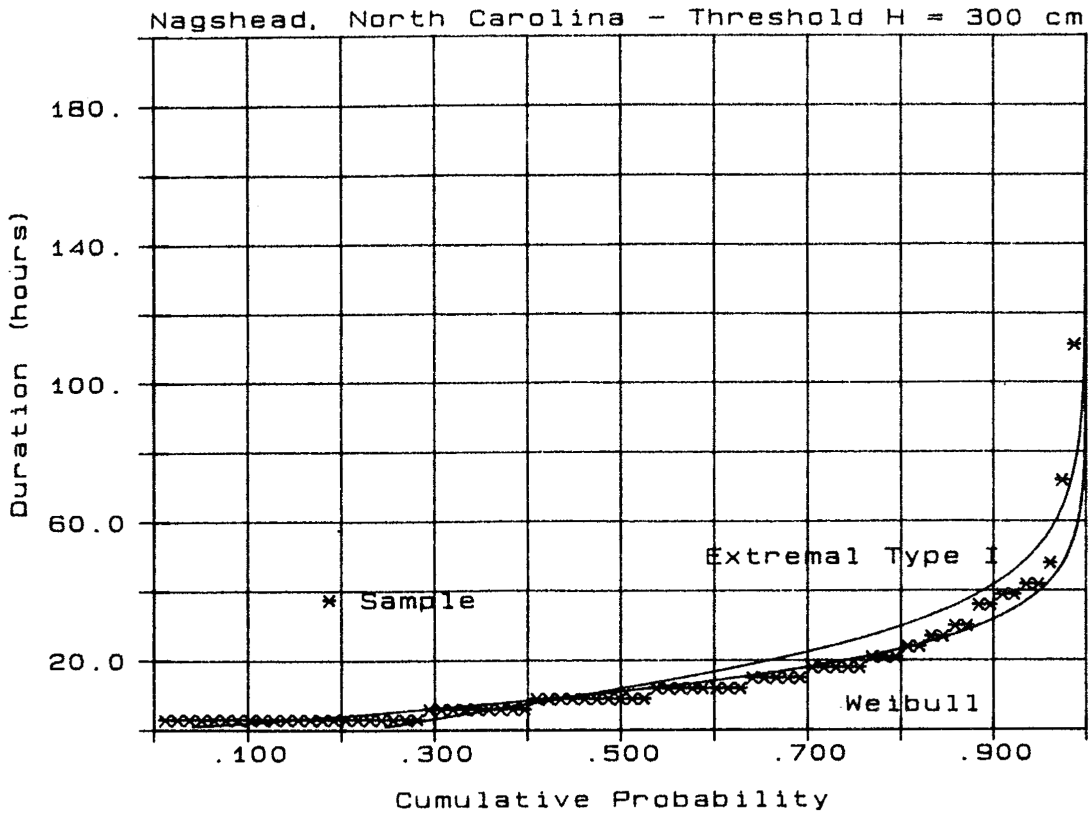

Figure 11. Duration cumulative probability:
Nagshead, North Carolina

at Nagshead, North Carolina, and Newport, Oregon. The Weibull distribution in both these cases can be seen to generally fit the overall data spread better, but the Extremal Type I comes closer to the few most extreme durations. The correlation coefficients, sums of square residuals, and standard errors in Tables A11-A15 indicate that the Weibull distribution generally fits the data better than the Extremal Type I, but both distributions fit it acceptably well in practical terms. Correlation coefficients above 0.90 would provide a rule-of-thumb acceptable fit in exercises of this type with weather-related data. Both distributions generally exceed this criterion.

Figure 13 shows correlation coefficients for both distributions plotted against percent occurrence of wave heights above the specified threshold, H1 , for Nagshead. Figure A16 shows a similar plot for Newport.

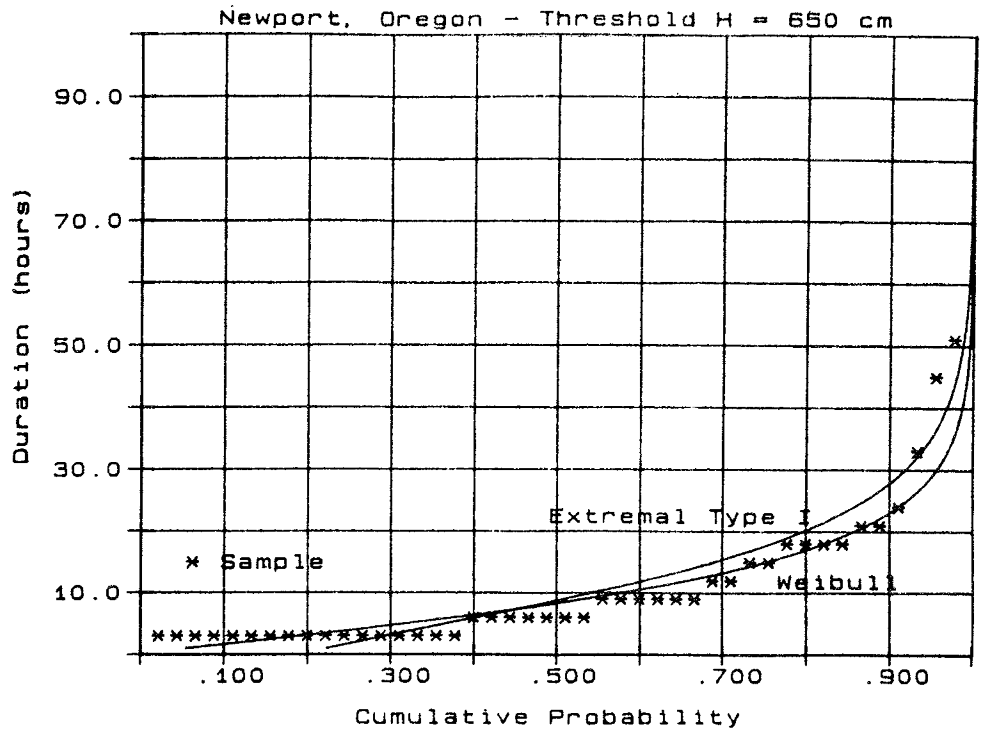

Figure 12. Duration cumulative probability: Newport, Oregon

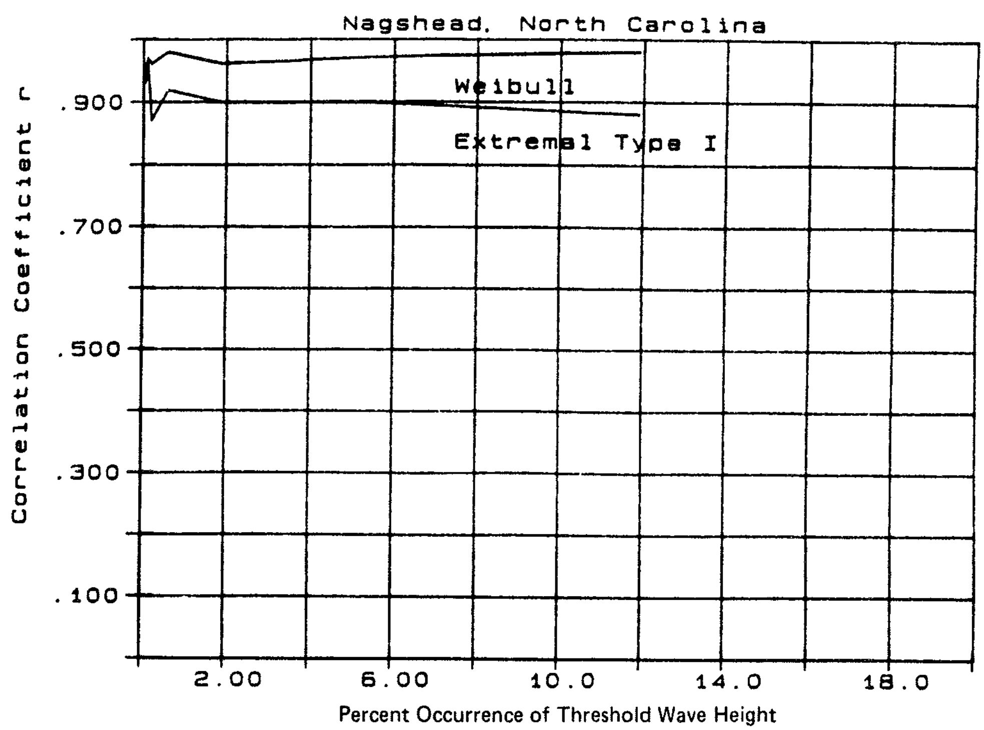

Figure 13. Correlation coefficient versus percent occurrence of wave height threshold, Nagshead, North Carolina

Figures A17 and A18 show the correlation coefficients for both distributions at Nagshead and Newport plotted against the number of extreme events per year. No obvious maximum occurs which could reliably be taken as an indication of an optimal choice for either λ or H1 .

Figures A19 and A21 are plots of the standard error against extreme events per year and percent occurrence for Nagshead, North Carolina. Figures A20 and A22 show the same information for Newport, Oregon. Again, no obvious minimum generally occurs to indicate an optimal choice for λ or H1 .

Figures 14 and 15 are graphs of the sample and distribution means and sample and distribution standard deviations plotted against percent occurrence and the number of extreme events per year both for Nagshead. Figures A23 and A24 are similar graphs for Newport. The Extremal Type I distribution mean and standard deviation can be seen to generally come closer to the sample mean and standard deviation. This is desirable, particularly in the case of the mean. The Central Limit Theorem states that sample means from an infinite population can be considered as random variables with a mean equal to the population mean. The standard deviation, as a measure of the spread of duration values about the mean, is an important indicator of how conservative a parameterized distribution might be. The Extremal Type I distribution can be seen to be closer to and consistently larger than (i.e. on the conservative side of) the sample standard deviation. The Weibull distribution standard deviation is both farther from the sample standard deviation and generally lower, i.e., predicting more central tendency than the sample. The Extremal Type I distribution in these respects appears superior to the Weibull distribution.

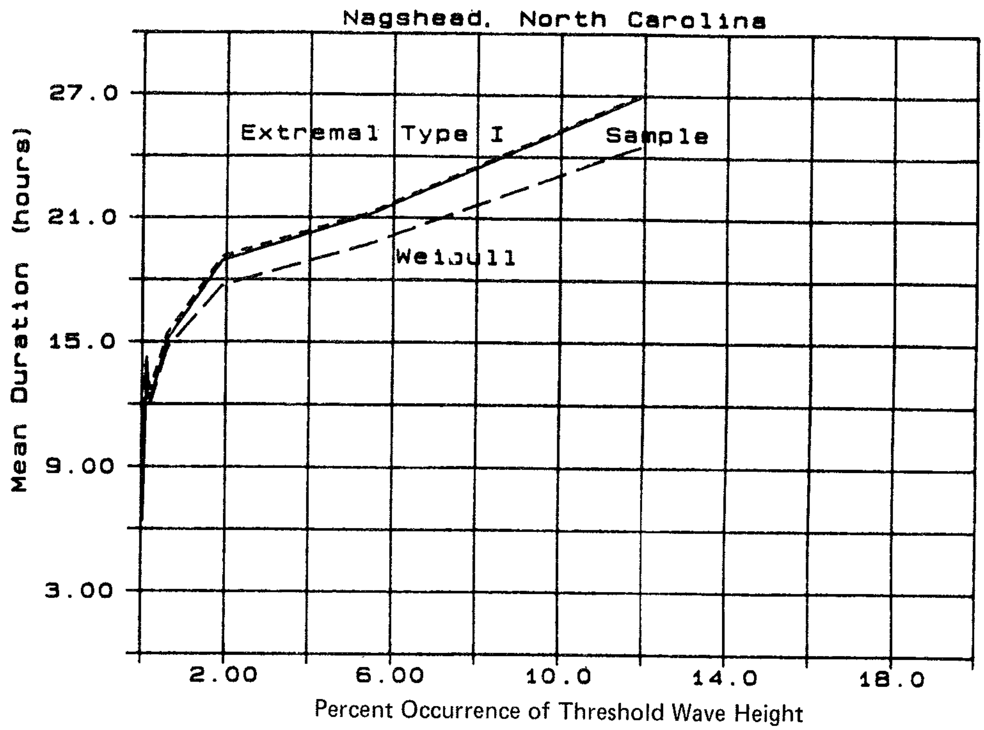

Figure 14. Mean duration versus percent occurrence of wave height threshold, Nagshead, North Carolina

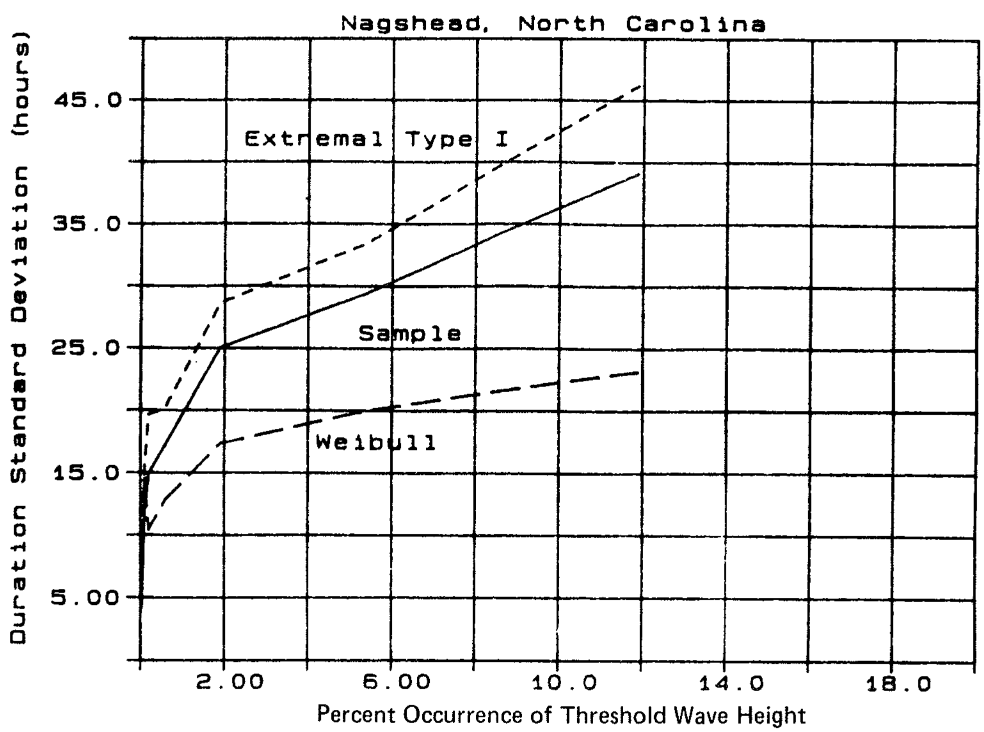

Figure 15. Duration standard deviation versus percent occurrence of wave height threshold, Nagshead, North Carolina

CHAPTER VIII: RELATIONSHIP OF DURATION TO PEAK CONDITIONS

Method of Analysis

The potential linear or nonlinear relationship of an extreme event's duration with peak conditions of the extreme event were investigated with the aid of statistical software package SPSS (Nie et al. 1975). The stepwise multiple regression capabilities of SPSS were of particular value in testing whether extreme event duration appeared to be dependent on peak conditions, as measured by various parameters such as H, T, H^2, T^2, H/gT^2, H^2L, and direction. Simple linear regressions of extreme event durations, as derived by a range of thresholds, first were performed. In the same program execution, SPSS allowed a stepwise multiple regression of duration against H, H^2, T, and T^2 to be performed. This procedure estimated the incremental contribution of each of these potentially controlling (independent) variables to the data fit by the least squares method. An equation of the following form was thus possible, assuming the contribution of each of these tested parameters was significant:

$$t = aH + bH^2 + cT + dT^2 \tag{70}$$

where a, b, c, and d are constants.

The purpose in this exercise was not to derive a predictor equation, but to see if a significant relationship existed. Therefore, the obvious interdependence of H^2 with H and T^2 with T was not of undesirable consequence. One common technique to test for existence of a nonlinear relationship, versus a linear relationship, is also to test the square of the variable on a trial basis. A substantially improved fit with the square of the parameter included in the regression equation generally indicates that a nonlinear relationship, whether polynomial or otherwise, is more reliable than a simple linear relationship. The correlation coefficient, r, as applied above in the fit of distribution functions, was taken as the primary measure of the strength of a relationship in this analysis.

Tables A16-A20 show results from execution of SPSS for all cases tested for each of the five Phase III sites. A listing of the SPSS command file used to perform each of these executions is presented in Appendix D along with a sample output. The tables give correlation coefficients for duration against H, H^2, T, and T^2 (individual simple linear regressions), against all four of these parameters in a stepwise procedure and against H^2L (simple linear regression). There is little indication in any case of a linear relationship of duration with H/gT^2 with correlation coefficients for this parameter consistently near zero. Similarly, correlation coefficients of duration with predominant direction of wave propagation at the peaks of extreme events were consistently near zero.

Discussion of Results of the Regression Analysis

The parameter H, the peak zero moment wave height, is consistently the most significant parameter, which confirms that an extreme event identification and duration definition procedure using this parameter is best. Another notable trend indicated by the above results is the observation that correlation coefficients for the Pacific sites are consistently lower than those for Atlantic sites. A possible explanation for this is the fact that Pacific storms typically form well away from the coast and travel onshore. They tend to be well formed when their effects first become significant and their tracks are more or less in the same direction (eastward to some degree). Atlantic (extratropical) storms can form onshore and travel seaward, travel longshore, or linger in one spot, as exemplified by the previous account of conditions in late October 1956. This more variable track (particularly the potential for a roughly stationary storm) may cause the duration above a specified threshold in many cases to be more dependent on the time-history of the storm's internal intensity than its track past a fixed site.

There was no strong correlation of duration (applying the rule-of-thumb criterion of 0.90) with any of the variables on either coast. The regression slopes, i.e. the β parameter in Equation 40, also were consistently small numbers, much closer to zero than to one. The low slopes, even for H, indicate that dependence of durations on peak wave conditions is weak. A fully rigorous proof of dependence or otherwise would require many more tests and computations than those presented here. The lack of an obvious strong dependence, however, raises the suggestion that, for practical purposes, extreme event duration might be taken as independent of peak conditions of the extreme event. This would make estimates of joint probability, for example forecast of durations of wave heights above a threshold for a rare event (e.g., the 50- or 100-year extreme event), relatively easy to compute. An example of how such an estimate might be made follows:

Example Computation of Peak Wave Height and Duration Joint Probability

Problem: What is the joint probability of zero moment wave heights greater than 3.0 m lasting longer than 12 hr during an extreme event whose peak zero moment wave height is greater than 4.5 m at Nagshead, North Carolina?

Solution: The definition of duration at H1 allows the associated parameters presented in Table A12 to be applied. Choosing the Extremal Type I distribution to represent both marginal distribution of peak wave heights and marginal distribution of durations: $\varepsilon_t = 6.30$, $\beta_t = 15.8$, $\varepsilon_H = 326.3$, and $\beta_H = 48.0$. The Poisson parameter, λ , from Table A5, is 3.8. The marginal probabilities of exceedance are:

$$P(t' > t) = 1 - F(t) = 1 - \exp\left\{-\exp\left[-\frac{(t - \varepsilon_t)}{\beta_t}\right]\right\}$$

$$= 1 - \exp\left\{-\exp\left[-\frac{(12 - 6.30)}{15.8}\right]\right\}$$

$$= 0.502$$

$$P(H' > H) = 1 - F(H) = 1 - \exp\left\{-\exp\left[-\frac{(H - \varepsilon_H)}{\beta_H}\right]\right\}$$

$$= 1 - \exp\left\{-\exp\left[-\frac{(450 - 326.3)}{48.0}\right]\right\}$$

$$= 0.073$$

The joint probability, taken as the product of independent marginal probabilities defined from the same population (H1 = 300), is:

$$P(t' > 12 \, , H' > 450 \mid H1 = 300) = 0.502(0.073) = 0.037$$

The associated return period is:

$$RT(t,H) = \frac{1}{\{\lambda[1 - F(t,H)]\}} = \frac{1}{[3.8(0.037)]} = 7.0 \text{ years}$$

The associated nonencounter probability in a 50-year time period is:

$$NE(t,H) = \exp\left[\frac{-L}{RT(t,H)}\right] = \exp\left(\frac{-50}{7.0}\right) = 0.00079 = 0.08\%$$

The associated risk of encountering such a condition in a 50-year time span is $1 - NE(t,H) = 0.921 = 92.1\%$.

Discussion: Given the assumptions stated above, the probability of exceedance of a peak wave height of 4.5 m of any duration is 7.3 percent. The condition of duration exceeding 12 hr eliminates about half of the possibilities; therefore, the joint probability is about half as much. The joint return period is also correspondingly longer. The Poisson assumption inherent in definition of return period and nonencounter probability can be extended to the joint peak wave height and duration distribution if waiting periods between extreme events are much greater than durations of the extreme events. The Poisson distribution is a discrete distribution, and its application technically extends only to discrete events.

CHAPTER IX: CONCLUSIONS

Literature Review

A review of scientific and engineering literature related to duration of sea states reveals little direct work in this area. The work of Houmb and Vik (1977) is most pertinent to objectives of this study. These investigators worked with several years of intermittently measured wave information at five points along the North Sea coast of Norway. They found the duration of extreme sea states, as defined by the exceedance of a wave height threshold, to fit a Weibull distribution. They approached the problem as much as possible from a theoretical perspective in order to maximize the reliability of observations based on limited data.

Identification of Extreme Events

This study applies the Phase III (shallow water) Wave Information Studies (WIS) database of hindcast wave data because of its unusually long, continuous 20-year period of record and because of its synoptic (ocean wide) perspective on wave conditions. The WIS numerical simulations involve some practical simplifications, but no database of measured wave information is available which could be used to investigate such a long period of record over a wide geographical area. Data from five Phase III stations are applied in this study to investigate duration of extreme wave conditions. Three are on the Atlantic coast (from New Jersey to central Florida) and two are on the Pacific coast (Oregon to central California).

The conventional parameter for long-term wave statistics, zero moment wave height, is chosen as the most practical and reliable indicator of intensity of wave conditions. A computer program is presented which reviews Phase III information and records the number of sequential records (each 3 hr apart) in which the geometric average (combined) sea and swell wave height is above a specific threshold. A single record below the threshold between two that were above is ignored, i.e., the two records above are treated as part of a single event. The percent occurrence of waves above a threshold is found to vary linearly with the number of extreme events identified, regardless of absolute intensity of wave climate on either coast.

Distribution of Durations

The Weibull and Extremal Type I distributions are fit by the method of least squares to durations of extreme events identified and to peak wave heights. Both distributions show acceptable correlation to the wave data, but the Extremal Type I is found to provide superior estimates of both durations and peak wave heights.

Relationship of Duration to Peak Intensity

A multilinear regression analysis is performed to address the potential relationship of extreme event duration to peak conditions of the extreme event. Peak intensity, as measured by the zero moment wave height, has only a weak linear relationship to duration. Other alternate parameters of intensity show little evidence of significant linear relation to duration. The investigation does not rigorously prove statistical independence, but the assumption of independence of duration from peak intensity is proposed as an expedient measure. This assumption greatly simplifies prediction of durations of wave conditions above a critical threshold.

CHAPTER X: REFERENCES

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APPENDIX A

ADDITIONAL FIGURES AND TABLES

Figure No.		Page

Table No.		Page

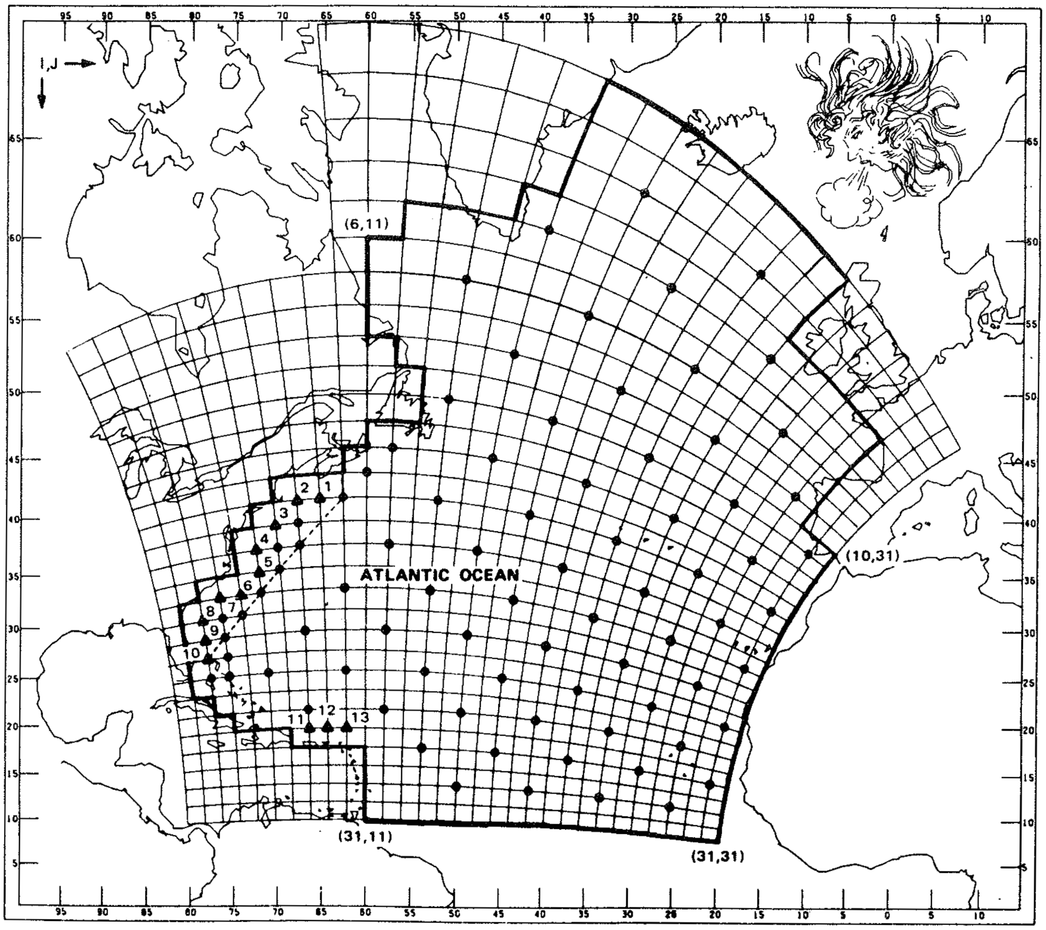
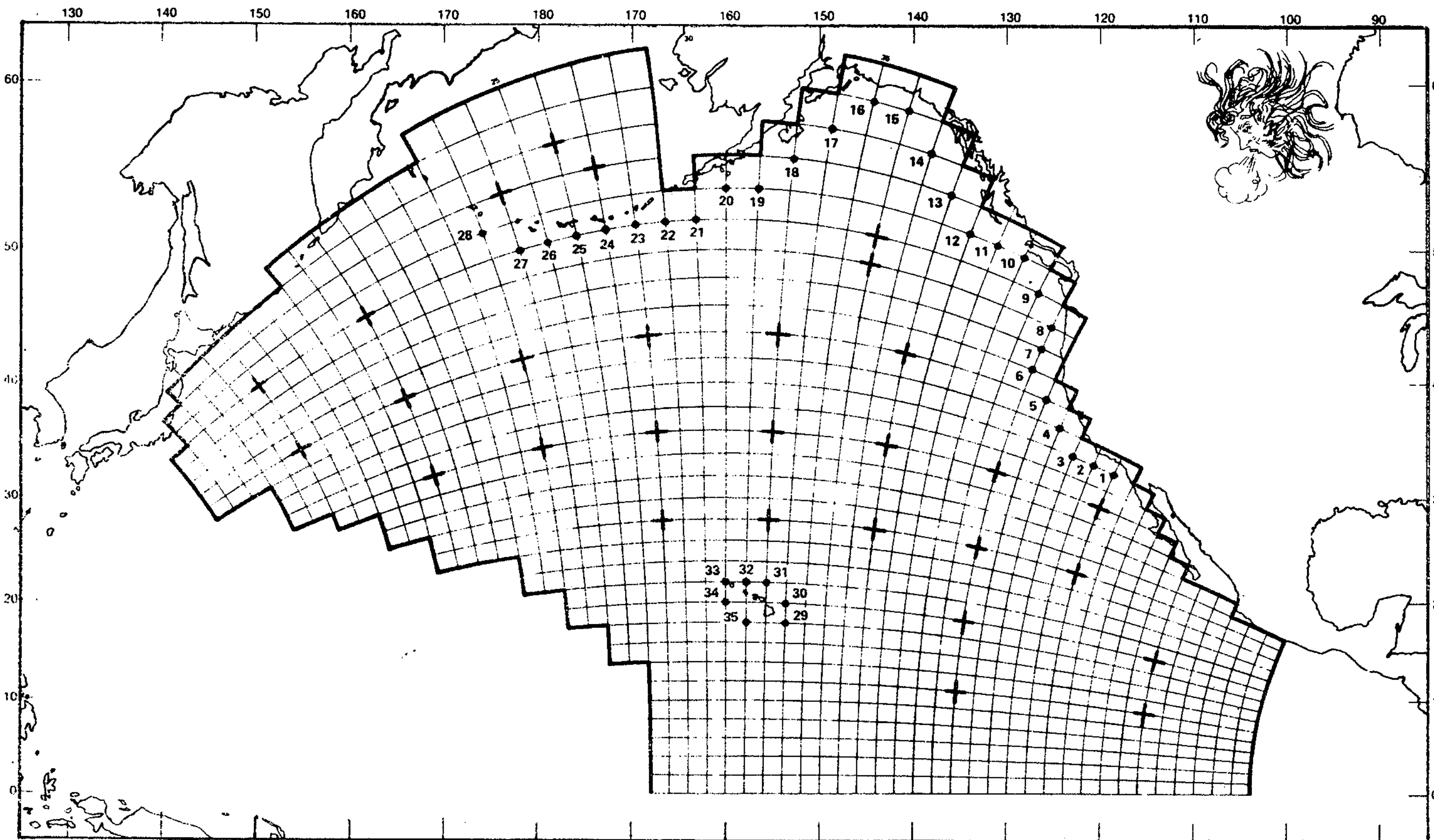


Figure A1. WIS Phase I grid, North Atlantic Ocean



MERCATOR PROJECTION OF PACIFIC OCEAN

Figure A2. WIS Phase I grid, North Pacific Ocean

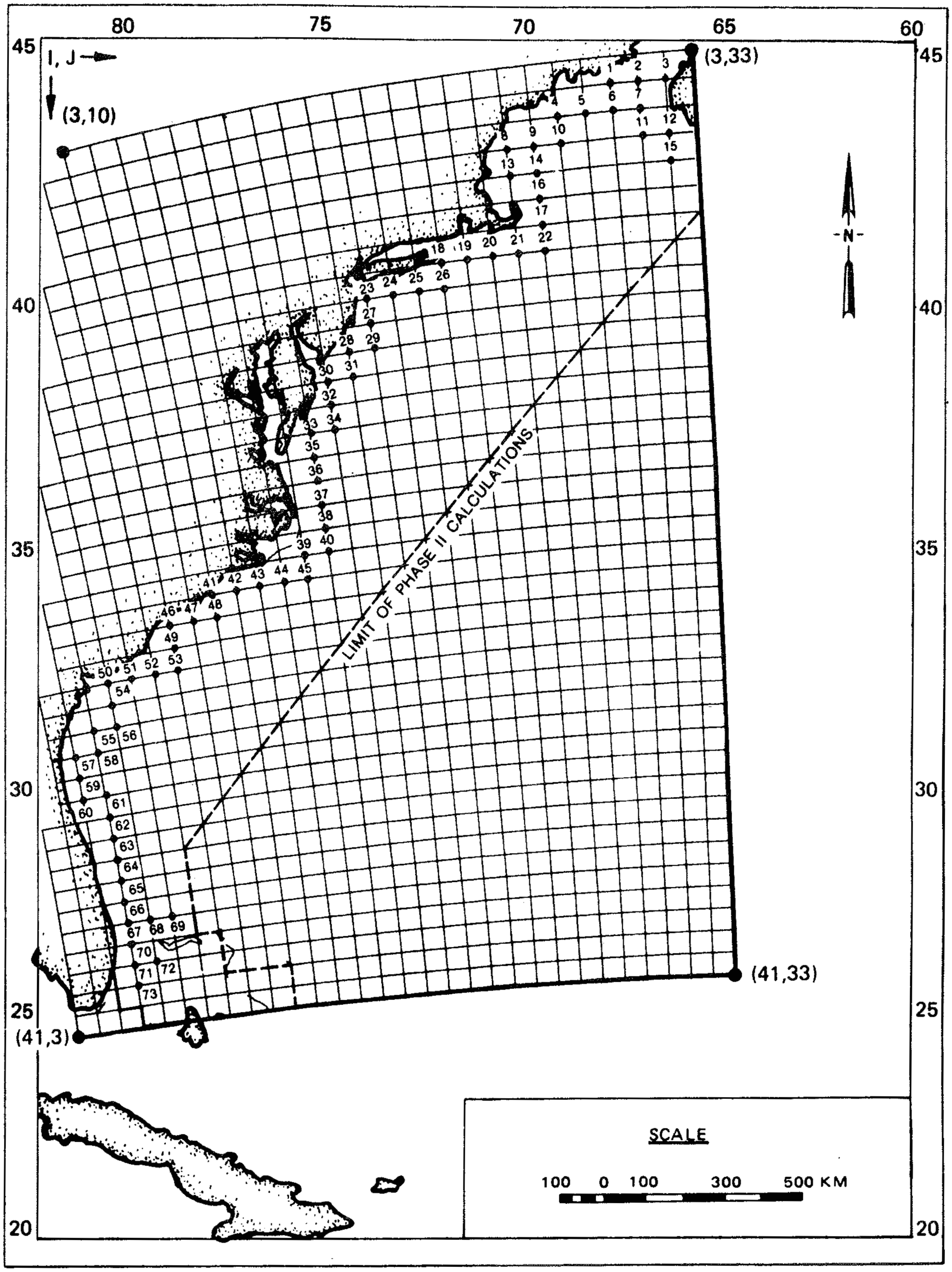


Figure A3. WIS Phase II grid, Atlantic coast

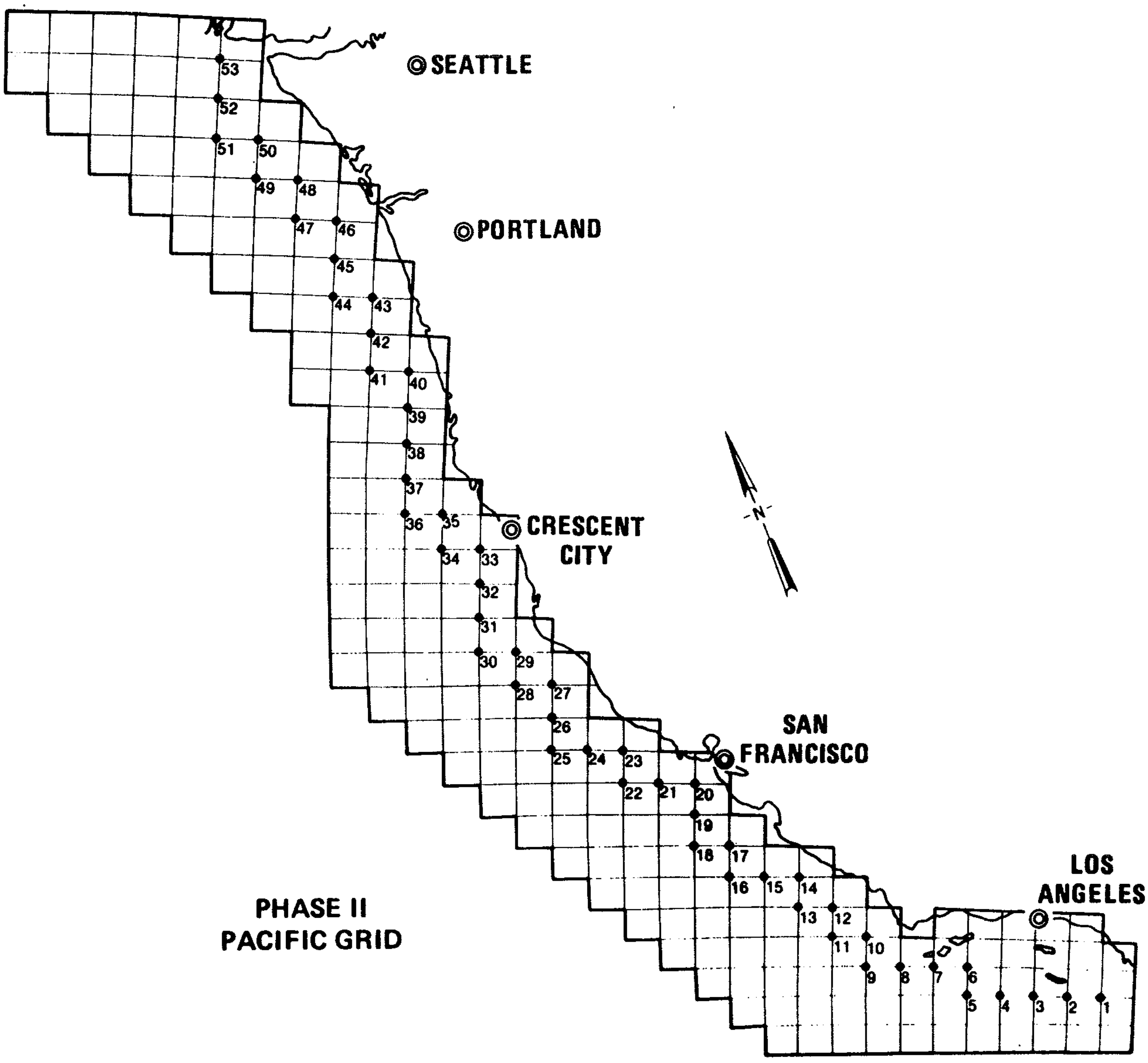


Figure A4. WIS Phase II grid, Pacific coast

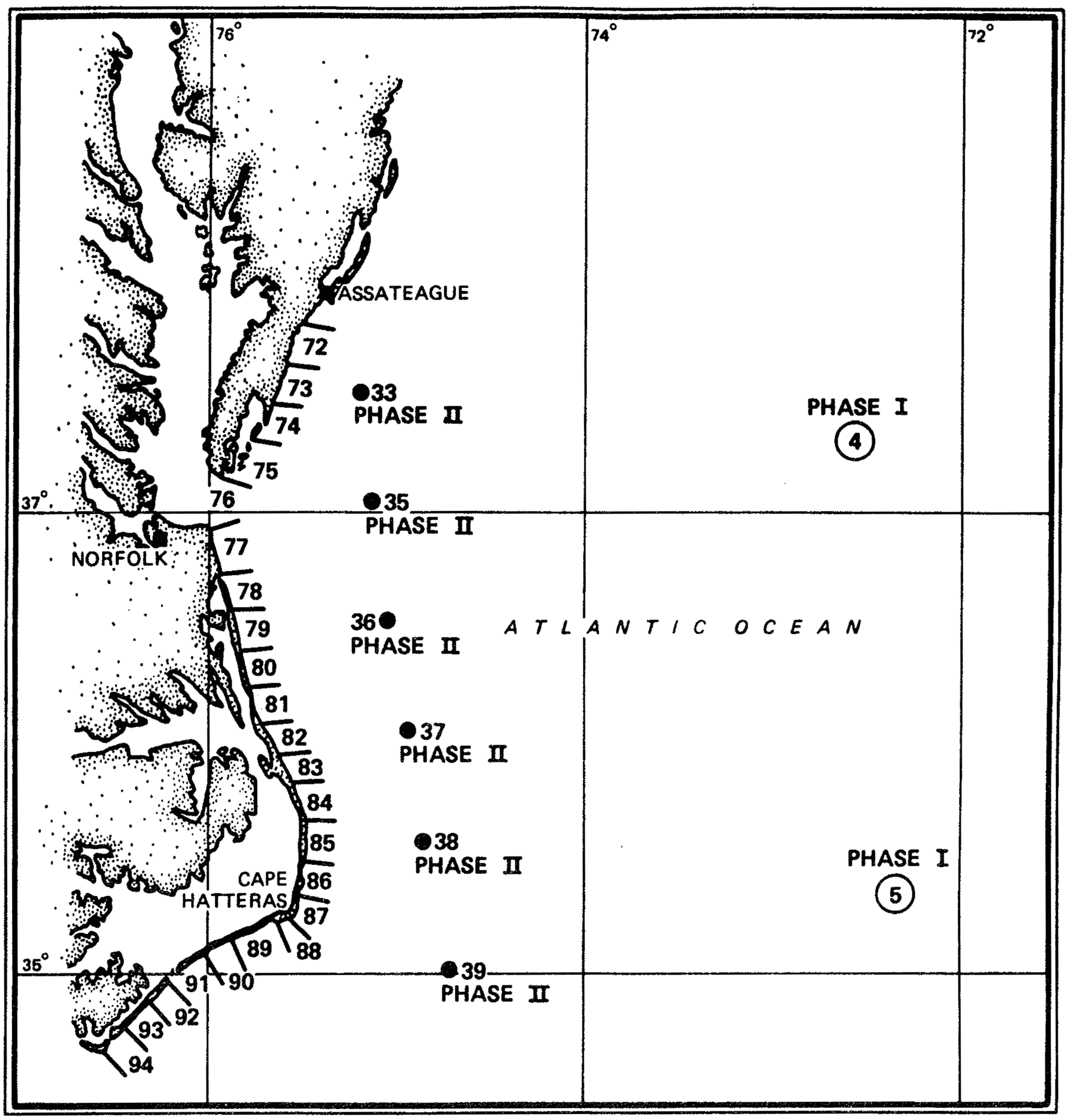


Figure A5. Mid-Atlantic coast portion, WIS Phase III stations

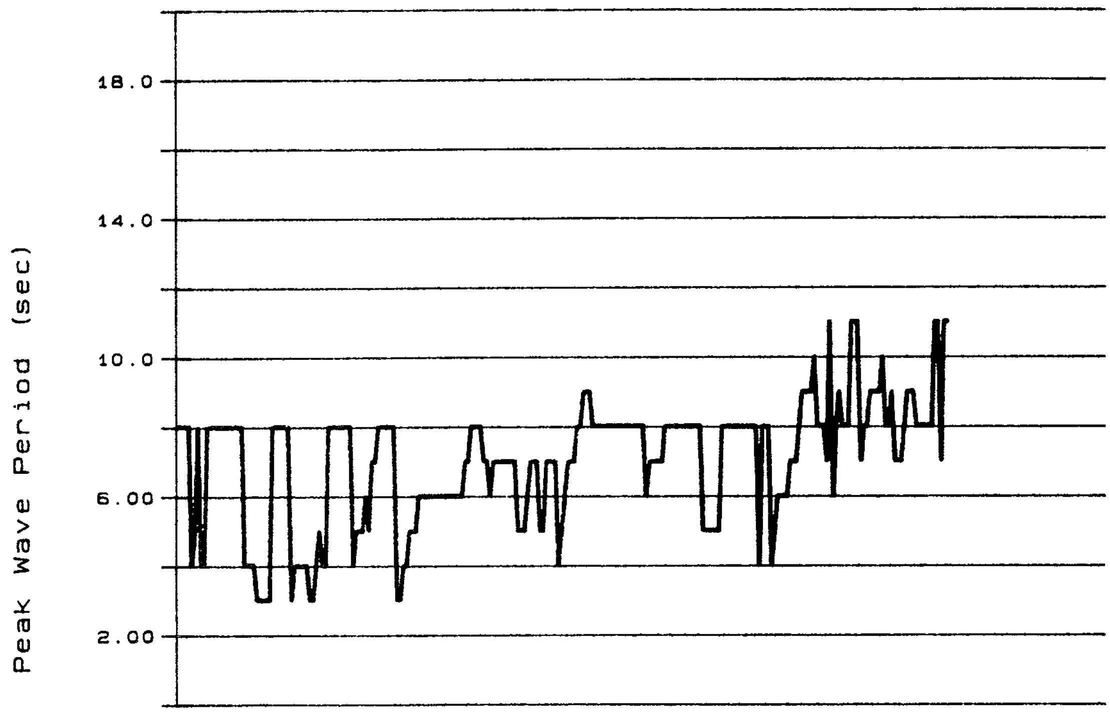


Figure A6. Wave period time series, Nagshead, NC, October 1956

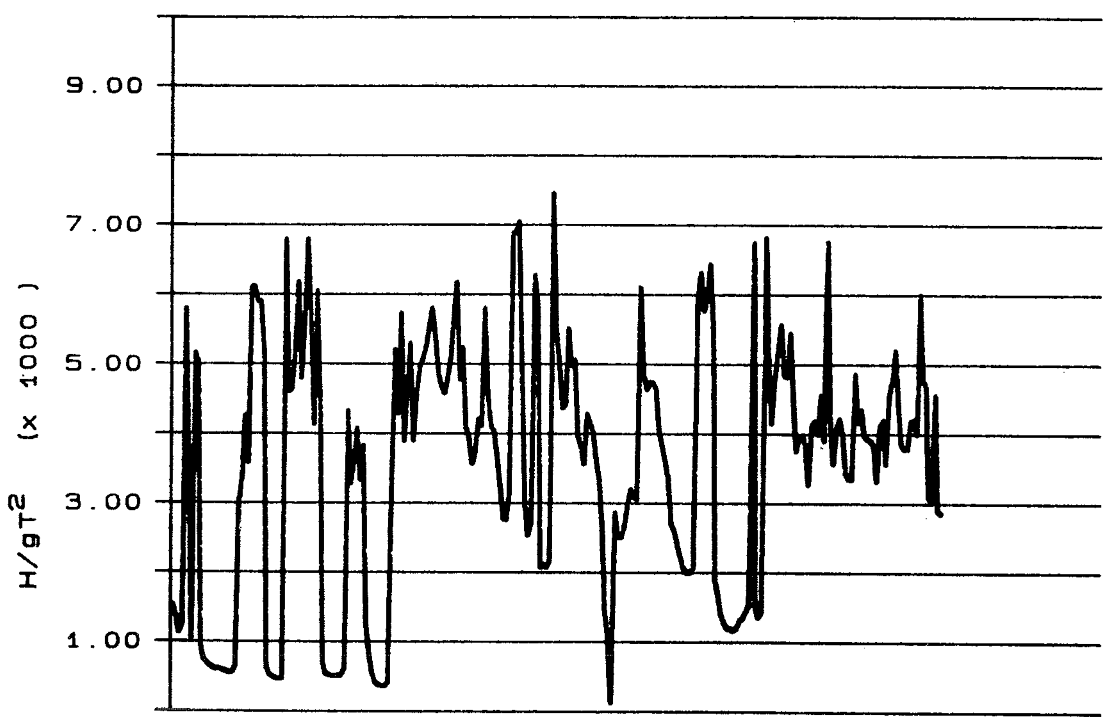


Figure A7. Wave steepness time series, Nagshead, NC, October 1956

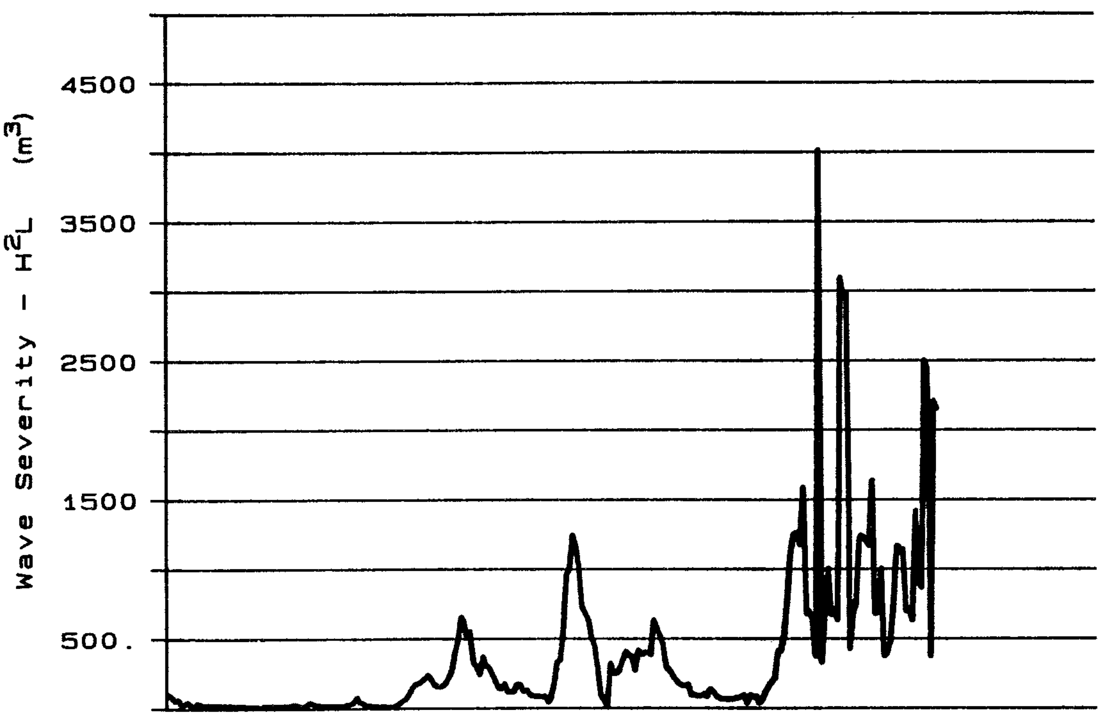


Figure A8. Wave severity time series, Nagshead, NC, October 1956, plotted as H^2L

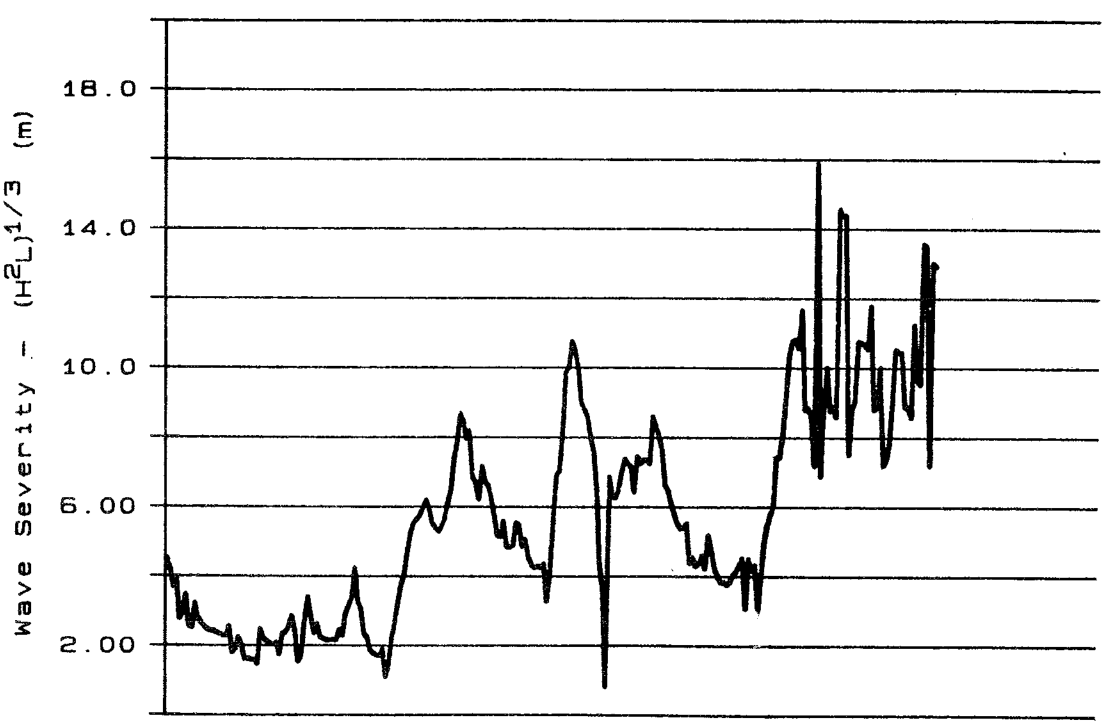


Figure A9. Wave severity time series, Nagshead, NC, October 1956, plotted as $(H^2L)^{1/3}$

Figure A10. Wave direction time series, Nagshead, NC, October 1956

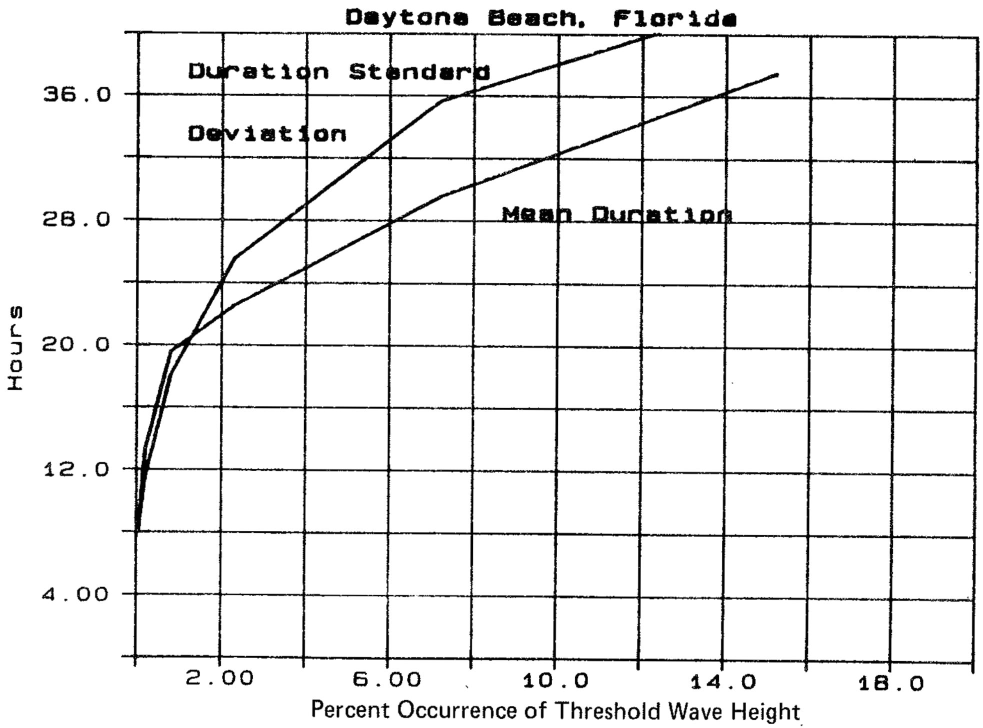


Figure A11. t and st versus percent occurrence, Daytona Beach, FL

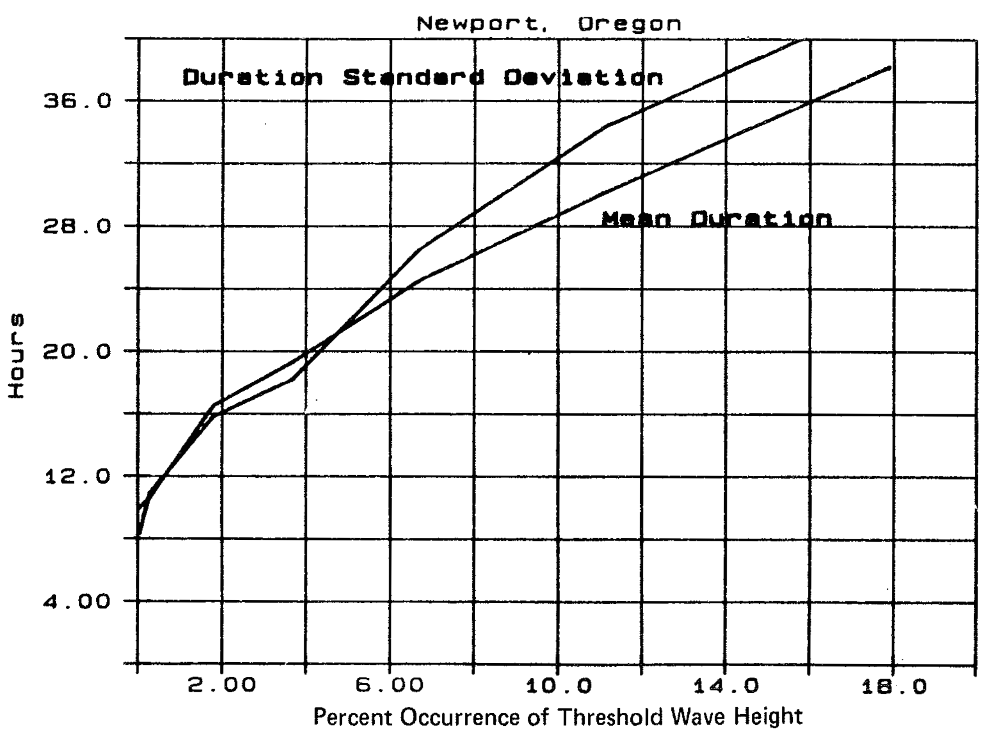


Figure A12. t and st versus percent occurrence, Newport, OR

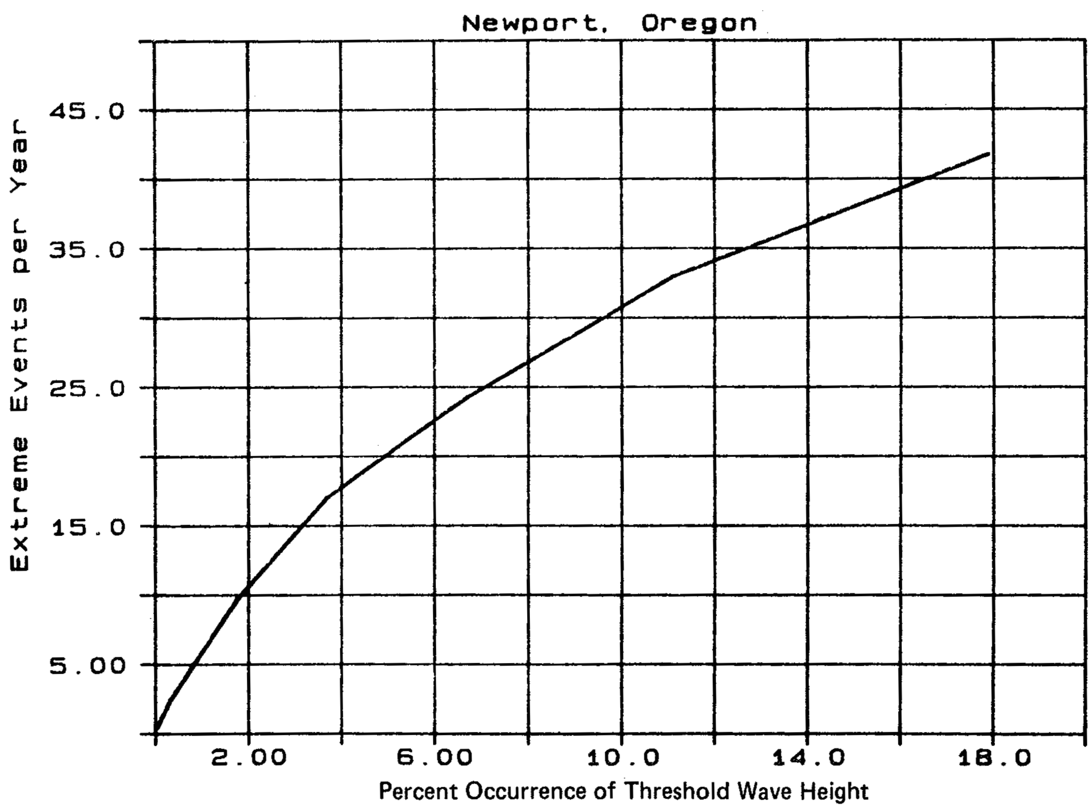


Figure A13. Extreme events per year versus percent occurrence, Newport, OR

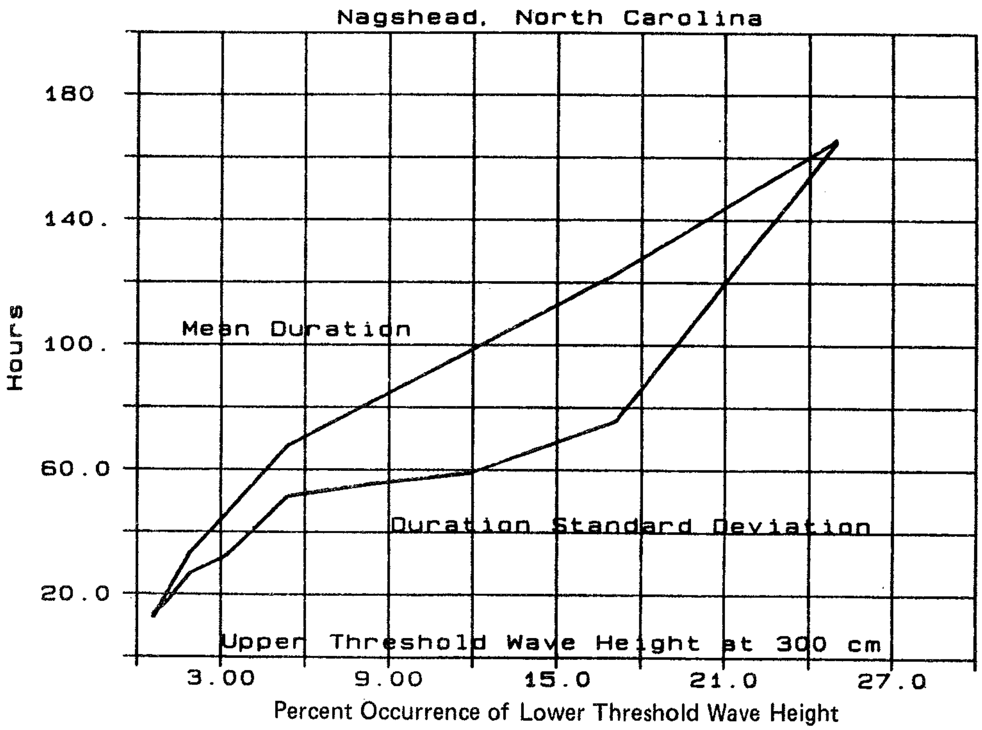


Figure A14. t and st versus percent occurrence with peak threshold, Nagshead, NC

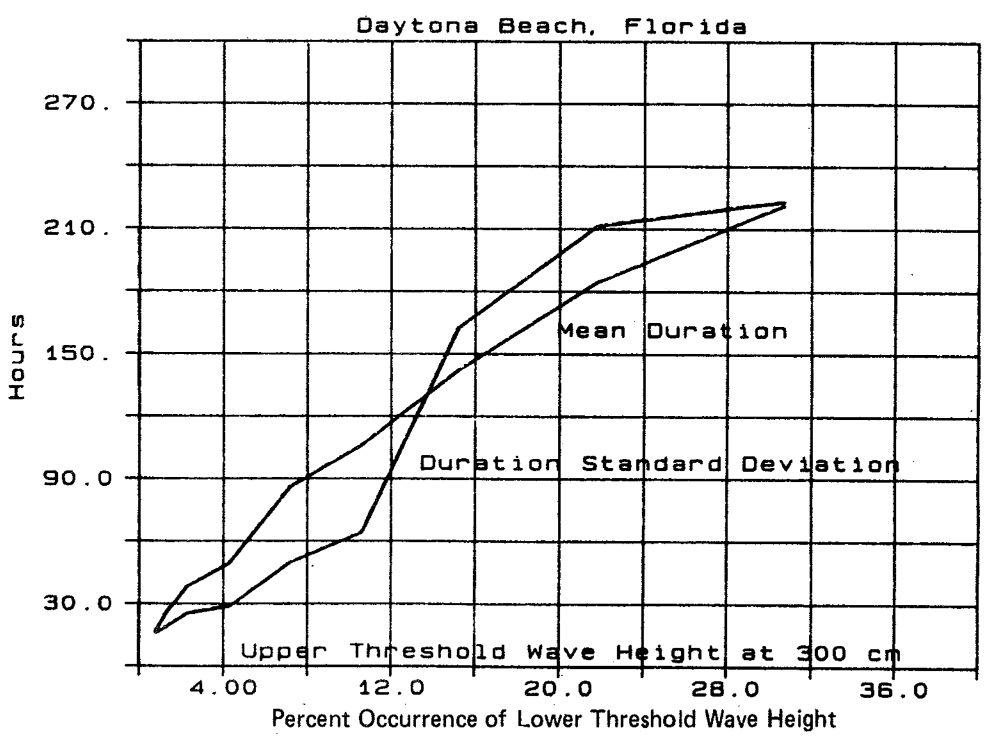


Figure A15. t and st versus percent occurrence with peak threshold, Daytona Beach, FL

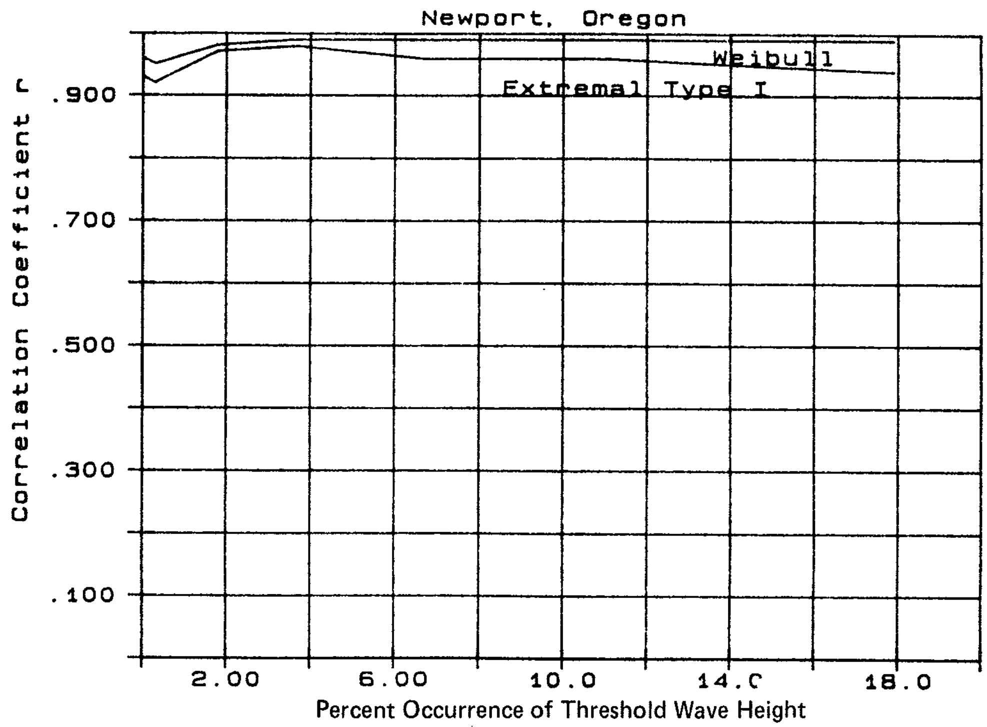


Figure A16. Correlation coefficient versus percent occurrence, Newport, OR

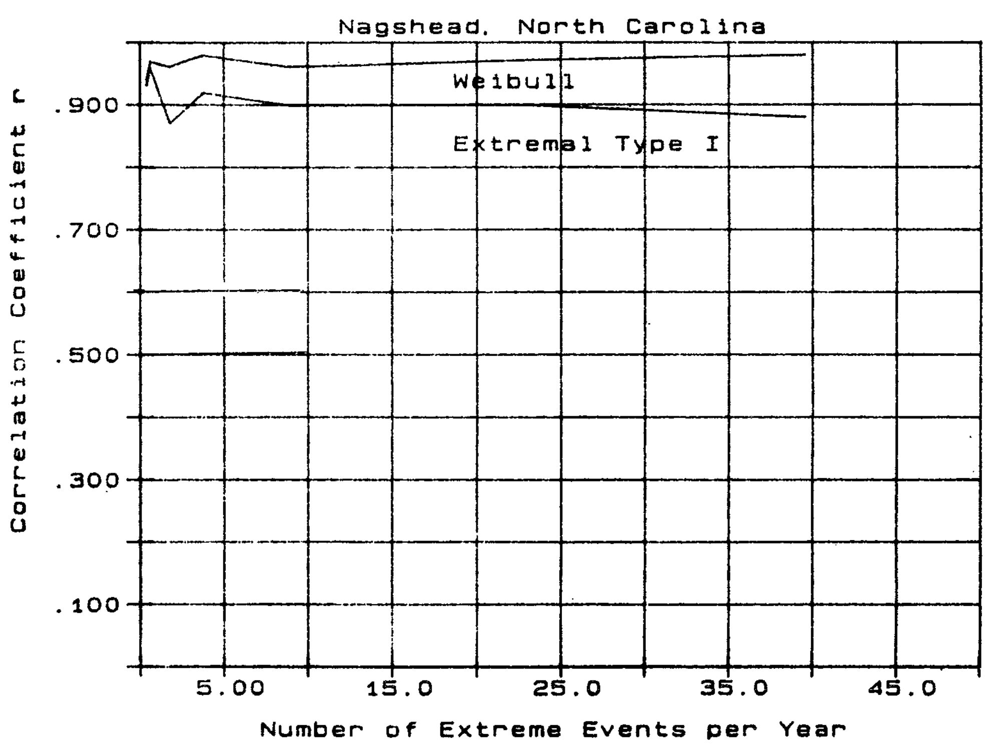


Figure A17. Correlation coefficient versus extreme events per year, Nagshead, NC

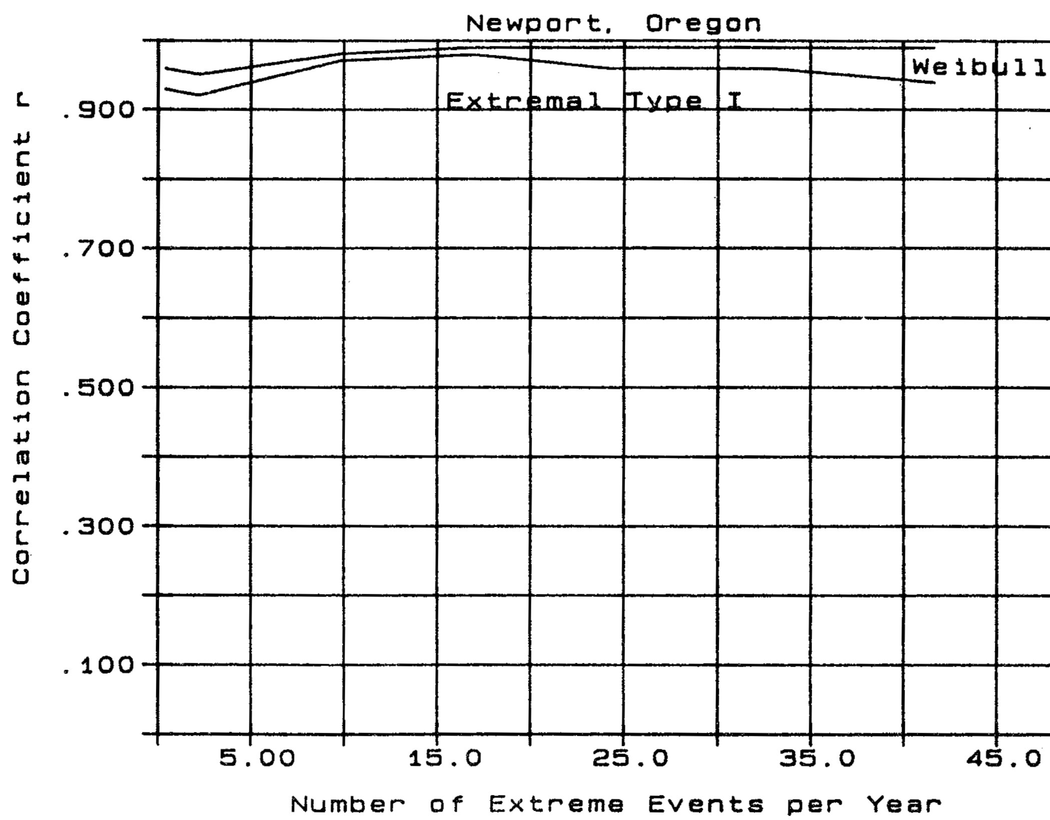


Figure A18. Correlation coefficient versus extreme events per year, Newport, OR

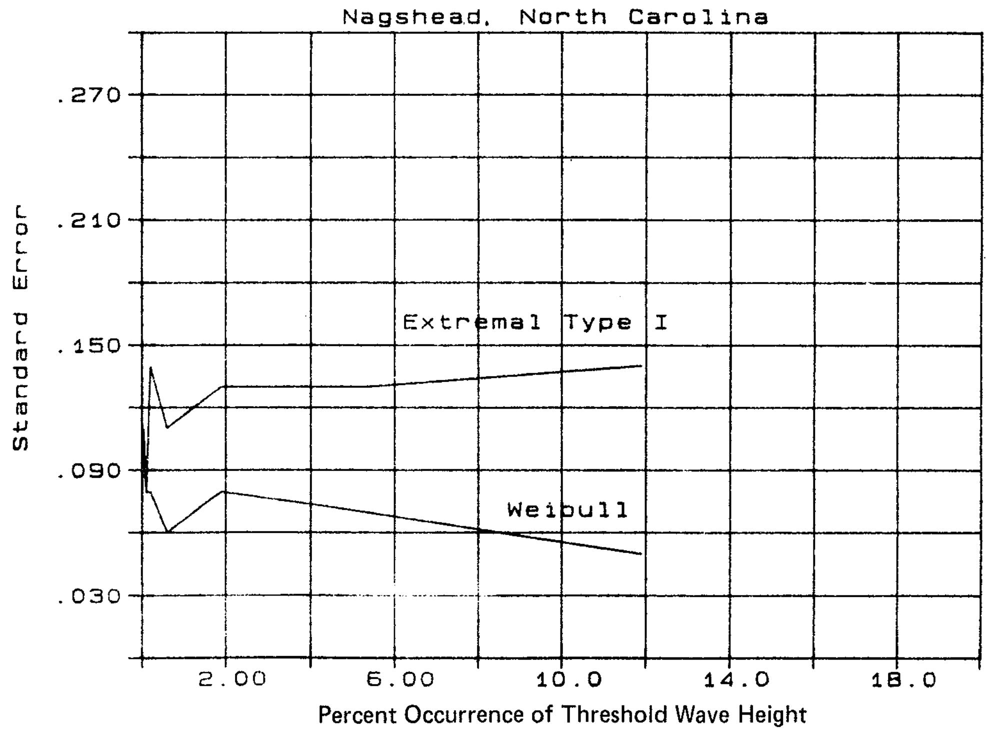


Figure A19. Standard error versus percent occurrence, Nagshead, NC

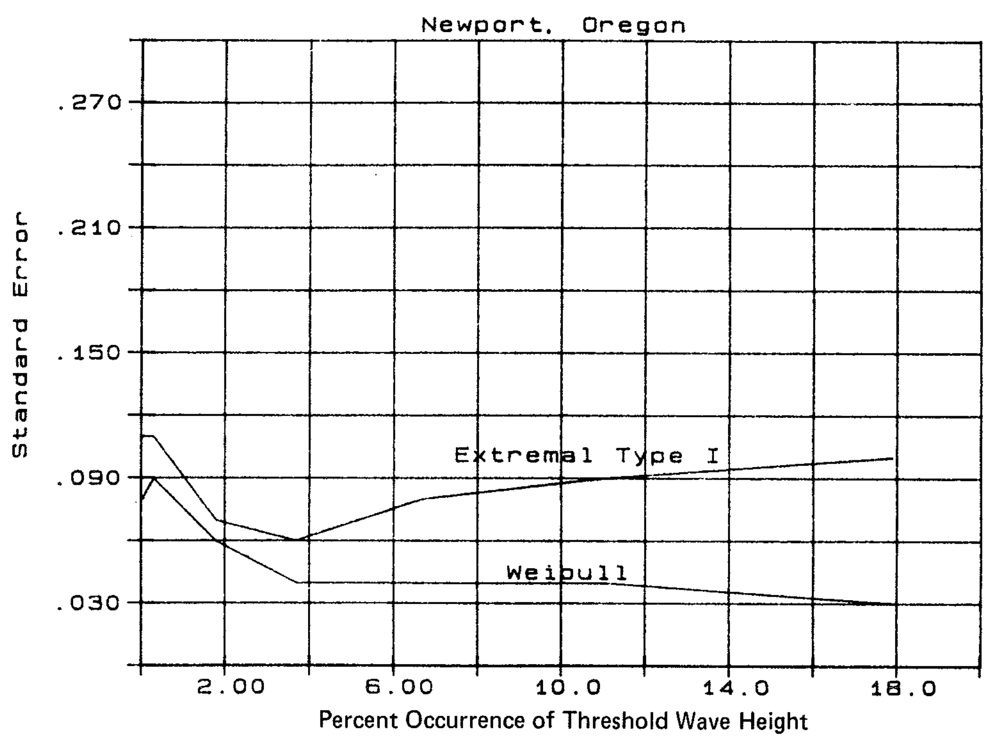


Figure A20. Standard error versus percent occurrence, Newport, OR

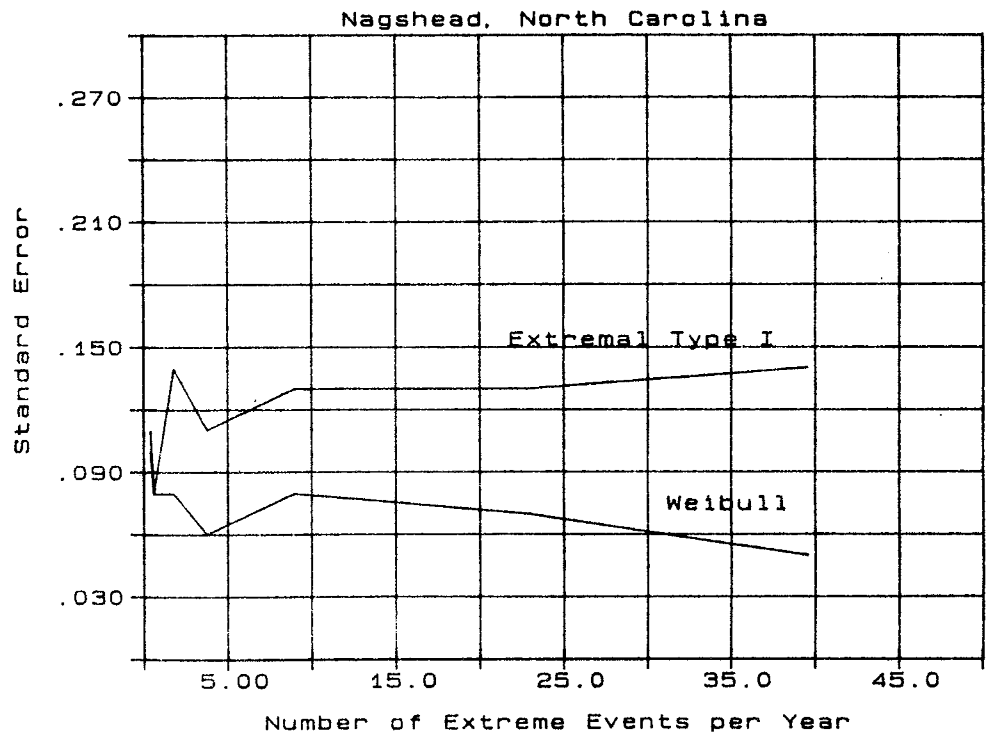


Figure A21. Standard error versus extreme events per year, Nagshead, NC

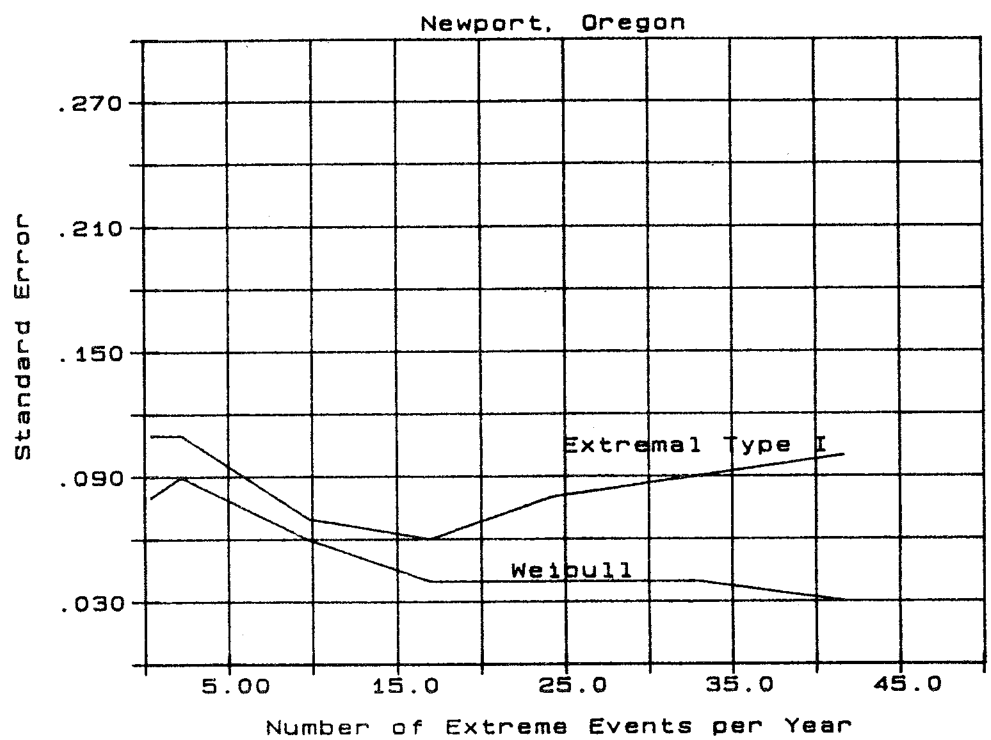


Figure A22. Standard error versus extreme events per year, Newport, OR

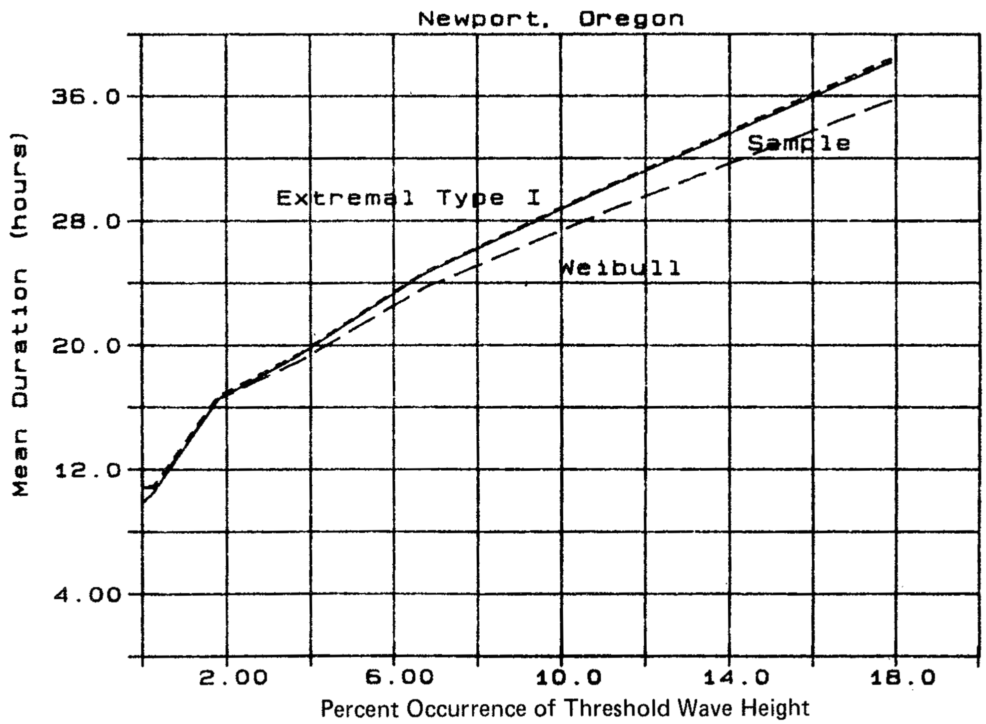


Figure A23. Mean duration versus percent occurrence, Newport, OR

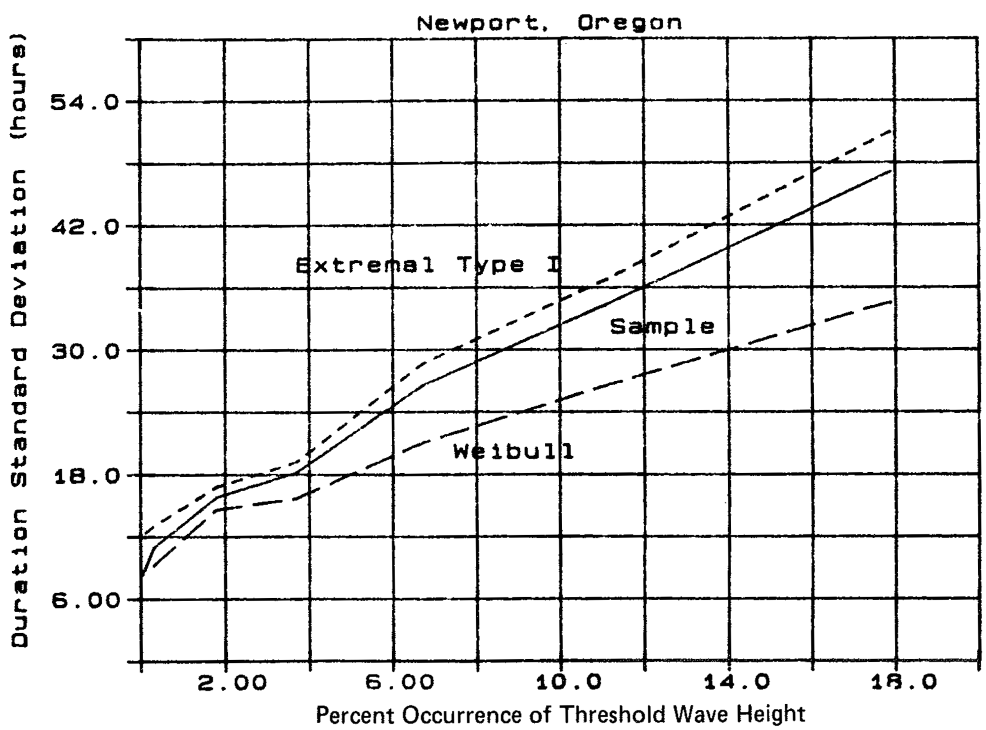


Figure A24. Duration standard deviation versus percent occurrence, Newport, OR

Table A1

October 1956 Phase III Data, Nagshead, NC

Station: Date YY/MM/DD	A3083 Hour	------Sea Readings------ Height (cm)	Period (secs)	Direct (azim)	-----Swell Readings----- Height (cm)	Period (secs)	Direct (azim)	--------Combined-------- Height (cm)	Period (secs)	Direct (azim)
56/10/27	00:00	219	7	90	0	0	0	219	7	90
56/10/27	03:00	137	6	103	441	11	83	462	11	83
56/10/27	06:00	208	6	98	119	11	83	240	6	98
56/10/27	09:00	261	8	90	0	0	0	261	8	90
56/10/27	12:00	282	9	93	0	0	0	282	9	93
56/10/27	15:00	258	8	98	0	0	0	258	8	98
56/10/27	18:00	265	8	101	0	0	0	265	8	101
56/10/27	21:00	250	8	100	0	0	0	250	8	100
56/10/28	00:00	93	5	130	395	11	75	406	11	75
56/10/28	03:00	117	5	130	378	11	75	396	11	75
56/10/28	06:00	149	6	127	369	11	75	398	11	75
56/10/28	09:00	234	7	111	0	0	0	234	7	111
56/10/28	12:00	260	8	108	0	0	0	260	8	108
56/10/28	15:00	273	8	107	0	0	0	273	8	107
56/10/28	18:00	314	9	99	0	0	0	314	9	99
56/10/28	21:00	311	9	99	0	0	0	311	9	99

Table A2

October 1956 Phase II Data, Nagshead, NC

Station: Date YY/MM/DD	A2037 Hour	------Sea Readings------ Height (cm)	Period (secs)	Direct (azim)	-----Swell Readings----- Height (cm)	Period (secs)	Direct (azim)	--------Combined-------- Height (cm)	Period (secs)	Direct (azim)
56/10/27	00:00	488	7	63	0	0	0	488	7	63
56/10/27	03:00	167	6	49	438	11	77	469	11	77
56/10/27	06:00	477	6	45	119	11	78	492	6	45
56/10/27	09:00	508	8	62	0	0	0	508	8	62
56/10/27	12:00	507	9	59	0	0	0	507	9	59
56/10/27	15:00	486	8	42	0	0	0	486	8	42
56/10/27	18:00	473	8	48	0	0	0	473	8	48
56/10/27	21:00	452	8	49	0	0	0	452	8	49
56/10/28	00:00	110	5	17	399	11	82	414	11	82
56/10/28	03:00	139	5	17	382	11	82	407	11	82
56/10/28	06:00	197	6	17	373	11	83	422	11	83
56/10/28	09:00	446	7	36	0	0	0	446	7	36
56/10/28	12:00	425	8	37	0	0	0	425	8	37
56/10/28	15:00	447	8	39	0	0	0	447	8	39
56/10/28	18:00	477	9	49	0	0	0	477	9	49
56/10/28	21:00	505	9	50	0	0	0	505	9	50

Table A3

October 1956 Phase I Deepwater Data, Offshore of Cape Natteras, NC

Station: Date YY/MM/DD	A1005 Hour	Sea Readings Height (cm)	Sea Period (secs)	Sea Direct (azim)	Swell Readings Height (cm)	Swell Period (secs)	Swell Direct (azim)	Combined Height (cm)	Combined Period (secs)	Combined Direct (azim)
56/10/26	00:00	515	8	39	0	0	0	515	8	39
56/10/26	03:00	571	9	40	0	0	0	571	9	40
56/10/26	06:00	624	9	42	0	0	0	624	9	42
56/10/26	09:00	646	9	45	0	0	0	646	9	45
56/10/26	12:00	660	9	47	0	0	0	660	9	47
56/10/26	15:00	648	9	49	0	0	0	648	9	49
56/10/26	18:00	620	8	50	0	0	0	620	8	50
56/10/26	21:00	611	9	53	0	0	0	611	9	53
56/10/27	00:00	598	9	56	0	0	0	598	9	56
56/10/27	03:00	554	8	57	0	0	0	554	8	57
56/10/27	06:00	518	7	62	0	0	0	518	7	62
56/10/27	09:00	489	6	64	0	0	0	489	6	64
56/10/27	12:00	464	6	65	0	0	0	464	6	65
56/10/27	15:00	443	6	66	0	0	0	443	6	66
56/10/27	18:00	228	5	69	360	12	157	426	12	157
56/10/27	21:00	415	7	69	0	0	0	415	7	69
56/10/28	00:00	431	8	70	0	0	0	431	8	70
56/10/28	03:00	425	7	69	0	0	0	425	7	69
56/10/28	06:00	416	6	69	0	0	0	416	6	69
56/10/28	09:00	421	6	69	0	0	0	421	6	69
56/10/28	12:00	423	6	69	0	0	0	423	6	69
56/10/28	15:00	420	65	70	0	0	0	420	6	70
56/10/28	18:00	414	6	69	0	0	0	414	6	69
56/10/28	21:00	446	8	70	0	0	0	446	8	70
56/10.29	00:00	549	9	68	0	0	0	549	9	68

Table A4

Duration Information for Atlantic City, NJ

H1 cm	H > H1 cm	H > H1 %	Number of Events	Number of Events/yr	t_{min} hrs	t_{max} hrs	$\bar{t}$ hrs	σ_t hrs
200	1442	2.5	323	16.2	3	54	12.8	10.9
250	384	0.7	112	5.6	3	30	9.9	7.3
300	81	0.1	29	1.4	3	24	8.1	6.8
350	18	0.03	9	0.4	3	15	5.7	4.1

Table A5

Duration Information for Nagshead, NC

H1 cm	H > H1 cm	H > H1 %	Number of Events	Number of Events/yr	t_{min} hrs	t_{max} hrs	$\bar{t}$ hrs	σ_t hrs
150	6983	11.9	792	39.6	3	570	26.9	39.1
200	3167	5.4	460	23.0	3	306	21.1	29.4
250	1093	1.9	179	9.0	3	165	18.9	25.0
300	374	0.6	77	3.8	3	111	15.1	17.2
350	143	0.2	36	1.8	3	84	12.2	14.9
400	56	0.10	13	0.6	3	42	12.9	12.0
450	16	0.03	8	0.4	3	15	6.4	4.1

Table A6

Duration Information for Daytona Beach, FL

H1 cm	H > H1 cm	H > H1 %	Number of Events	Number of Events/yr	t_{min} hrs	t_{max} hrs	$\bar{t}$ hrs	σ_t hrs
150	8855	15.2	716	35.8	3	1032	37.5	65.5
200	4183	7.2	432	21.6	3	303	29.5	35.6
250	1340	2.3	186	9.3	3	129	22.5	25.5
300	478	0.8	75	3.8	3	81	19.5	18.0
350	143	0.2	33	1.6	3	60	13.3	11.5
400	31	0.05	12	0.6	3	33	8.0	8.4
450	8	0.01	3	0.2	3	18	9.0	7.9

Table A7

Duration Information for Newport, OR

H1 cm	H > H1 cm	H > H1 %	Number of Events	Number of Events/yr	t_{min} hrs	t_{max} hrs	$\bar{t}$ hrs	σ_t hrs
400	10472	17.9	834	41.7	3	405	38.2	47.1
450	6494	11.1	658	32.9	3	279	30.1	34.3
500	3897	6.7	484	24.2	3	261	24.5	26.5
550	2152	3.7	341	17.0	3	108	19.3	18.2
600	1049	1.8	196	9.8	3	81	16.5	15.8
650	151	0.3	44	2.2	3	51	10.6	10.9
700	22	0.04	7	0.4	3	27	9.9	8.3

Table A8

Duration Information for Half-Moon Bay, CA

H1 cm	H > H1 cm	H > H1 %	Number of Events	Number of Events/yr	t_{min} hrs	t_{max} hrs	$\bar{t}$ hrs	σ_t hrs
500	768	1.3	105	5.2	3	123	21.7	21.2
550	373	0.6	50	2.5	3	108	22.0	21.7
600	168	0.3	23	1.2	3	78	21.5	17.8
650	17	0.03	3	0.2	12	21	17.0	4.6

Table A9

Duration Parameters with a Peak Wave Height Threshold, Nagshead, NC

H1 cm	H2 cm	% Records H > H1	Number of Events	Number of Events/yr	t_{min} hrs	t_{max} hrs	$\bar{t}$ hrs	σ_t hrs
100	300	25.0	52	2.6	18	1056	165.3	164.8
125	300	17.1	52	2.6	15	381	122.6	75.5
150	300	11.9	52	2.6	12	375	98.0	58.8
175	300	8.4	54	2.7	9	333	81.7	55.3
200	300	5.4	55	2.8	6	306	67.6	51.3
225	300	3.2	62	3.1	6	174	45.6	32.1
250	300	1.9	68	3.4	3	165	33.1	26.5
275	300	1.1	77	3.8	3	117	20.5	17.6
300	300	0.6	90	4.5	3	102	12.5	13.6
100	250	25.0	118	5.9	18	1056	119.6	120.6
125	250	17.1	119	6.0	15	381	92.5	64.1
150	250	11.9	123	6.2	12	375	74.6	52.6
175	250	8.4	129	6.4	6	333	60.0	47.4
200	250	5.4	136	6.8	3	306	45.4	40.3
225	250	3.2	173	8.6	3	174	26.0	25.6
250	250	1.9	212	10.6	3	165	15.5	19.9

Table A10

Duration Parameters with a Peak Wave Height Threshold, Daytona Beach, FL

H1 cm	H2 cm	% Records H > H1	Number of Events	Number of Events/yr	t_{min} hrs	t_{max} hrs	$\bar{t}$ hrs	σ_t hrs
100	300	30.7	49	2.4	36	1197	221.0	223.1
125	300	21.8	49	2.4	24	1191	184.3	211.1
150	300	15.2	51	2.6	15	1035	141.8	162.1
175	300	10.6	54	2.7	12	354	105.7	63.8
200	300	7.2	54	2.7	9	303	85.8	49.5
225	300	4.3	68	3.4	3	141	49.1	28.4
250	300	2.3	69	3.4	3	114	38.1	25.1
275	300	1.3	75	3.8	3	87	25.5	18.7
300	300	0.8	84	4.2	3	81	17.1	15.8
100	250	30.7	119	6.0	9	1197	162.6	162.1
125	250	21.8	121	6.0	9	1191	131.0	147.8
150	250	15.2	127	6.4	6	1035	101.5	111.5
175	250	10.6	133	6.6	6	354	78.0	52.4
200	250	7.2	140	7.0	3	303	59.9	41.3
225	250	4.3	193	9.6	3	141	30.9	24.8
250	250	2.3	238	11.9	3	114	16.9	20.0

Table A11

Distribution Parameters for Durations and Peak Wave Heights at Atlantic City, NJ

	Duration			Peak Wave Height		
Parameter	Sample	Type I	Weibull	Sample	Type I	Weibull
H1 = 200 cm (2.5% occurrence level)						
ε/α	-	7.75	1.23	-	222.6	7.06
β	-	8.93	14.0	-	32.1	258.0
x	12.8	12.9	13.1	240.9	241.1	241.4
σ	10.9	11.4	10.7	39.5	41.1	40.2
r	-	0.97	0.97	-	0.98	0.93
Σres^{2*}	-	1.40	1.47	-	0.838	3.51
std.err.	-	0.066	0.068	-	0.051	0.104
H1 = 250 cm (0.7% occurrence)						
ε/α	-	6.36	1.45	-	271.4	9.77
β	-	6.23	11.04	-	27.3	301.9
x	9.9	10.0	10.0	286.8	287.2	287.0
σ	7.3	8.0	7.0	32.7	35.0	35.3
r	-	0.97	0.97	-	0.99	0.93
Σres^{2*}	-	0.512	0.534	-	0.222	1.21
std.err.	-	0.068	0.070	-	0.045	0.105

(Continued)

*Sum of the square residuals.

Table A11 (Concluded)

Parameter	Duration Sample	Duration Type I	Duration Weibull	Peak Wave Height Sample	Peak Wave Height Type I	Peak Wave Height Weibull
H1 = 300 cm (0.1% occurrence)						
ε/α	-	4.65	1.45	-	318.7	12.6
β	-	6.39	9.20	-	25.2	345.5
x	8.1	8.3	8.6	332.2	333.2	331.6
σ	6.8	8.2	7.1	27.7	32.3	32.0
r	-	0.92	0.92	-	0.98	0.97
Σres^{2*}	-	0.366	0.366	-	0.067	0.139
std.err.	-	0.116	0.116	-	0.050	0.072
H1 = 350 cm (0.03% occurrence)						
ε/α	-	3.43	1.36	-	354.5	15.4
β	-	4.57	6.78	-	21.8	376.6
x	5.7	6.1	6.2	365.2	367.1	364.0
σ	4.1	5.9	4.6	18.4	28.0	29.0
r	-	0.89	0.89	-	0.86	0.77
Σres^{2*}	-	0.128	0.122	-	0.159	0.243
std.err.	-	0.135	0.132	-	0 .151	0.186

* Sum of the square residuals.

Table A12

Distribution Parameters for Durations and Peak Wave Heights at Nagshead, NC

Parameter	Duration Sample	Duration Type I	Duration Weibull	Peak Wave Height Sample	Peak Wave Height Type I	Peak Wave Height Weibull
H1 = 150 cm (11.9% occurrence)						
ε/α	-	6.19	1.06	-	181.5	4.80
β	-	36.0	25.0	-	44.5	226.8
x	26.9	27.0	24.5	207.0	207.2	207.7
σ	39.1	46.2	23.2	55.1	57.1	49.4
r	-	0.88	0.98	-	0.99	0.95
Σres^2	-	14.5	2.18	-	1.59	6.43
std.err.	-	0.14	0.05	-	0.04	0.09
H1 = 200 cm (5.4% occurrence)						
ε/α	-	6.20	0.99	-	218.5	5.55
β	-	26.0	19.6	-	43.2	264.8
x	21.1	21.2	19.7	243.2	243.4	244.6
σ	29.4	33.4	20.0	51.8	55.4	51.0
r	-	0.90	0.97	-	0.95	0.89
Σres^2	-	7.35	2.53	-	3.50	7.81
std.err.	-	0.13	0.07	-	0.09	0.13

(Continued)

Table A12 (Continued)

Parameter	Duration Sample	Duration Type I	Duration Weibull	Peak Wave Height Sample	Peak Wave Height Type I	Peak Wave Height Weibull
H1 = 250 cm (1.9% occurrence)						
ϵ/α	-	6.21	1.02	-	272.3	6.35
β	-	22.3	17.9	-	45.4	321.4
x	18.9	19.1	17.7	298.0	298.5	299.1
σ	25.0	28.6	17.3	53.9	58.2	55.0
r	-	0.90	0.96	-	0.96	0.90
Σres^2	-	2.79	1.04	-	1.05	2.73
std.err.	-	0.13	0.08	-	0.08	0.12
H1 = 300 cm (0.6% occurrence)						
ϵ/α	-	6.30	1.15	-	326.3	7.03
β	-	15.8	15.5	-	48.0	378.0
x	15.1	15.4	14.7	353.0	354.0	353.7
σ	17.2	20.2	12.8	55.7	61.6	59.2
r	-	0.92	0.98	-	0.97	0.92
Σres^2	-	0.91	0.30	-	0.35	0.95
std.err.	-	0.11	0.06	-	0.07	0.11

(Continued)

Table A12 (Continued)

Parameter	--------Duration------- Sample	Type I	Weibull	----Peak Wave Height--- Sample	Type I	Weibull
H1 = 350 cm (0.2% occurrence)						
ε/α	-	3.9	1.16	-	375.0	7.9
β	-	15.2	12.6	-	48.5	426.4
x	12.2	12.7	12.0	401.2	402.9	401.3
σ	14.9	19.6	10.4	53.4	62.2	60.3
r	-	0.87	0.96	-	0.96	0.90
Σres^2	-	0.68	0.22	-	0.21	0.52
std.err.	-	0.14	0.08	-	0.08	0.12
H1 = 400 cm (0.10% occurrence)						
ε/α	-	6.8	0.97	-	428.9	8.4
β	-	12.1	14.3	-	53.5	481.7
x	12.9	13.8	14.5	456.0	459.8	454.5
σ	12.0	15.5	15.0	53.4	68.6	64.8
r	-	0.96	0.97	-	0.97	0.95
Σres^2	-	0.08	0.06	-	0.05	0.10
std.err.	-	0.08	0.08	-	0.07	0.10

(Continued)

(Sheet 3 of 4)

Table A12 (Concluded)

Parameter	Duration Sample	Duration Type I	Duration Weibull	Peak Wave Height Sample	Peak Wave Height Type I	Peak Wave Height Weibull
H1 = 450 cm (0.03% occurrence)						
ε/α	-	4.2	1.51	-	460.4	8.6
β	-	4.5	7.52	-	52.9	512.2
x	6.4	6.8	6.8	486.0	490.9	484.1
σ	4.1	5.8	4.6	46.3	67.9	66.9
r	-	0.93	0.94	-	0.89	0.84
Σres^2	-	0.07	0.06	-	0.10	0.16
std.err.	-	0.11	0.10	-	0.13	0.16

(Sheet 4 of 4)

Table A13

Distribution Parameters for Durations and Peak Wave Heights at Daytona Beach, FL

	--------Duration-------			----Peak Wave Height---		
Parameter	Sample	Type I	Weibull	Sample	Type I	Weibull
H1 = 200 cm (7.2% occurrence)						
ε/α	-	12.4	0.96	-	221.3	5.63
β	-	30.0	27.9	-	42.4	266.9
x	29.5	29.7	28.4	295.3	245.7	246.7
σ	35.6	38.5	29.7	51.4	54.4	50.7
r	-	0.95	0.98	-	0.97	0.90
Σres^2	-	3.56	1.11	-	2.43	6.52
std.err.	-	0.09	0.05	-	0.08	0.12
H1 = 250 cm (2.3% occurrence)						
ε/α	-	10.2	1.01	-	217.0	6.48
β	-	21.6	21.9	-	42.8	318.1
x	22.5	22.7	21.8	295.3	295.7	296.3
σ	25.5	27.6	21.7	51.4	54.9	53.5
r	-	0.95	0.98	-	0.97	0.90
Σres^2	-	1.51	0.68	-	1.02	2.79
std.err.	-	0.09	0.06	-	0.07	0.12

(Continued)

Table A13 (Concluded)

Parameter	Duration Sample	Duration Type I	Duration Weibull	Peak Wave Height Sample	Peak Wave Height Type I	Peak Wave Height Weibull
H1 = 300 cm (0.8% occurrence)						
ε/α	-	10.8	1.21	-	329.7	8.43
β	-	15.5	20.6	-	39.4	372.6
x	19.5	19.8	19.3	351.6	352.4	351.7
σ	18.0	19.9	16.1	46.4	50.5	49.7
r	-	0.98	0.99	-	0.99	0.93
Σres^2	-	0.29	0.11	-	0.17	0.78
std.err.	-	0.06	0.04	-	0.05	0.10
H1 = 350 cm (0.2% occurrence)						
ε/α	-	7.39	1.40	-	375.7	10.4
β	-	10.9	14.6	-	35.4	413.9
x	13.3	13.7	13.3	394.8	396.2	394.4
σ	11.5	14.0	9.61	39.4	45.4	45.7
r	-	0.95	0.98	-	0.98	0.93
Σres^2	-	0.27	0.10	-	0.13	0.36
std.err.	-	0.09	0.06	-	0.06	0.11

Table A14

Distribution Parameters for Durations and Peak Wave Heights at Newport, OR

Parameter	Duration Sample	Duration Type I	Duration Weibull	Peak Wave Height Sample	Peak Wave Height Type I	Peak Wave Height Weibull
H1 = 400 cm (17.9% occurrence)						
ε/α	-	15.4	1.03	-	457.9	7.45
β	-	39.7	36.2	-	62.7	527.3
x	38.2	38.4	35.7	493.9	494.1	494.8
σ	47.1	51.0	34.6	77.7	80.4	78.5
r	-	0.94	0.99	-	0.99	0.95
Σres^2	-	8.07	0.87	-	1.52	6.81
std.err.	-	0.10	0.03	-	0.04	0.09
H1 = 450 cm (11.1% occurrence)						
ε/α	-	13.6	1.08	-	505.5	7.51
β	-	28.6	29.4	-	53.5	565.4
x	30.1	30.2	28.6	536.2	536.4	536.7
σ	34.3	36.7	26.5	65.9	63.6	67.7
r	-	0.96	0.99	-	0.99	0.96
Σres^2	-	4.75	0.92	-	1.55	3.96
std.err.	-	0.09	0.04	-	0.05	0.08

(Continued)

(Sheet 1 of 4)

Table A14 (Continued)

Parameter	Duration Sample	Duration Type I	Duration Weibull	Peak Wave Height Sample	Peak Wave Height Type I	Peak Wave Height Weibull
H1 = 500 cm (3.7% occurrence)						
ε/α	-	11.8	1.13	-	547.4	13.0
β	-	22.3	24.7	-	41.6	594.6
x	24.5	24.6	23.6	571.3	571.5	571.5
σ	26.5	28.6	21.0	51.4	53.4	53.5
r	-	0.96	0.99	-	0.98	0.98
Σres^2	-	3.01	0.72	-	1.24	1.68
std.err.	-	0.08	0.04	-	0.05	0.06
H1 = 550 cm (3.7% occurrence)						
ε/α	-	10.8	1.21	-	585.0	18.9
β	-	14.9	20.2	-	29.8	619.3
x	19.3	19.4	18.9	602.0	602.2	602.0
σ	18.2	19.2	15.7	37.1	38.2	39.5
r	-	0.98	0.99	-	0.99	0.97
Σres^2	-	1.32	0.61	-	0.40	1.47
std.err.	-	0.06	0.04	-	0.03	0.07

(Continued)

Table A14 (Continued)

Parameter	--------Duration------- Sample	Type I	Weibull	----Peak Wave Height--- Sample	Type I	Weibull
H1 = 600 cm (1.8% occurrence)						
ε/α	-	9.1	1.13	-	616.6	24.4
β	-	13.1	17.2	-	22.6	643.6
x	16.5	16.7	16.5	629.5	629.7	629.5
σ	15.8	16.8	14.6	27.6	29.0	32.1
r	-	0.97	0.98	-	0.99	0.98
Σres^2	-	1.09	0.75	-	0.41	2.53
std.err.	-	0.07	0.06	-	0.05	0.11
H1 = 650 cm (0.3% occurrence)						
ε/α	-	5.1	1.12	-	663.7	32.2
β	-	10.1	11.4	-	18.8	685.1
x	10.6	11.0	10.8	673.9	674.5	673.5
σ	10.9	13.0	9.2	21.4	24.1	26.3
r	-	0.92	0.95	-	0.98	0.93
Σres^2	-	0.52	0.34	-	0.13	0.50
std.err.	-	0.11	0.09	-	0.06	0.11

(Continued)

(Sheet 3 of 4)

Table A14 (Concluded)

Parameter	Duration: Sample	Duration: Type I	Duration: Weibull	Peak Wave Height: Sample	Peak Wave Height: Type I	Peak Wave Height: Weibull
H1 = 700 (0.04% occurrence)						
ε/α	-	5.4	1.15	-	711.5	71.9
β	-	9.4	11.5	-	10.4	721.1
x	9.9	10.8	10.9	716.4	717.5	715.5
σ	8.3	12.1	9.5	9.2	13.3	12.6
r	-	0.93	0.96	-	0.97	0.98
Σres^2	-	0.06	0.03	-	0.03	0.02
std.err.	-	0.11	0.08	-	0.07	0.06

(Sheet 4 of 4)

Table A15

Distribution Parameters for Durations and Peak Wave Heights at Half-Moon Bay, CA

Parameter	Duration Sample	Duration Type I	Duration Weibull	Peak Wave Height Sample	Peak Wave Height Type I	Peak Wave Height Weibull
H1 = 500 cm (1.3% occurrence)						
ϵ/α	-	11.7	1.11	-	533.1	14.2
β	-	18.0	22.5	-	36.3	574.0
x	21.7	22.0	21.7	553.4	554.0	553.3
σ	21.2	23.1	19.6	43.7	46.6	47.6
r	-	0.98	0.99	-	0.99	0.97
Σres^2	-	0.42	0.19	-	0.12	0.58
std.err.	-	0.06	0.04	-	0.03	0.08
H1 = 550 cm (0.6% occurrence)						
ϵ/α	-	11.4	1.14	-	575.9	19.4
β	-	19.4	22.9	-	28.3	607.4
x	22.0	22.6	21.8	591.3	592.1	590.8
σ	21.7	24.9	19.2	32.4	36.1	37.7
r	-	0.96	0.99	-	0.99	0.95
Σres^2	-	0.32	0.09	-	0.06	0.39
std.err.	-	0.08	0.04	-	0.04	0.09

(Continued)

Table A15 (Concluded)

Parameter	Duration Sample	Duration Type I	Duration Weibull	Peak Wave Height Sample	Peak Wave Height Type I	Peak Wave Height Weibull
H1 = 600 cm (0.3% occurrence)						
ε/α	-	12.8	1.17	-	607.1	20.0
β	-	16.6	23.7	-	26.9	637.4
x	21.5	22.3	22.4	621.3	622.7	620.5
σ	17.8	21.3	19.3	28.0	34.5	38.4
r	-	0.99	0.99	-	0.93	0.86
Σres^2	-	0.05	0.02	-	0.22	0.46
std.err.	-	0.05	0.03	-	0.10	0.15

Table A16

Results of Regression of Duration Against Conditions at the Peak of the Event for Atlantic City, NJ

H1	%(H > H1)	r_H	r_H^2	r_T	r_T^2	r_{H,H^2,T,T^2}	$r_{H\,L}^2$
200	2.5	0.71	0.69	0.54	0.52	0.74	0.66
250	0.7	0.69	0.69	0.40	0.38	0.71	0.63
300	0.1	0.80	0.80	0.23	0.21	0.80	0.65
350	0.03	0.10	0.10	0.24	0.22	0.63	0.21

Table A17

Results of Regression of Duration Against Conditions at the Peak of the Event for Nagshead, NC

H1	%(H > H1)	r_H	r_H^2	r_T	r_T^2	r_{H,H^2,T,T^2}	$r_{H\,L}^2$
200	5.6	0.82	0.81	0.61	0.62	0.82	0.75
250	2.0	0.80	0.79	0.50	0.51	0.81	0.72
300	0.7	0.72	0.72	0.60	0.62	0.74	0.73
350	0.3	0.55	0.54	0.54	0.56	0.70	0.56

Table A18

Results of Regression of Duration Against Conditions at the Peak of the Event for Daytona Beach, FL

H1	%(H > H1)	r_H	r_{H^2}	r_T	r_{T^2}	r_{H,H^2,T,T^2}	$r_{H^2 L}$
200	7.1	0.76	0.73	0.50	0.48	0.79	0.65
250	2.2	0.81	0.79	0.46	0.46	0.82	0.72
300	0.8	0.51	0.50	0.46	0.47	0.59	0.52
350	0.2	0.44	0.46	0.52	0.55	0.66	0.59

Table A19

Results of Regression of Duration Against Conditions at the Peak of the Event for Newport, OR

H1	%(H > H1)	r_H	r_{H^2}	r_T	r_{T^2}	r_{H,H^2,T,T^2}	$r_{H^2 L}$
500	6.9	0.59	0.60	0.32	0.32	0.61	0.55
550	3.8	0.59	0.59	0.24	0.24	0.59	0.50
600	1.9	0.42	0.42	0.20	0.20	0.45	0.38
650	0.3	0.66	0.66	-0.08	-0.08	0.66	0.26

Table A20

Results of Regression of Duration Against Conditions at the Peak of the Event for Half-Moon Bay, CA

H1	%(H > H1)	r_H	r_{H^2}	r_T	r_{T^2}	r_{H,H^2,T,T^2}	$r_{H^2 L}$
500	1.3	0.62	0.61	0.24	0.24	0.63	0.52
550	0.6	0.52	0.52	0.22	0.22	0.57	0.48
600	0.3	0.32	0.32	0.09	0.09	0.34	0.26

APPENDIX B

PERTINENT DATA FROM THE WAVE INFORMATION STUDIES PROGRAM

Table B1

Phase III Wave Data, Atlantic City, NJ

STATION 61 20 YEARS FOR ALL DIRECTIONS
SHORELINE ANGLE = 54.0 DEGREES AZIMUTH
WATER DEPTH = 10.00 METRES
PERCENT OCCURRENCE(X100) OF HEIGHT AND PERIOD FOR ALL DIRECTIONS

HEIGHT(METRES)	PERIOD(SECONDS)										TOTAL
	0.0-2.9	3.0-3.9	4.0-4.9	5.0-5.9	6.0-6.9	7.0-7.9	8.0-8.9	9.0-9.9	10.0-10.9	11.0-LONGER	
0. - 0.49	389	884	402	372	528	1094	443	71	98	197	4478
0.50 - 0.99	.	403	1084	394	107	617	446	63	113	227	3454
1.00 - 1.49	.	.	74	352	147	220	146	23	150	119	1231
1.50 - 1.99	.	.	.	15	91	155	71	12	19	22	385
2.00 - 2.49	.	.	.	.	17	81	44	10	5	6	163
2.50 - 2.99	.	.	.	.	.	8	19	11	3	4	45
3.00 - 3.49	.	.	.	.	.	.	1	3	3	1	8
3.50 - 3.99	.	.	.	.	.	.	.	.	1	.	1
4.00 - 4.49	.	.	.	.	.	.	.	.	.	.	0
4.50 - 4.99	.	.	.	.	.	.	.	.	.	.	0
5.00 - GREATER	.	.	.	.	.	.	.	.	.	.	0
TOTAL	389	1287	1560	1133	890	2175	1170	193	392	576	

AVE HS(M) = 0.65 LARGEST HS(M) = 4.13 TOTAL CASES = 58440

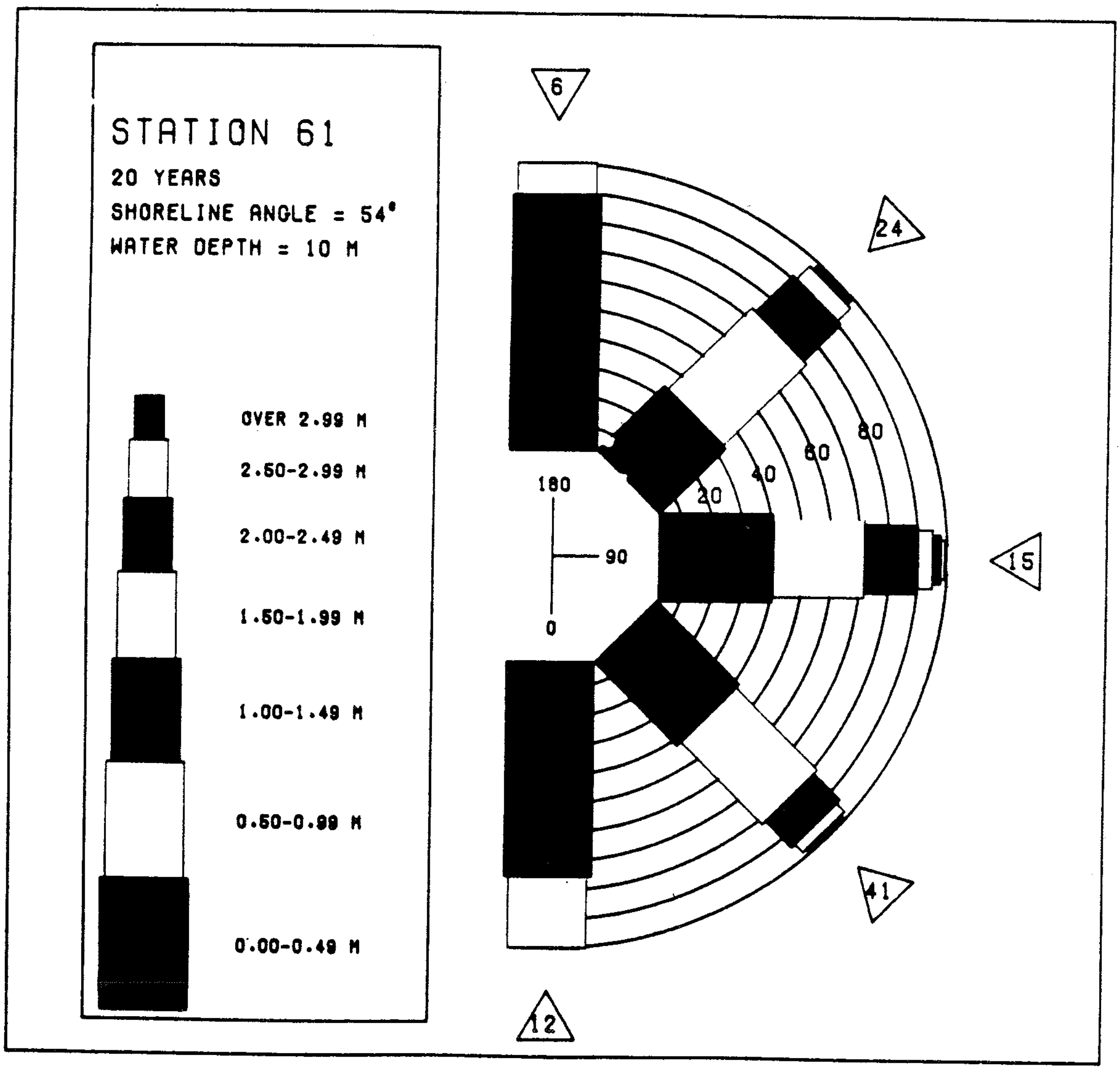


Figure B1. Phase III wave rose, Atlantic City, NJ

Table B2

Phase III Wave Data, Nagshead, NC

STATION 83 20 YEARS FOR ALL DIRECTIONS
SHORELINE ANGLE = 335.0 DEGREES AZIMUTH
WATER DEPTH = 10.00 METRES
PERCENT OCCURRENCE(X100) OF HEIGHT AND PERIOD FOR ALL DIRECTIONS

HEIGHT(METRES)	PERIOD(SECONDS) 0.0-2.9	3.0-3.9	4.0-4.9	5.0-5.9	6.0-6.9	7.0-7.9	8.0-8.9	9.0-9.9	10.0-10.9	11.0-LONGER	TOTAL
0. - 0.49	479	938	795	842	458	557	288	15	24	45	4461
0.50 - 0.99	.	206	811	470	157	640	354	15	56	97	2806
1.00 - 1.49	.	.	84	382	190	313	201	20	44	72	1306
1.50 - 1.99	.	.	.	32	154	271	112	30	20	36	655
2.00 - 2.49	.	.	.	.	31	187	80	22	19	14	353
2.50 - 2.99	.	.	.	.	.	18	50	34	11	8	121
3.00 - 3.49	.	.	.	.	.	.	5	20	9	4	38
3.50 - 3.99	.	.	.	.	.	.	.	2	8	3	13
4.00 - 4.49	.	.	.	.	.	.	.	.	1	4	5
4.50 - 4.99	.	.	.	.	.	.	.	.	.	1	1
5.00 - GREATER	.	.	.	.	.	.	.	.	.	.	0
TOTAL	479	1144	1690	1726	990	1986	1090	158	192	284	

AVE HS(M) = 0.71 LARGEST HS(M) = 5.92 TOTAL CASES = 58440

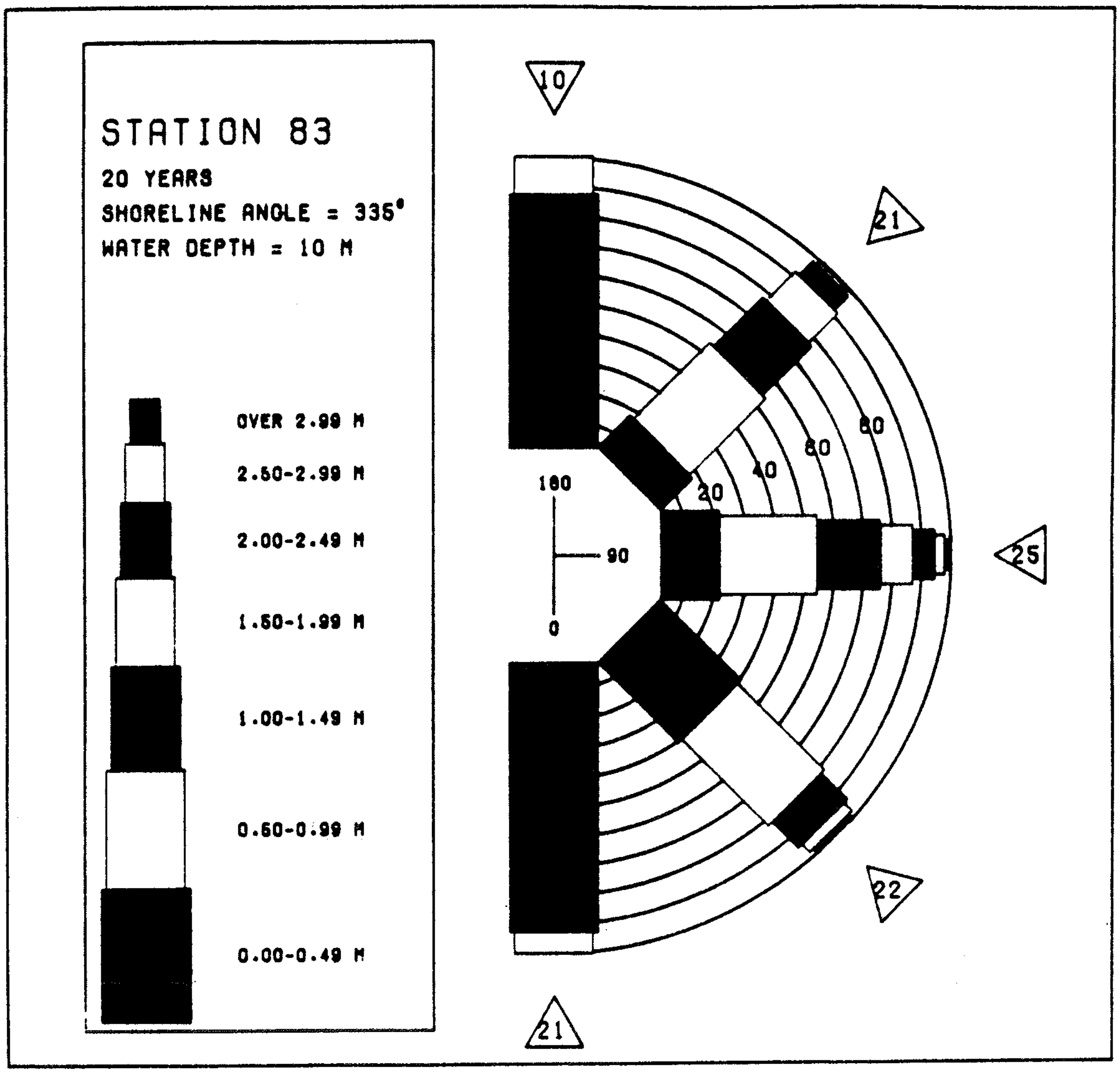


Figure B2. Phase III wave rose, Nagshead, NC

Table B3

Phase III Wave Data, Daytona Beach, FL

STATION 142 20 YEARS FOR ALL DIRECTIONS
SHORELINE ANGLE = 334.0 DEGREES AZIMUTH
WATER DEPTH = 10.00 METRES
PERCENT OCCURRENCE(X100) OF HEIGHT AND PERIOD FOR ALL DIRECTIONS

HEIGHT(METRES)	PERIOD(SECONDS) 0.0-2.9	3.0-3.9	4.0-4.9	5.0-5.9	6.0-6.9	7.0-7.9	8.0-8.9	9.0-9.9	10.0-10.9	11.0-LONGER	TOTAL
0. - 0.49	617	833	364	317	744	715	329	26	15	38	3998
0.50 - 0.99	.	338	919	220	132	719	362	89	66	53	2898
1.00 - 1.49	.	.	145	427	94	361	266	89	111	73	1566
1.50 - 1.99	.	.	.	71	144	247	160	35	62	84	803
2.00 - 2.49	.	.	.	.	34	246	128	21	25	31	485
2.50 - 2.99	.	.	.	.	.	7	85	29	9	14	144
3.00 - 3.49	.	.	.	.	.	.	3	31	13	8	55
3.50 - 3.99	.	.	.	.	.	.	.	.	13	4	17
4.00 - 4.49	.	.	.	.	.	.	.	.	2	1	3
4.50 - 4.99	.	.	.	.	.	.	.	.	.	1	1
5.00 - GREATER	.	.	.	.	.	.	.	.	.	1	1
TOTAL	617	1171	1428	1035	1148	2295	1333	320	316	307	0

AVE HS(M) = 0.82 LARGEST HS(M) = 5.03 TOTAL CASES = 58440

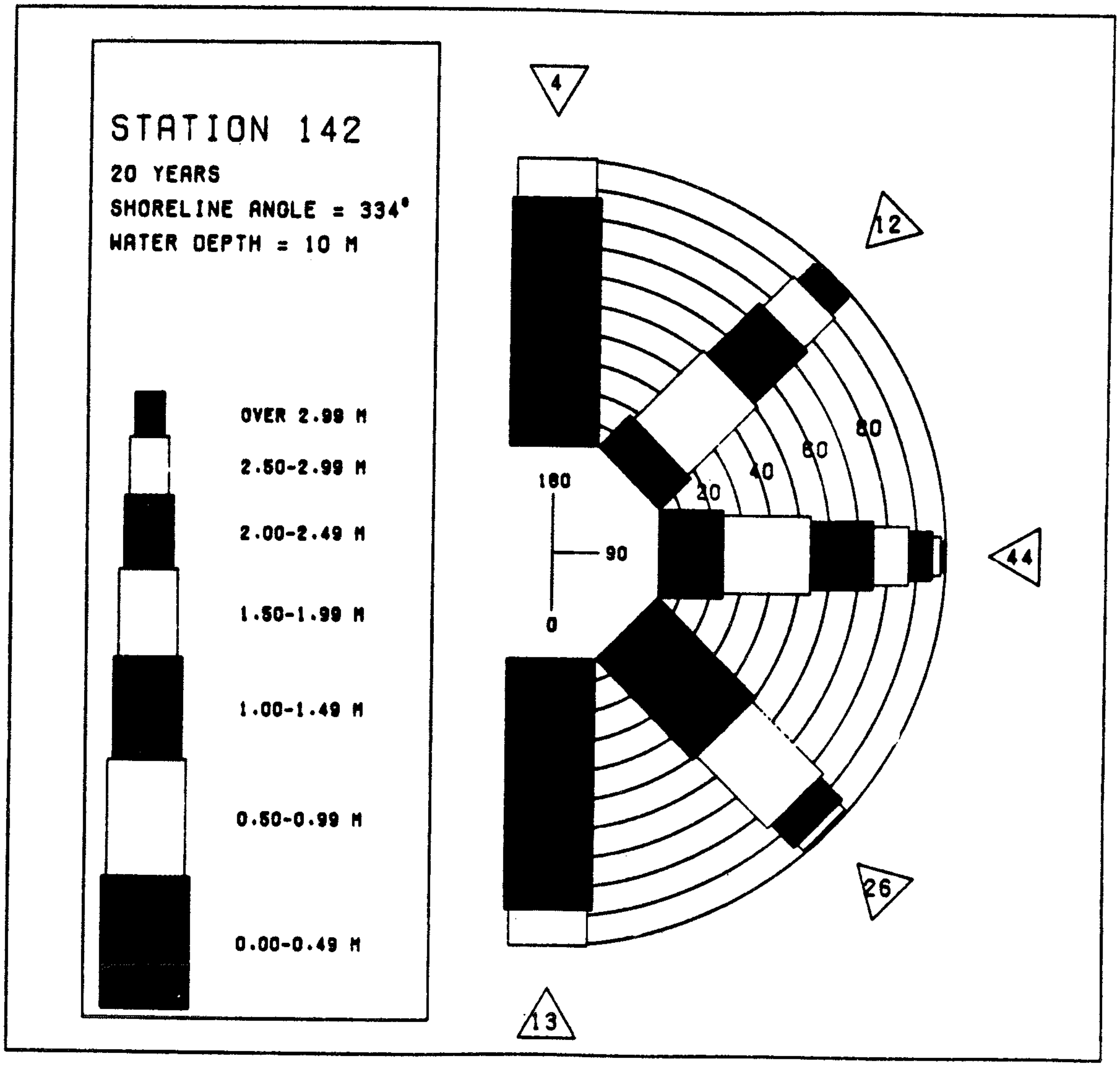


Figure B3. Phase III wave rose, Daytona Beach, FL

Table B4

Phase III Wave Data, Newport, OR

STATION 36 20 YEARS FOR ALL DIRECTIONS
SHORELINE ANGLE = 181.0 DEGREES AZIMUTH
WATER DEPTH = 10.00 METERS
PERCENT OCCURRENCE(X100) OF HEIGHT AND PERIOD FOR ALL DIRECTIONS

HEIGHT(METERS)	PERIOD(SECONDS) 0.0-2.9	3.0-3.9	4.0-4.9	5.0-5.9	6.0-6.9	7.0-7.9	8.0-8.9	9.0-9.9	10.0-10.9	11.0-LONGER	TOTAL
0. - 0.49	.	.	.	.	.	.	.	.	.	.	0
0.50 - 0.99	.	.	12	10	21	62	69	68	37	25	304
1.00 - 1.49	.	.	29	82	55	158	263	336	313	167	1403
1.50 - 1.99	.	.	3	159	98	77	137	329	486	521	1810
2.00 - 2.49	.	.	.	7	69	42	29	104	360	813	1424
2.50 - 2.99	.	.	.	.	7	28	15	28	122	1001	1201
3.00 - 3.49	.	.	.	.	.	8	13	17	56	1004	1098
3.50 - 3.99	.	.	.	.	.	.	4	18	24	885	931
4.00 - 4.49	.	.	.	.	.	.	.	6	14	662	682
4.50 - 4.99	.	.	.	.	.	.	.	.	3	442	445
5.00 - GREATER	.	.	.	.	.	.	.	.	.	671	671
TOTAL	0	0	44	258	250	375	530	906	1415	6191	

AVE HS(M) = 2.76 LARGEST HS(M) = 7.27 TOTAL CASES = 58440

Table B5

Phase III Wave Data, Half-Moon Bay, CA

STATION 105 20 YEARS FOR ALL DIRECTIONS
SHORELINE ANGLE = 152.0 DEGREES AZIMUTH
WATER DEPTH = 10.00 METERS
PERCENT OCCURRENCE(X100) OF HEIGHT AND PERIOD FOR ALL DIRECTIONS

HEIGHT(METERS)	PERIOD(SECONDS) 0.0-2.9	3.0-3.9	4.0-4.9	5.0-5.9	6.0-6.9	7.0-7.9	8.0-8.9	9.0-9.9	10.0-10.9	11.0-LONGER	TOTAL
0. - 0.49	.	.	.	.	.	4	7	2	1	.	14
0.50 - 0.99	.	.	8	43	53	150	177	145	111	69	756
1.00 - 1.49	.	.	11	222	189	234	275	390	492	545	2358
1.50 - 1.99	.	.	.	88	223	152	101	179	411	1042	2196
2.00 - 2.49	.	.	.	2	41	99	48	54	150	1288	1682
2.50 - 2.99	.	.	.	.	3	42	34	23	38	977	1117
3.00 - 3.49	.	.	.	.	.	5	19	16	17	707	764
3.50 - 3.99	.	.	.	.	.	.	8	12	10	465	495
4.00 - 4.49	.	.	.	.	.	.	.	6	7	288	301
4.50 - 4.99	.	.	.	.	.	.	.	1	1	152	154
5.00 - GREATER	.	.	.	.	.	.	.	.	.	130	130
TOTAL	0	0	19	355	509	686	669	828	1238	5663	

AVE HS(M) = 2.14 LARGEST HS(M) = 7.04 TOTAL CASES = 58440

Table B6

Phase III Duration Data, Atlantic Coast

Duration, hr, of Waves with H_s over a Specified Wave Height

Station No.	Duration	H_s, m 0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5
1	Mean	25*	18	14	9	6	7	5	9	--	--	--
	Max	531	222	138	60	33	24	9	9	--	--	--
2	Mean	28	20	16	10	7	6	6	5	--	--	--
	Max	534	240	144	69	39	27	21	9	--	--	--
3	Mean	30	21	18	11	8	6	7	5	3	--	--
	Max	534	243	144	69	42	36	24	9	3	--	--
4	Mean	31	21	19	12	9	6	6	6	3	3	--
	Max	534	243	150	75	42	36	24	21	3	3	--
5	Mean	35	23	18	12	9	7	6	5	--	--	--
	Max	3,567	294	153	69	42	36	21	12	--	--	--
6	Mean	34	22	17	11	9	7	5	8	--	--	--
	Max	3,564	294	153	69	39	36	18	12	--	--	--
7	Mean	26	19	16	12	10	8	6	6	--	--	--
	Max	834	360	123	84	57	24	12	6	--	--	--
8	Mean	30	19	16	12	9	8	6	8	--	--	--
	Max	606	357	126	63	51	30	15	12	--	--	--
9	Mean	28	19	16	12	10	9	6	9	--	--	--
	Max	606	360	135	63	48	27	15	9	--	--	--
10	Mean	32	21	18	13	9	7	5	3	--	--	--
	Max	594	192	105	63	42	24	15	3	--	--	--
11	Mean	32	21	18	12	9	7	6	3	--	--	--
	Max	597	192	105	63	45	24	15	3	--	--	--
12	Mean	25	18	15	11	8	8	6	5	3	--	--
	Max	594	231	66	45	33	24	18	9	3	--	--
13	Mean	23	17	14	10	8	7	6	6	--	--	--
	Max	594	228	66	45	33	21	15	9	--	--	--
14	Mean	24	19	15	12	11	8	7	3	--	--	--
	Max	429	213	96	57	45	30	18	3	--	--	--
15	Mean	25	19	16	12	10	8	6	5	--	--	--
	Max	786	228	96	54	42	36	18	6	--	--	--
16	Mean	27	21	15	11	9	7	6	--	--	--	--
	Max	861	825	153	69	33	21	12	--	--	--	--
17	Mean	29	22	17	13	10	8	7	5	--	--	--
	Max	864	477	156	105	60	30	15	6	--	--	--
18	Mean	29	22	18	12	10	9	6	--	--	--	--
	Max	864	477	168	102	60	27	15	--	--	--	--
19	Mean	30	23	17	12	9	8	6	--	--	--	--
	Max	696	474	159	69	60	27	15	--	--	--	--
20	Mean	29	22	18	15	11	8	7	5	--	--	--
	Max	633	360	153	108	60	33	15	6	--	--	--
21	Mean	19	18	17	11	10	8	11	--	--	--	--
	Max	282	132	114	102	63	18	12	--	--	--	--
22	Mean	23	19	14	10	8	5	4	--	--	--	--
	Max	735	324	102	84	39	21	9	--	--	--	--
23	Mean	22	19	16	12	9	10	8	--	--	--	--
	Max	735	456	114	96	69	30	15	--	--	--	--
24	Mean	23	20	17	13	13	12	9	6	--	--	--
	Max	633	456	168	111	96	66	36	9	--	--	--
25	Mean	25	22	18	14	14	12	14	7	--	--	--
	Max	798	456	153	123	102	69	39	9	--	--	--
26	Mean	23	21	17	13	14	12	16	7	--	--	--
	Max	798	456	186	120	99	69	39	9	--	--	--
27	Mean	16	15	15	11	10	6	6	--	--	--	--
	Max	246	243	117	96	45	18	6	--	--	--	--
28	Mean	26	22	20	15	15	14	13	7	--	--	--
	Max	1,265	795	360	186	108	72	39	9	--	--	--
29	Mean	26	22	21	17	14	13	13	8	3	--	--
	Max	603	492	270	147	102	69	39	18	3	--	--
30	Mean	25	21	19	16	14	11	12	7	3	--	--
	Max	585	492	240	129	99	69	39	15	3	--	--
31	Mean	29	22	20	15	12	9	9	7	--	--	--
	Max	1,431	405	186	.120	72	45	30	12	--	--	--
32	Mean	32	24	22	18	16	16	12	8	6	--	--
	Max	621	618	528	186	117	63	48	18	6	--	--
33	Mean	48	26	19	15	12	9	8	6	8	3	--
	Max	2,775	435	189	132	60	30	27	18	12	3	--
34	Mean	49	28	20	15	12	10	8	7	8	3	--
	Max	2,775	567	195	132	60	33	27	21	12	3	--
35	Mean	42	23	17	14	11	9	7	6	12	6	--
	Max	1,971	231	186	108	60	30	27	21	12	6	--
36	Mean	37	22	18	13	10	9	10	6	9	3	--
	Max	894	537	309	222	129	42	24	15	9	3	--
37	Mean	33	19	15	12	9	7	6	6	12	6	--
	Max	597	189	129	63	45	24	24	12	12	6	--
38	Mean	43	23	17	14	11	9	7	5	9	3	--
	Max	1,665	432	183	66	54	30	27	12	9	3	--

(Continued)

* Duration is shown in hours for H_s readings "greater than" 0.5, 1.0, etc.

(Sheet 1 of 5)

Table B6 (Continued)

Station No.	Duration	H_s, m 0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5
39	Mean	33	21	16	12	9	7	7	5	6	3	--
	Max	990	372	186	66	42	27	21	9	6	3	--
40	Mean	25	17	14	10	7	7	5	5	--	--	--
	Max	897	348	126	54	27	21	15	6	--	--	--
41	Mean	22	15	13	10	7	7	4	6	--	--	--
	Max	543	180	111	54	24	18	9	6	--	--	--
42	Mean	42	23	19	12	9	6	5	3	--	--	--
	Max	1,287	258	147	57	39	27	15	3	--	--	--
43	Mean	41	23	19	12	9	6	5	3	--	--	--
	Max	1,287	258	147	54	30	24	15	3	--	--	--
44	Mean	41	23	19	12	9	6	5	3	--	--	--
	Max	1,287	258	147	57	30	24	15	3	--	--	--
45	Mean	41	23	18	12	9	6	5	3	--	--	--
	Max	1,287	258	144	54	30	24	15	3	--	--	--
46	Mean	42	23	19	12	9	6	5	3	--	--	--
	Max	1,287	258	147	57	30	24	15	3	--	--	--
47	Mean	34	19	16	12	9	8	6	5	--	--	--
	Max	1,029	237	126	66	36	27	12	6	--	--	--
48	Mean	34	19	16	12	9	8	6	5	--	--	--
	Max	1,035	291	129	66	36	27	12	6	--	--	--
49	Mean	35	19	16	11	9	6	5	6	--	--	--
	Max	2,466	339	285	66	27	21	12	6	--	--	--
50	Mean	35	20	16	11	8	6	6	6	--	--	--
	Max	2,481	339	285	63	33	21	12	6	--	--	--
51	Mean	31	19	15	10	8	6	6	6	--	--	--
	Max	1,569	303	279	63	24	18	12	6	--	--	--
52	Mean	27	17	14	10	7	6	3	3	--	--	--
	Max	942	306	75	51	24	15	3	3	--	--	--
53	Mean	27	18	13	9	7	6	3	--	--	--	--
	Max	948	210	111	75	30	15	3	--	--	--	--
54	Mean	26	18	15	11	7	6	5	--	--	--	--
	Max	678	210	117	81	33	18	9	--	--	--	--
55	Mean	27	19	15	11	7	7	5	--	--	--	--
	Max	1,353	213	120	81	30	18	9	--	--	--	--
56	Mean	27	19	17	12	9	8	5	--	--	--	--
	Max	735	231	174	87	42	21	9	--	--	--	--
57	Mean	27	19	16	12	10	8	5	--	--	--	--
	Max	783	261	171	90	45	21	9	--	--	--	--

Station No.	Duration	H_s, m 0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5
58	Mean	28	20	17	13	10	9	6	3	--	--	--
	Max	786	297	171	114	66	24	12	3	--	--	--
59	Mean	32	20	16	11	9	7	6	--	--	--	--
	Max	1,578	411	174	84	36	18	9	--	--	--	--
60	Mean	35	23	19	13	9	8	7	12	--	--	--
	Max	3,117	1,869	387	126	51	27	27	12	--	--	--
61	Mean	35	22	16	10	8	7	5	3	--	--	--
	Max	2,952	1,572	168	54	30	24	9	3	--	--	--
62	Mean	34	23	19	13	9	9	8	7	3	--	--
	Max	3,162	1,869	387	144	57	30	27	12	3	--	--
63	Mean	35	24	19	13	9	8	8	12	--	--	--
	Max	3,117	1,869	387	126	51	27	21	12	--	--	--
64	Mean	32	22	15	10	8	8	5	3	--	--	--
	Max	2,955	1,572	204	72	42	24	9	3	--	--	--
65	Mean	28	25	24	16	11	10	9	10	8	--	--
	Max	1,782	1,773	597	192	66	42	27	21	9	--	--
66	Mean	28	18	17	15	11	11	9	11	12	6	--
	Max	1,323	342	336	180	81	69	54	21	21	6	--
67	Mean	30	18	16	14	10	10	10	13	12	6	--
	Max	1,806	342	336	195	81	69	54	21	21	6	--
68	Mean	33	19	15	12	10	15	8	9	--	--	--
	Max	2,841	342	186	75	60	51	21	9	--	--	--
69	Mean	34	19	15	12	10	10	14	10	15	--	--
	Max	2,841	342	333	189	75	60	51	21	15	--	--
70	Mean	35	18	14	11	9	9	10	12	6	--	--
	Max	2,292	513	324	177	66	54	21	21	6	--	--
71	Mean	31	17	13	9	8	8	6	3	--	--	--
	Max	2,208	324	285	114	39	21	18	3	--	--	--
72	Mean	30	17	14	12	10	9	11	7	12	--	--
	Max	2,211	342	324	186	69	54	42	21	12	--	--
73	Mean	31	18	15	13	10	9	10	13	8	--	--
	Max	2,292	347	327	186	69	60	48	21	12	--	--
74	Mean	36	21	16	13	11	11	13	7	8	6	--
	Max	1,347	459	339	318	183	99	42	15	12	6	--
75	Mean	31	20	17	14	11	12	13	7	8	3	--
	Max	1,347	429	339	315	183	102	39	15	12	3	--
76	Mean	31	20	18	15	12	11	15	7	8	6	--
	Max	1,347	429	339	318	183	99	42	15	12	6	--

(Continued)

Table B6 (Continued)

Station No.	Duration	Hs, m 0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5
77	Mean	33	22	20	17	14	13	9	13	6	3	--
	Max	1,347	426	345	294	108	87	48	39	12	3	--
78	Mean	34	23	22	17	14	12	9	12	6	3	--
	Max	1,851	429	345	231	114	93	48	39	12	3	--
79	Mean	30	23	21	17	15	12	11	13	6	3	--
	Max	873	510	348	222	159	93	48	42	12	3	--
80	Mean	32	24	21	17	15	11	12	14	6	3	--
	Max	1,473	750	390	222	159	93	48	42	12	3	--
81	Mean	33	25	22	17	15	12	11	15	6	3	--
	Max	1,473	753	477	222	159	93	48	42	12	3	--
82	Mean	42	28	24	19	16	12	11	12	5	6	3
	Max	2,121	1,056	375	312	165	102	72	42	9	9	3
83	Mean	42	27	23	18	16	12	11	13	5	5	3
	Max	2,127	1,056	375	309	165	102	72	42	9	6	3
84	Mean	42	26	23	18	15	13	11	13	6	5	3
	Max	2,127	1,059	375	309	165	102	72	42	9	6	3
85	Mean	33	23	20	15	14	14	12	14	5	3	--
	Max	846	405	372	267	168	102	90	39	9	3	--
86	Mean	32	22	18	15	12	12	15	14	6	--	--
	Max	846	387	366	264	168	96	78	39	9	--	--
87	Mean	36	25	22	18	14	13	15	12	8	3	--
	Max	831	789	570	300	207	159	87	39	15	3	--
88	Mean	41	23	17	12	9	7	6	--	--	--	--
	Max	1,365	285	183	63	39	27	12	--	--	--	--
89	Mean	37	21	15	11	9	8	5	--	--	--	--
	Max	1,314	282	186	69	36	21	9	--	--	--	--
90	Mean	40	22	16	12	11	8	6	--	--	--	--
	Max	1,311	1,005	186	69	48	21	9	--	--	--	--
91	Mean	38	21	16	11	11	8	6	--	--	--	--
	Max	1,308	429	183	69	36	18	9	--	--	--	--
92	Mean	38	20	15	12	10	8	5	--	--	--	--
	Max	1,308	429	183	69	48	18	9	--	--	--	--
93	Mean	37	20	16	12	9	8	5	--	--	--	--
	Max	1,305	579	180	69	54	21	9	--	--	--	--
94	Mean	33	20	16	12	10	8	11	--	--	--	--
	Max	1,209	522	180	171	51	27	21	--	--	--	--
95	Mean	36	21	16	11	9	8	6	3	--	--	--
	Max	951	189	93	57	36	27	9	3	--	--	--
96	Mean	38	22	18	12	10	9	5	--	--	--	--
	Max	948	384	246	90	39	27	12	--	--	--	--
97	Mean	32	19	14	11	10	7	5	--	--	--	--
	Max	942	336	246	90	36	27	6	--	--	--	--
98	Mean	31	19	14	11	10	7	6	--	--	--	--
	Max	942	336	264	87	36	18	9	--	--	--	--
99	Mean	30	19	15	11	10	7	6	--	--	--	--
	Max	819	336	258	87	33	27	15	--	--	--	--
100	Mean	29	18	15	11	10	7	6	--	--	--	--
	Max	819	336	258	84	36	27	18	--	--	--	--
101	Mean	34	18	14	10	9	8	8	--	--	--	--
	Max	885	411	156	57	48	24	12	--	--	--	--
102	Mean	30	18	13	10	9	10	8	--	--	--	--
	Max	825	411	129	54	48	24	12	--	--	--	--
103	Mean	32	19	16	11	9	11	8	--	--	--	--
	Max	825	429	141	72	51	36	12	--	--	--	--
104	Mean	31	19	16	11	9	11	8	--	--	--	--
	Max	825	429	141	72	51	36	12	--	--	--	--
105	Mean	25	16	14	10	8	6	6	--	--	--	--
	Max	558	174	84	42	30	18	9	--	--	--	--
106	Mean	27	17	15	12	9	7	5	--	--	--	--
	Max	567	255	96	48	30	18	9	--	--	--	--
107	Mean	26	18	15	11	9	7	4	--	--	--	--
	Max	621	330	114	42	27	18	6	--	--	--	--
108	Mean	26	18	15	11	10	8	4	--	--	--	--
	Max	591	333	117	42	27	21	6	--	--	--	--
109	Mean	30	18	17	11	8	7	3	--	--	--	--
	Max	1,425	216	99	45	33	21	3	--	--	--	--
110	Mean	32	19	15	10	8	6	6	--	--	--	--
	Max	2,154	216	105	60	39	21	6	--	--	--	--
111	Mean	35	21	16	11	9	8	5	--	--	--	--
	Max	2,322	234	108	60	39	24	6	--	--	--	--
112	Mean	35	21	17	12	8	8	3	--	--	--	--
	Max	2,619	225	123	66	42	27	3	--	--	--	--
113	Mean	36	22	19	13	11	7	9	--	--	--	--
	Max	1,113	432	204	93	57	36	15	--	--	--	--
114	Mean	45	27	21	15	12	9	11	6	3	--	--
	Max	2,706	525	447	180	78	48	30	15	3	--	--

(Continued)

(Sheet 3 of 5)

Table B6 (Continued)

Station No.	Duration	H_s, m 0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5
115	Mean	39	21	18	12	11	7	5	18	9	--	--
	Max	2,679	399	336	135	90	21	9	45	9	--	--
116	Mean	42	24	19	14	12	8	8	6	--	--	--
	Max	2,694	456	396	174	78	36	18	6	--	--	--
117	Mean	42	23	18	13	11	9	8	18	--	--	--
	Max	2,694	456	393	156	78	36	18	18	--	--	--
118	Mean	38	23	20	13	10	7	5	3	3	3	--
	Max	2,694	531	231	126	48	21	12	3	3	3	--
119	Mean	29	19	16	11	10	8	3	3	3	9	--
	Max	2,391	507	177	108	33	21	3	3	3	9	--
120	Mean	36	22	19	12	8	5	6	3	3	27	--
	Max	1,635	309	114	102	30	9	6	3	3	27	--
121	Mean	35	23	18	11	8	4	6	3	3	--	--
	Max	1,632	306	114	102	24	9	6	3	3	--	--
122	Mean	36	22	19	11	8	5	6	3	3	--	--
	Max	1,635	309	114	102	30	9	6	3	3	--	--
123	Mean	42	25	19	13	11	7	9	3	9	--	--
	Max	3,018	486	288	96	51	30	12	3	9	--	--
124	Mean	42	25	19	13	10	7	6	3	27	--	--
	Max	3,018	489	285	99	51	30	12	3	27	--	--
125	Mean	41	24	19	12	11	7	6	3	81	--	--
	Max	3,018	486	288	99	51	27	12	3	81	--	--
126	Mean	42	26	20	14	12	8	8	3	--	--	--
	Max	2,904	912	252	99	60	30	15	3	--	--	--
127	Mean	41	27	23	15	12	7	4	3	--	--	--
	Max	2,469	1,593	441	99	60	21	6	3	--	--	--
128	Mean	42	28	24	17	12	6	4	3	3	--	--
	Max	2,421	1,593	441	99	60	21	6	3	3	--	--
129	Mean	42	27	23	16	12	7	4	3	9	--	--
	Max	2,124	1,593	441	99	60	21	6	3	9	--	--
130	Mean	42	29	24	17	12	7	4	3	3	--	--
	Max	2,124	1,593	447	93	63	24	6	3	3	--	--
131	Mean	34	23	22	16	7	8	3	--	--	--	--
	Max	2,256	303	150	102	39	15	3	--	--	--	--
132	Mean	34	24	22	17	8	9	3	--	--	--	--
	Max	2,259	303	150	102	45	18	3	--	--	--	--
133	Mean	34	23	22	18	8	9	3	--	--	--	--
	Max	2,193	303	153	123	45	18	3	--	--	--	--
134	Mean	34	24	23	18	8	10	3	--	--	--	--
	Max	2,193	300	153	126	45	21	3	--	--	--	--
135	Mean	44	28	27	20	12	8	5	4	3	--	--
	Max	1,755	723	324	186	75	33	9	6	3	--	--
136	Mean	44	28	27	20	12	8	5	4	3	--	--
	Max	1,755	723	324	186	75	33	9	6	3	--	--
137	Mean	43	29	27	20	13	8	5	4	3	--	--
	Max	1,755	723	300	186	75	33	9	6	3	--	--
138	Mean	48	35	32	22	16	13	8	5	3	3	--
	Max	1,902	1,221	567	237	111	51	27	9	3	3	--
139	Mean	49	35	31	23	16	14	8	5	3	3	--
	Max	1,902	1,221	567	237	111	57	27	9	3	3	--
140	Mean	49	35	32	24	16	14	8	5	3	3	--
	Max	1,902	1,221	567	237	111	57	27	9	3	3	--
141	Mean	55	39	32	25	18	16	12	7	6	3	--
	Max	1,797	1,197	1,035	303	114	81	60	18	9	3	--
142	Mean	56	39	33	25	17	17	12	7	6	3	--
	Max	1,797	1,197	1,035	303	114	81	60	21	9	3	--
143	Mean	55	40	33	25	17	17	12	8	8	3	--
	Max	1,797	1,197	1,035	303	114	81	60	21	12	3	--
144	Mean	53	38	33	23	15	10	5	3	23	9	--
	Max	1,803	1,791	711	240	114	60	9	3	36	9	--
145	Mean	54	38	33	23	15	10	5	3	68	27	--
	Max	1,803	1,791	750	237	129	60	9	3	108	27	--
146	Mean	52	37	32	22	15	10	5	--	--	--	--
	Max	1,803	1,791	708	240	111	60	9	--	--	--	--
147	Mean	48	30	26	17	12	7	15	--	--	--	--
	Max	2,175	1,668	738	186	51	24	27	--	--	--	--
148	Mean	55	34	31	21	15	11	6	--	--	--	--
	Max	2,439	1,704	1,083	330	114	57	9	--	--	--	--
149	Mean	56	33	28	19	16	12	9	8	--	--	--
	Max	4,587	1,416	279	144	105	45	18	9	--	--	--
150	Mean	57	35	29	20	15	12	10	8	--	--	--
	Max	4,587	1,431	279	147	105	45	18	9	--	--	--
151	Mean	59	35	29	20	15	12	10	8	--	--	--
	Max	4,587	1,416	279	144	105	45	18	9	--	--	--
152	Mean	58	32	24	17	13	8	3	23	--	--	--
	Max	4,503	1,497	216	147	72	18	3	27	--	--	--

(Continued)

Table B6 (Concluded)

Station No.	Duration	Hs, m 0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5
153	Mean	58	31	24	17	13	8	3	68	--	--	--
	Max	4,503	1,485	213	147	72	18	3	81	--	--	--
154	Mean	58	32	24	17	13	8	3	203	--	--	--
	Max	4,503	1,485	213	147	72	18	3	243	--	--	--
155	Mean	58	41	32	20	14	11	6	6	--	--	--
	Max	2,259	1,560	816	252	78	39	9	6	--	--	--
156	Mean	57	38	31	20	14	10	5	3	--	--	--
	Max	2,259	1,560	798	252	78	39	9	3	--	--	--
157	Mean	57	37	30	19	14	9	5	3	--	--	--
	Max	2,259	1,551	798	252	78	21	9	3	--	--	--
158	Mean	44	29	27	18	13	8	6	9	--	--	--
	Max	3,600	1,026	585	225	84	33	9	9	--	--	--
159	Mean	42	26	23	12	12	12	3	27	--	--	--
	Max	3,450	999	537	156	57	33	3	27	--	--	--
160	Mean	42	26	23	14	11	11	3	81	--	--	--
	Max	3,450	816	576	156	60	33	3	81	--	--	--
161	Mean	26	17	13	8	5	--	--	--	--	--	--
	Max	348	147	90	39	18	--	--	--	--	--	--
162	Mean	26	17	13	9	5	--	--	--	--	--	--
	Max	348	204	90	42	18	--	--	--	--	--	--
163	Mean	26	17	13	9	6	--	--	--	--	--	--
	Max	348	204	90	42	21	--	--	--	--	--	--
164	Mean	25	16	12	6	6	--	--	--	--	--	--
	Max	366	162	87	42	9	--	--	--	--	--	--
165	Mean	24	15	10	5	3	--	--	--	--	--	--
	Max	366	201	87	27	3	--	--	--	--	--	--
166	Mean	23	14	7	5	9	--	--	--	--	--	--
	Max	363	201	45	15	9	--	--	--	--	--	--

(Sheet 5 of 5)

APPENDIX C

COMPUTER PROGRAM STRMDIST

FORTRAN Listing - Program "STRMDIST"

```
$$N,J
$:IDENT:R0CDOPS,OPSMITH
$:OPTION:FORTRAN
$:USE:.GTLIT
$:FORTY
C                         HONEYWELL VERSION 1/2/86
C       **  PROGRAM "STRMDIST" READS A WIS PHASE III DATA FILE **
C       **  AND IDENTIFIES STORMS WHERE CONSECUTIVE RECORDS    **
C       **  HAVE WAVE HEIGHTS EXCEEDING A SPECIFIED THRESHOLD. **
C       **  THE NUMBER, PEAK CONDITIONS AND DURATIONS OF THESE **
C       **  STORMS ARE THEN TABULATED.  STORMS ONLY 6 HOURS    **
C       **  APART ARE CONSIDERED AS A SINGLE EVENT.  THE       **
C       **  PROGRAM ALSO FITS AN EXTREMAL TYPE I AND A WEIBULL **
C       **  DISTRIBUTION TO THE PEAK WAVE HEIGHTS AND THE DUR- **
C       **  ATIONS AND REPORTS THE PARAMETERS OF EACH.         **
C
        DIMENSION DUR(999),HPEAK(999),TPEAK(999),DPEAK(999),DTPEAK(999)
        INTEGER DATM,HSEA,TSEA,DSEA,HSWL,TSWL,DSWL,STNO,DATIM,H,T,D
        INTEGER STMNO,DN,HPK,TPK,DPK,HPEAK,TPEAK,DPEAK,DUR,H1,H2,RECNO
        INTEGER DTPEAK,DATIME,DTPK,FLAG,NOREC,NOYRS,TMIN,TMAX
        INTEGER YR,YRP,MO,MOP,DY,DYP,TM,TMP,YRDIFF,MODIFF,DYDIFF,
       &TMDIFF,HPKMIN,HPKMAX
        CHARACTER*64 FNAME
        CHARACTER*8 VARIABLE
        CHARACTER*8 VARIABLE
C
C      **  READ WIS DATA FILE AND WRITE FILE OF STORMS  **
C      **  EXCEEDING 1ST WAVE HEIGHT THRESHOLD, H1 (CM) **
C
        FNAME="NAGSHEAD, NORTH CAROLINA"
        NOREC=58440
        NOYRS=20
        H1=300
C      **  K = THE STORM NO. ASSIGNED TO CONSECUTIVE RECORDS  **
        K=0
C      **  J = THE NO. RECORDS WHERE H > H1                   **
        J=0
        YRP=999
        MOP=999
        DYP=999
        TMP=999
        CALL ATTACH(01,"/A3083;",1,0,ISTAT)
        ISTAT=FLD(6,6,ISTAT)
        IF(ISTAT.NE.0) GO TO 900
        CALL FMEDIA(07,6)
        CALL FMEDIA(08,6)
        CALL FMEDIA(09,6)
        READ(1,10) YR,MO,DY,TM,HSEA,TSEA,DSEA,HSWL,TSWL,DSWL
     10 FORMAT(2X,4I2,6I6)
C
```

```
C     **  COMPUTE COMPOSITE SEA AND SWELL WAVE HEIGHT, 1ST RECORD  **
C
      H=INT(SQRT(FLOAT(HSEA)**2+FLOAT(HSWL)**2))
      IF(H.LT.H1) GO TO 30
      J=1
      K=1
C
C     **  SET COMPOSITE PERIOD AND DIRECTION TO THAT OF SEA OR     **
C     **  SWELL, WHICHEVER HAS A HIGHER INCREMENTAL WAVE HEIGHT    **
C
      IF(HSEA.GT.HSWL) GO TO 15
      T=TSWL
      D=DSWL
      GO TO 20
   15 T=TSEA
      D=DSEA
   20 WRITE(7,25) K,YR,MO,DY,TM,H,T,D
   25 FORMAT(2X,I4,1X,4I2,3I6)
      YRP=YR
      MOP=MO
      DYP=DY
      TMP=TM
   30 READ(1,10,END=200) YR,MO,DY,TM,HSEA,TSEA,DSEA,HSWL,TSWL,DSWL
C
C     **  COMPUTE COMPOSITE SEA AND SWELL WAVE HEIGHT               **
C
      H=INT(SQRT(FLOAT(HSEA)**2+FLOAT(HSWL)**2))
   32 IF(H.LT.H1) GO TO 190
      J=J+1
C
C     **  SET COMPOSITE PERIOD AND DIRECTION TO THAT OF SEA OR     **
C     **  SWELL, WHICHEVER HAS A HIGHER INCREMENTAL WAVE HEIGHT    **
C
      IF(HSEA.GT.HSWL) GO TO 35
      T=TSWL
      D=DSWL
      GO TO 40
   35 T=TSEA
      D=DSEA
   40 YRDIFF=YR-YRP
      MODIFF=MOP-MO
      DYDIFF=DYP-DY
      TMDIFF=TMP-TM
C
C       ***  CHECK FOR CONSECUTIVE RECORDS (SAME STORM)  ***
C
C      **    CONSECUTIVE RECORDS, SAME DAY    **
      IF(YRDIFF.EQ.0.AND.MODIFF.EQ.0.AND.DYDIFF.EQ.0.AND.
     &TMDIFF.EQ.-3) GO TO 45
C       ** CONSECUTIVE RECORDS, DAY END        **
       IF(YRDIFF.EQ.0.AND.MODIFF.EQ.0.AND.DYDIFF.EQ.-1.AND.
      &TMDIFF.EQ.21) GO TO 45
C       **    CONSECUTIVE RECORDS, MONTH END     **
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      IF(YRDIFF.EQ.0.AND.MODIFF.EQ.-1.AND.DYDIFF.EQ.27.AND.
     &TMDIFF.EQ.21) GO TO 45
      IF(YRDIFF.EQ.0.AND.MODIFF.EQ.-1.AND.DYDIFF.EQ.28.AND.
     &TMDIFF.EQ.21) GO TO 45
      IF(YRDIFF.EQ.0.AND.MODIFF.EQ.-1.AND.DYDIFF.EQ.29.AND.
     &TMDIFF.EQ.21) GO TO 45
      IF(YRDIFF.EQ.0.AND.MODIFF.EQ.-1.AND.DYDIFF.EQ.30.AND.
     &TMDIFF.EQ.21) GO TO 45
C       **   CONSECUTIVE RECORDS, YEAR END      **
      IF(YRDIFF.EQ.1.AND.MODIFF.EQ.11.AND.DYDIFF.EQ.30.AND.
     &TMDIFF.EQ.21) GO TO 45
C
C       ***  CHECK FOR RECORDS 6 HRS APART AND ADJUST RECORD  ***
C       ***  BETWEEN SUCH THAT THE PROGRAM SEES ONE CONT-     ***
C       ***  INUOUS STORM (IGNORING THE ONE RECORD BELOW      ***
C       ***  THE THRESHOLD)                                   ***
C
C       **    RECORDS 6 HRS APART, SAME DAY    **
      IF(YRDIFF.EQ.0.AND.MODIFF.EQ.0.AND.DYDIFF.EQ.0.AND.
     &TMDIFF.EQ.-6) GO TO 47
C        ** RECORDS 6 HRS APART, DAY END        **
      IF(YRDIFF.EQ.0.AND.MODIFF.EQ.0.AND.DYDIFF.EQ.-1.AND.
     &TMDIFF.EQ.18) GO TO 47
C       **   RECORDS 6 HRS APART, MONTH END     **
      IF(YRDIFF.EQ.0.AND.MODIFF.EQ.-1.AND.DYDIFF.EQ.27.AND.
     &TMDIFF.EQ.18) GO TO 47
      IF(YRDIFF.EQ.0.AND.MODIFF.EQ.-1.AND.DYDIFF.EQ.28.AND.
     &TMDIFF.EQ.18) GO TO 47
      IF(YRDIFF.EQ.0.AND.MODIFF.EQ.-1.AND.DYDIFF.EQ.29.AND.
     &TMDIFF.EQ.18) GO TO 47
      IF(YRDIFF.EQ.0.AND.MODIFF.EQ.-1.AND.DYDIFF.EQ.30.AND.
     &TMDIFF.EQ.18) GO TO 47
C       **   RECORDS 6 HRS APART, YEAR END      **
      IF(YRDIFF.EQ.1.AND.MODIFF.EQ.11.AND.DYDIFF.EQ.30.AND.
     &TMDIFF.EQ.18) GO TO 47
      K=K+1
      GO TO 45
   47 BACKSPACE 1
      BACKSPACE 1
      READ(1,10) YR,MO,DY,TM,HSEA,TSEA,DSEA,HSWL,TSWL,DSWL
      H=H1
      GO TO 32
   45 WRITE(7,25) K,YR,MO,DY,TM,H,T,D
      YRP=YR
      MOP=MO
      DYP=DY
      TMP=TM
  190 GO TO 30
C
C   **  FILE CODE 7 INCLUDES RECORDS WHERE H IS GREATER THAN   **
C   **  THE FIRST WAVE HEIGHT THRESHOLD H1.  CONSECUTIVE        **
C   **  RECORDS SHARE A COMMON "STORM NUMBER", K.               **
C
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```
C
C   **  READ FILE OF STORMS , COMPUTE DURATIONS AND    **
C   **    IDENTIFY PEAK CONDITIONS                      **
C
  200 RECNO=J
      REWIND 7
      DN=0
      HPK=0
      J=1
   95 READ(7,97,END=400) STNO,DATM,H,T,D
   97 FORMAT(2X,I4,1X,I8,3I6)
      IF(STNO.EQ.J) GO TO 350
      DUR(J)=DN
      DTPEAK(J)=DTPK
      HPEAK(J)=HPK
      TPEAK(J)=TPK
      DPEAK(J)=DPK
      J=J+1
      DN=0
      HPK=0
  350 DN=DN+3
      IF(H.LE.HPK) GO TO 95
      DTPK=DATM
      HPK=H
      TPK=T
      DPK=D
      GO TO 95
  400 DUR(J)=DN
      DTPEAK(J)=DTPK
      HPEAK(J)=HPK
      TPEAK(J)=TPK
      DPEAK(J)=DPK
      POISSON=FLOAT(J)/FLOAT(NOYRS)
      PERCENT=RECNO*100./NOREC
      TMIN=DUR(1)
      HPKMIN=HPEAK(1)
      HPKMAX=H1
      TMAX=3
      TSUM=0.0
      HPKSUM=0.0
      DO 700 I=1,J
      TSUM=TSUM+FLOAT(DUR(I))
      HPKSUM=HPKSUM+FLOAT(HPEAK(I))
      IF(HPEAK(I).LT.HPKMIN) HPKMIN=HPEAK(I)
      IF(HPEAK(I).GT.HPKMAX) HPKMAX=HPEAK(I)
      IF(DUR(I).LT.TMIN) TMIN=DUR(I)
      IF(DUR(I).GT.TMAX) TMAX=DUR(I)
  700 CONTINUE
      HPKMEAN=HPKSUM/FLOAT(J)
      TMEAN=TSUM/FLOAT(J)
      TDIFFSUM=0.0
      HDIFSUM=0.0
      DO 710 I=1,J
```

```
      HDIFSQ=(FLOAT(HPEAK(I))-HPKMEAN)**2
      HDIFSUM=HDIFSUM+HDIFSQ
      TDIFSQ=(FLOAT(DUR(I))-TMEAN)**2
      TDIFFSUM=TDIFFSUM+TDIFSQ
  710 CONTINUE
      STDEVT=SQRT(TDIFFSUM/FLOAT(J-1))
      STDEVH=SQRT(HDIFSUM/FLOAT(J-1))
C
C   **    PRINT TABLE OF STORM PARAMETERS   **
C
      WRITE(6,440) FNAME
  440 FORMAT(1H1,///,25X,"ANALYSIS OF STORM DURATION",
     &//,8X,"DATA FILE:  ",A64)
      WRITE(6,450) H1
  450 FORMAT(//,1X,"STORM NO.",2X,"DATE/TIME OF PEAK ",2X,
     &"DURATION H>",I3,2X,"PEAK H",2X,"PEAK T",2X,"PEAK DIR",/)
      DO 500 L=1,J
      WRITE(6,470) L,DTPEAK(L),DUR(L),HPEAK(L),TPEAK(L),DPEAK(L)
      WRITE(9,470) L,DTPEAK(L),DUR(L),HPEAK(L),TPEAK(L),DPEAK(L)
  470 FORMAT(4X,I3,10X,I8,8X,I4,12X,I3,6X,I2,6X,I3)
  500 CONTINUE
      WRITE(6,510) POISSON
  510 FORMAT(/,4X,F5.2," STORMS PER YEAR")
      WRITE(6,28) H1,RECNO,PERCENT,NOREC
   28 FORMAT(/,4X,"NO. RECORDS WHERE H > ",I3," = ",I5,
     &" (",F4.1,"% OF ",I5," RECORDS)")
      WRITE(6,719) HPKMIN,HPKMAX,HPKMEAN,STDEVH
  719 FORMAT(/,4X,"MIN. PEAK H = ",I3,"   MAX. = ",I4,
     &"   MEAN = ",F5.1,"   STD. DEV. = ",F5.1)
      WRITE(6,720) TMIN,TMAX,TMEAN,STDEVT
  720 FORMAT(/,4X,"MIN. DURATION = ",I3,"   MAX. = ",I3,
     &"   MEAN = ",F5.1,"   STD. DEV. = ",F5.1)
      WRITE(6,475)
  475 FORMAT(//,4X,"THE DATE/TIME IS YRMODYHR, DURATION IS IN HOURS,
     &H (HEIGHT) IS IN CM,",/,4X,"T (PERIOD) IS IN SEC AND DIRECTION",
     &" IS IN DEGREES RELATIVE TO THE SHORELINE")
      VARIABLE='PEAK H  '
      CALL PROBDIST (VARIABLE,J,HPEAK,POISSON)
      VARIABLE='DURATION'
      CALL PROBDIST (VARIABLE,J,DUR,POISSON)
      GO TO 620
  900 PRINT 901
  901 FORMAT(1X,19HATTACH UNSUCCESSFUL)
      CALL DETACH(01,,)
  620 STOP
      END
C
C
C
      SUBROUTINE PROBDIST (VARIABLE,N,HS,LAMBDA)
C  SUBROUTINE PROBDIST ADAPTED 1/86 BY ORSON P. SMITH FROM
C  PROGRAM "WAVDIST1".  11/85 VERSION BY ROBERT B. LUND
C  DESIGN BRANCH-COASTAL ENGINEERING RESEARCH CENTER
```

```
C  U.S. ARMY ENGINEERS WATERWAYS EXPERIMENT STATION
C  P.O. BOX 631
C  VICKSBURG, MS  39180-0631
C  FOR FURTHER INFORMATION CONCERNING THE APPLICATION
C  OF "WAVDIST1", CALL....
C  ROBERT B. LUND (601)-634-2068 FTS:2068
C  ORSON P. SMITH (601)-634-2013  FTS:542-2013
C DOYLE L. JONES (601)-634-2069  FTS:542-2069
C
C FORTRAN 4       HONEYWELL DPS-8
C REF: "RELIABILITY OF LONG-TERM WAVE CONDITIONS PREDICTED WITH DATA SETS
C       OF SHORT DURATION" CETN-I-5
C REF: "HANDBOOK OF MATHEMATICAL FUNCTIONS" BY ABRAMOWITZ AND SEGUN
C REF: "EXTREMAL PREDICTION IN WAVE CLIMATOLOGY" BY BORGMAN AND RESIO
C  REF. "LONG-TERM DISTRIBUTIONS OF OCEAN WAVES"
C       ISAACSON AND MACKENZIE
C
C      N = NUMBER OF STORMS
C    RET = RETURN PERIOD
C LAMBDA = POISSON LAMBDA PARAMETER (AVERAGE NO. STORMS PER YEAR)
C     HS = THE INDEPENDENT VARIABLE
C   DIFF = THE RESIDUAL FOR EACH DATA POINT
C   YACT = THE PROBABILITY AS ESTIMATED BY THE PLOTTING FORMULA M/K+1
C   YEST = THE PROBABILITY AS ESTIMATED BY THE DISTRIBUTION
C  ALPHA = THE ARRAY OF LOCATION PARAMETERS FOR THE DISTRIBUTIONS
C   BETA = THE ARRAY OF SCALE PARAMETERS FOR THE DISTRIBUTIONS
C      A = THE SLOPE OF EACH "PLOTTED LINE"
C      B = THE Y-INTERCEPT OF EACH "PLOTTED LINE"
C      C = THE ARRAY OF COEFFICIENTS FOR THE GAMMA INTEGRAL EXPANSION
C     ST = THE SUM OF THE SQUARE RESIDUALS
C   CORR = THE NON-LINEAR CORRELATION FOR EACH DISTRIBUTION
C    STE = THE STANDARD ERROR OF THE ESTIMATE OF Y ON X
C    MSD = THE MEAN SQUARE DEVIATION

C  DECLARATION OF VARIABLES, FUNCTIONS, AND CHARACTERS
        DIMENSION YACT(999,3),YEST(999,3),DUM1(999),DUM2(999),HS(999)
        DIMENSION YAVG(3),CORR(3),ALPHA(4),BETA(4),VAR(4),DM(3)
        DIMENSION RET(5),CHS(5,3),A(3),B(3),ST(3),SB(3),STE(3)
        DIMENSION STDEV(3)
        REAL MEAN(3),MSD(3)
        REAL LAMBDA
      INTEGER HS

        F1(X)=EXP(-EXP(-(X-EPSI)/PHI))
        F2(X)=1.0-EXP((-(X/SIGMA)**C))
        F3(X)=EXP(-((SIGMA2/X)**U))

        CHARACTER*20 IFLAG(4)
        CHARACTER*17 DEF
        CHARACTER*34 FORM(3)
        CHARACTER*24 TITLE
        CHARACTER*1 LOGIC
        CHARACTER*60 BOX(16)
```

```
      CHARACTER*8 VARIABLE
C  INITIALIZATION OF STRINGS AND CONSTANTS
      IFLAG(1)='EXTREMAL TYPE I'
      IFLAG(2)='WEIBULL'
      IFLAG(3)='LOG EXTREMAL'
      DEF='F(x)=Pr(X<x)= '
      FORM(1)='EXP(-EXP(-(x-EPSI)/PHI))'
      FORM(2)='1-EXP(-(x/BETA)**ALPHA)'
      FORM(3)='EXP(-(BETA/x)**ALPHA)'
      TITLE='LEAST SQUARES RESULTS - '

      DATA RET /5.0,10.0,25.0,50.0,100.0/
      EULER=.5772156649
      C2=.7796968

C
C    **  SET LOGIC = 'Y' FOR PRINTOUT OF RESIDUAL TABLES  **
C
    LOGIC='N'
C  RANK DATA AND ASSIGN A PROB. OF NON-EXCEEDENCE TO EACH
      CALL ORDER(HS,N)
      DO 25 I=1,N
      DO 25 K=1,3
      YACT(I,K)=FLOAT(I)/FLOAT(N+1)
   25 CONTINUE

C  INITIALIZE VARIABLES FOR LEAST SQUARES FIT OF THE DISTRIBUTIONS
      SX=0
      SY=0
      SXX=0
      SLX=0
      SLLY=0
      SLXX=0
      SLLQY=0
      SXLLY=0
      SLXLLY=0
      TOOBIG=0

C  CALCULATE SUMS FOR THE LEAST SQUARES METHOD
      DO 40 J=1,N
      SX=SX+HS(J)
      SY=SY+YACT(J,1)
      SXX=SXX+HS(J)**2
      SLX=SLX+ALOG(HS(J))
      SLXX=SLXX+(ALOG(HS(J)))**2
      SLLY=SLLY-ALOG(-ALOG(YACT(J,1)))
      SLLQY=SLLQY+ALOG(-ALOG(1.0-YACT(J,1)))
      SXLLY=SXLLY-HS(J)*ALOG(-ALOG(YACT(J,1)))
      SLXLLY=SLXLLY-ALOG(HS(J))*ALOG(-ALOG(YACT(J,1)))
 40   TOOBIG=TOOBIG+ALOG(HS(J))*(ALOG(-ALOG(1.0-YACT(J,1))))

C  CALCULATE SLOPE AND INTERCEPT OF EACH "PLOTTED LINE"
      A(1)=(N*SXLLY-SX*SLLY)/(N*SXX-SX**2)
```

```
      A(2)=(N*TOOBIG-SLX*SLLQY)/(N*SLXX-SLX**2)
C     A(3)=(N*SLXLLY-SLX*SLLY)/(N*SLXX-SLX**2)
      B(1)=(SXX*SLLY-SXLLY*SX)/(N*SXX-SX**2)
      B(2)=(SLXX*SLLQY-TOOBIG*SLX)/(N*SLXX-SLX**2)
C     B(3)=(SLXX*SLLY-SLXLLY*SLX)/(N*SLXX-SLX**2)
C CALCULATE PARAMETERS OF EACH DISTRIBUTION FROM SLOPE AND INTERCEPT DATA
      PHI=1.0/A(1)
      EPSI=-B(1)/A(1)
      C=A(2)
      SIGMA=EXP(-B(2)/A(2))
C     U=A(3)
C     SIGMA2=EXP(-B(3)/A(3))

C ASSIGN ARRAYS ALPHA AND BETA THE PARAMETERS OF EACH DISTRIBUTION
C FOR EASY PRINTOUT OF DATA
      ALPHA(1)=EPSI
      BETA(1)=PHI
      ALPHA(2)=C
      BETA(2)=SIGMA
C     ALPHA(3)=U
C     BETA(3)=SIGMA2
C CALCULATE PROBABILITY AS ESTIMATED BY DISTRIBUTION
      DO 100 J=1,N
      YEST(J,1)=F1(HS(J))
      YEST(J,2)=F2(HS(J))
C     YEST(J,3)=F3(HS(J))
  100 CONTINUE

C CALCULATE AVERAGE PROBABILITY AND CORRELATION COEFFICIENTS
      DO 110 K=1,2
      YAVG(K)=SY/FLOAT(N)
      MSD(K)=0
      ST(K)=0
  110 SB(K)=0

      DO 120 K=1,2
      DO 130 I=1,N
      ST(K)=ST(K)+(YACT(I,K)-YEST(I,K))**2
  130 SB(K)=SB(K)+(YACT(I,K)-YAVG(K))**2
      IF( (1.0-ST(K)/SB(K)) .LT. 0) CORR(K)=0.
      IF( (1.0-ST(K)/SB(K)) .LT. 0) GO TO 125
      CORR(K)=SQRT(1.0-ST(K)/SB(K))
  125 IF( N .EQ. 2) GO TO 120
      STE(K)=SQRT(ST(K)/(N-2))
  120 CONTINUE

C CALCULATE DATA FOR RETURN PERIOD TABLES
      DO 57 J=1,5
      PROB=1.0-1.0/(LAMBDA*RET(J))
      IF(PROB .LE. 0) PROB=.0000001
      CHS(J,1)=-ALOG(-ALOG(PROB))*PHI+EPSI
      CHS(J,2)=(-ALOG(1.0-PROB))**(1.0/C)*SIGMA
C     CHS(J,3)=SIGMA2/((-ALOG(PROB))**(1.0/U))
```

```
   57 CONTINUE

C CALCULATE MEAN SQUARE DEVIATION FOR EACH DISTRIBUTION
        DO 58 I=1,N
        Z1=YACT(I,1)
        Z2=EPSI-(ALOG(-ALOG(Z1)))*PHI
        Z3=BETA(2)*((-ALOG(1-Z1))**(1.0/ALPHA(2)))
C       Z4=BETA(3)/((-ALOG(Z1))**(1.0/ALPHA(3)))
        MSD(1)=MSD(1)+(Z2-HS(I))**2
        MSD(2)=MSD(2)+(Z3-HS(I))**2
C       MSD(3)=MSD(3)+(Z4-HS(I))**2
   58 CONTINUE
        MSD(1)=MSD(1)/(N*PHI**2)
        MSD(2)=MSD(2)/(N*BETA(2)**2)
C       MSD(3)=MSD(3)/(N*BETA(3)**2)

C  CALCULATE MEAN AND VARIANCE FOR EACH DISTRIBUTION
        MEAN(1)=EPSI+EULER*PHI
        VAR(1)=1.6449341*PHI**2
        PARA=1.0+1.0/C
        CALL GAMMA(PARA,WME)
        MEAN(2)=SIGMA*WME
        FAC1=SIGMA**2*WME**2
        PARA=1.0+2.0/C
        CALL GAMMA(PARA,WV2)
        FAC2=SIGMA**2*WV2
        VAR(2)=FAC2-FAC1
C       PARA=1.0-1.0/U
C       CALL GAMMA(PARA,HPC)
C       MEAN(3)=SIGMA2*HPC
C       PARA=1.0-2.0/U
C       CALL GAMMA(PARA,HPD)
C       VAR(3)=SIGMA2**2*HPD-MEAN(3)**2

C  WRITE OUT THE DATA FOR EACH DISTRIBUTION
      WRITE(6,136)
  136 FORMAT(1H1)
        WRITE(6,135) TITLE,VARIABLE
 135    FORMAT(///,16X,A26,A8,///)
        DO 150 K=1,2
      STDEV(K)=SQRT(VAR(K))
        WRITE(6,160) IFLAG(K),DEF,FORM(K)
 160    FORMAT(15X,A30,//,1X,A17,2X,A34)
        IF( K .EQ. 1) WRITE(6,159) EPSI,PHI
 159    FORMAT(1X,"EPSI="6X,F10.3,/,1X,"PHI=",7X,F10.3)
        IF( K .GT. 1) WRITE(6,161) ALPHA(K),BETA(K)
 161    FORMAT(1X,"ALPHA=",6X,F10.3,/,1X,"BETA=",6X,F10.3)
        WRITE(6,162) MEAN(K),VAR(K),STDEV(K)
 162    FORMAT(1X,"MEAN=",6X,F10.3,/,1X,"VARIANCE=",2X,F10.3,
     &/,1X,"STD. DEV. = ",2X,F7.3)
        IF ( LOGIC .EQ. 'N') GO TO 171
        DO 170 I=1,N
        DUM1(I)=YACT(I,K)
```

```
 170     DUM2(I)=YEST(I,K)
         L2=N
         CALL RESIDUAL(HS,DUM1,DUM2,L2)
 171     WRITE(6,163) CORR(K),ST(K)
 163     FORMAT(/,1X,"NON-LINEAR CORRELATION IS",5X,F10.7,/
      &  ,1X,"SUM SQUARE RESIDUALS IS",6X,F11.7)
         IF( N .EQ. 2 ) GO TO 167
         WRITE(6,164) STE(K)
 164     FORMAT(1X,"STANDARD ERROR IS",13X,F10.7)
 167     WRITE(6,166) MSD(K)
 166     FORMAT(1X,"MEAN SQUARE DEVIATION IS",6X,F10.7,///)
 207     WRITE(6,208) VARIABLE
 208     FORMAT(7X,"RETURN PERIOD TABLE",/,6X,"YEAR",13X,A8)
         DO 211 J=1,5
         WRITE(6,212) RET(J),CHS(J,K)
 212     FORMAT(1X,F9.2,8X,F9.2)
 211     CONTINUE
         WRITE(6,165)
 165     FORMAT(////)
 150     CONTINUE
         RETURN
         END




C SUBROUTINE TO PUT NUMBERS IN ORDER BY ASCENDING X
         SUBROUTINE ORDER(X,N)
         DIMENSION X(N)
       INTEGER X,TX
         DO 20 K=2,N
         J=N-K+2
         DO 10 I=1,J-1
         IF( X(I) .LT. X(I+1)) GO TO 10
         TX=X(I)
         X(I)=X(I+1)
         X(I+1)=TX
 10      CONTINUE
 20      CONTINUE
         RETURN
         END



C  SUBROUTINE TO HELP PRINT OUT DATA
         SUBROUTINE RESIDUAL(X,YACT,YEST,N)
         DIMENSION X(N),YACT(N),YEST(N),DIFF(200)
       INTEGER X
         SSR=0
         DO 10 I=1,N
         DIFF(I)=(YACT(I)-YEST(I))**2
 10      SSR=SSR+DIFF(I)
         WRITE(6,15)
```

```
15    FORMAT(//,1X,"    XVALUE      YVALUE      YEST        DIFF ",/,)
      DO 25 I=1,N
      WRITE(6,20) X(I),YACT(I),YEST(I),SQRT(DIFF(I))
    WRITE(8,20) X(I),YACT(I),YEST(I),SQRT(DIFF(I))
20    FORMAT(1X,I11,F11.4,F11.4,F11.4,/,)
25    CONTINUE
      RETURN
      END


C  SUBROUTINE TO EVALUATE THE GAMMA FUNCTION
C  PROGRAM ADJUSTS ALPHA TO BE BETWEEN 1.0 AND 2.0
C  AND THEN MULTIPLIES BY GF TO COMPENSATE
      SUBROUTINE GAMMA(ALPHA,AREA)
      DOUBLE PRECISION C(25),SUM
      GF=1.0
      IF(ALPHA) 1,2,3

2     PRINT,'TROUBLE IN GAMMA'
      AREA=1.0
      GO TO 200

C  FOR GAMMA OF A POSITIVE NUMBER
3     M=INT(ALPHA)
      EPSI=ALPHA-FLOAT(M)
      IF( M .EQ. 0) GF=GF/ALPHA
      IF( M .EQ. 0) ALPHA=ALPHA+1.0
      IF( M .EQ. 0) GO TO 100
      IF( M .EQ. 1) GF=1.0
      IF( M .EQ. 1) GO TO 100
      DO 10 I=2,M
10    GF=GF*(FLOAT(I-1)+EPSI)
      ALPHA=1.0+EPSI
      GO TO 100

C  FOR GAMMA OF A NEGATIVE NUMBER
1     M=INT(ALPHA)
      EPSI=ALPHA-FLOAT(M)
      DO 20 I=1,2-M
      J=M+(I-1)
20    GF=GF/(EPSI+FLOAT(J))
      ALPHA=EPSI+2.0

C  COEFFICIENTS FOR SERIES EXPANSION OF THE GAMMA INTEGRAL
C  SEE HANDBOOK OF MATHEMATICAL FUNCTIONS BY ABRAMOWITZ AND SEGUN
100   C(1)=1.0000000000000000
      C(2)=.5772156649015329
      C(3)=-.6558780715202538
      C(4)=-.0420026350340952
      C(5)=.1665386113822915
      C(6)=-.0421977345555443
      C(7)=-.0096219715278887
      C(8)=.007218943246663
```

```
          C(9)=-.0011651675918591
          C(10)=-.0002152416741149
          C(11)=.0001280502823882
          C(12)=-.0000201348547807
          C(13)=-.0000012504934821
          C(14)=.0000011330272320
          C(15)=-.0000002056338417
          C(16)=6.116095E-09
          C(17)=5.0020075E-09
          C(18)=-1.1812746E-09
          C(19)=1.043427E-10
          C(20)=7.7823E-12
          C(21)=-3.69680E-12
          C(22)=5.1E-13
          C(23)=-2.06E-14
          C(24)=-5.4E-15
          C(25)=1.4E-15

C    SUM SERIES
          SUM=0.0
          DO 50 K=1,25
          SUM=SUM+C(K)*(ALPHA**K)
 50       AREA=GF/SUM
 200      RETURN
          END
$:EXECUTE
$:LIMITS:30,100K
$:FILE:07,X7R,5L,NEW,STRMFILE
$:FILE:08,X8R,5L,NEW,DISTFILE
$:FILE:09,X9R,5L,NEW,AB3DST30
$:ENDJOB
```

ANALYSIS OF STORM DURATION

DATA FILE NAGSHEAD, NORTH CAROLINA

STORM NO.	DATE/TIME OF PEAK	DURATION H>350	PEAK H	PEAK T	PEAK DIR
1	56011015	27	449	11	98
2	56092718	6	379	10	101
3	56102703	3	461	11	83
4	56102800	9	405	11	75
5	56103100	3	377	8	69
6	56103109	6	364	11	74
7	58102103	3	352	10	91
8	58102118	9	488	10	75
9	58102212	27	508	11	88
10	60020100	9	386	10	93
11	60020118	3	351	10	101
12	60103121	3	354	10	97
13	61102421	6	387	11	99
14	62030721	18	459	11	114
15	62030909	30	591	13	94
16	62112806	84	466	12	97
17	62120200	3	351	9	88
18	62120212	15	371	9	88
19	63020421	6	363	9	88
20	64092221	21	391	10	111
21	66061306	3	403	10	105
22	68011121	12	400	10	100
23	68022512	15	385	10	116
24	69022109	3	355	10	114
25	69030306	3	360	11	101
26	70102718	12	364	10	103
27	72052700	15	365	10	94
28	73021112	33	465	11	111
29	73021300	3	372	10	101
30	73022806	9	379	10	107
31	73120906	3	352	9	66
32	75012118	6	389	10	106
33	75070106	9	399	10	112
34	75070215	9	430	11	104
35	75112415	9	375	10	107
36	75112506	3	397	10	102

1.80 STORMS PER YEAR

NO. RECORDS WHERE H > 350 = 149 (0.3% OF 58440 RECORDS)

MIN. PEAK H = 351 MAX. = 591 MEAN = 401.2 STD. DEV. = 53.4

MIN. DURATION = 3 MAX. = 84 MEAN = 12.2 STD. DEV. = 14.9

THE DATE/TIME IS YRMODYHR, DURATION IS IN HOURS, H (HEIGHT) IS IN CM, T (PERIOD) IS IN SEC AND DIRECTION IS IN DEGREES RELATIVE TO THE SHORELINE

LEAST SQUARES RESULTS - DURATION

EXTREMAL TYPE I

F(X)=PR(X<X)= EXP(-EXP(-(X-EPSI)/PHI))

EPSI=	3.918
PHI=	15.246
MEAN=	12.718
VARIANCE=	382.333
STD. DEV. =	19.553

NON-LINEAR CORRELATION IS	0.8720603
SUM SQUARE RESIDUALS IS	0.6796928
STANDARD ERROR IS	0.1413894
MEAN SQUARE DEVIATION IS	0.3527218

RETURN PERIOD TABLE

YEAR	DURATION
5.00	36.53
10.00	47.55
25.00	61.78
50.00	72.44
100.00	83.05

WEIBULL

F(X)=PR(X<X)= 1-EXP(-(X/BETA)**ALPHA)

ALPHA=	1.156
BETA=	12.636
MEAN=	12.007
VARIANCE=	108.437
STD. DEV. =	10.413

NON-LINEAR CORRELATION IS	0.9607089
SUM SQUARE RESIDUALS IS	0.2186227
STANDARD ERROR IS	0.0801878
MEAN SQUARE DEVIATION IS	0.3858081

RETURN PERIOD TABLE

YEAR	DURATION
5.00	24.96
10.00	31.64
25.00	40.15
50.00	46.40
100.00	52.52

LEAST SQUARES RESULTS - PEAK H

EXTREMAL TYPE I

F(X)=PR(X<X)= EXP(-EXP(-(X-EPSI)/PHI))

EPSI=	374.975
PHI=	48.460
MEAN=	402.947
VARIANCE=	3862.962
STD. DEV. =	62.153

NON-LINEAR CORRELATION IS	0.9629306
SUM SQUARE RESIDUALS IS	0.2064946
STANDARD ERROR IS	0.0779318
MEAN SQUARE DEVIATION IS	0.0998112

RETURN PERIOD TABLE

YEAR	PEAK H
5.00	478.63
10.00	513.66
25.00	558.90
50.00	592.77
100.00	626.49

WEIBULL

F(X)=PR(X<X)= 1-EXP(-(X/BETA)**ALPHA)

ALPHA=	7.888
BETA=	426.388
MEAN=	401.273
VARIANCE=	3638.859
STD. DEV. =	60.323

NON-LINEAR CORRELATION IS	0.9038882
SUM SQUARE RESIDUALS IS	0.5192849
STANDARD ERROR IS	0.1235843
MEAN SQUARE DEVIATION IS	0.0047993

RETURN PERIOD TABLE

YEAR	PEAK H
5.00	471.13
10.00	487.80
25.00	505.13
50.00	515.95
100.00	525.41

APPENDIX D

SPSS COMMAND FILE AS APPLIED IN THE REGRESSION ANALYSIS

COMMAND FILE LISTING - SPSS REGRESSION ANALYSIS

```
10$$S,T,J
20$:IDENT:R0CDOPS,OPSMITH
30$:SELECT:SPSS/SPSS
40$:SYSOUT:43,NULL
50$:LIMITS:,60K
60$:INCODE:IBMF
70RUN NAME:DURATION ANALYSIS
80VARIABLE LIST:DUR,H,T,D
90INPUT MEDIUM:DISK
100INPUT FORMAT:FIXED(33X,F4.0,12X,F3.0,6X,F2.0,6X,F3.0)
110N OF CASES:UNKNOWN
120VAR LABELS:DUR DURATION/H PEAK H/T PEAK T/D PEAK DIR/
130COMPUTE:HSQ=H**2
140COMPUTE:TSQ=T**2
150COMPUTE:STP=H/(981*TSQ)
160COMPUTE:SEV=156.13*HSQ*TSQ
170VAR LABELS:HSQ H**2/TSQ T**2/STP H OVER gT**2/SEV LH**2
180REGRESSION:VARIABLES=DUR,H,HSQ,T,TSQ/
190::REGRESSION=DUR WITH H,HSQ,T,TSQ(1) RESID=0/
210STATISTICS:ALL
220READ INPUT DATA
230REGRESSION:VARIABLES=DUR,SEV/
240::REGRESSION=DUR WITH SEV(1) RESID=0/
260STATISTICS:ALL
270REGRESSION:VARIABLES=DUR,D/
280::REGRESSION=DUR WITH D(1) RESID=0/
300STATISTICS:ALL
310REGRESSION:VARIABLES=DUR,STP/
320::REGRESSION=DUR WITH STP(1) RESID=0/
340STATISTICS:ALL
350SCATTERGRAM:DUR WITH H,T,HSQ,STP,SEV
360STATISTICS:ALL
370FINISH
380$:DATA:08
390$$SELECT(P05DST50)
400$:ENDJOB
```

Sample Output (Excerpts) - SPSS Regression Analysis

```
DURATION ANALYSIS

FILE    NONAME    (CREATION DATE = 01/16/86)

* * * * * * * * * * * * * * * * * * * * * * *   M U L T I P L E   R E G R E S S I O N   * * * * * * * * * * * * *   VARIABLE LIST  1
                                                                                                                      REGRESSION LIST  1
DEPENDENT VARIABLE..    DUR        DURATION

VARIABLE(S) ENTERED ON STEP NUMBER  1..    HSQ        H**2

MULTIPLE R           0.66311               ANALYSIS OF VARIANCE     DF      SUM OF SQUARES      MEAN SQUARE            F
R SQUARE             0.43971               REGRESSION                1.        2251.38783       2251.38783      32.96099
ADJUSTED R SQUARE    0.42637               RESIDUAL                 42.        2868.79399         68.30462
STANDARD ERROR       8.26466

----------------- VARIABLES IN THE EQUATION ------------------       ------------- VARIABLES NOT IN THE EQUATION -------------

VARIABLE                B         BETA     STD ERROR B        F       VARIABLE      BETA IN     PARTIAL   TOLERANCE          F
HSQ               0.00025      0.66311       0.00004     32.961       H           -11.82259    -0.25047     0.00025      2.744
(CONSTANT)     -101.43962                                             T             0.17425     0.21827     0.87916      2.051
                                                                      TSQ           0.17425     0.21827     0.87916      2.051

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VARIABLE(S) ENTERED ON STEP NUMBER  2..    T          PEAK T

MULTIPLE R           0.68294               ANALYSIS OF VARIANCE     DF      SUM OF SQUARES      MEAN SQUARE            F
R SQUARE             0.46640               REGRESSION                2.        2388.06289       1194.03145      17.91843
ADJUSTED R SQUARE    0.44037               RESIDUAL                 41.        2732.11892         66.63705
STANDARD ERROR       8.16315

----------------- VARIABLES IN THE EQUATION ------------------       ------------- VARIABLES NOT IN THE EQUATION -------------

VARIABLE                B         BETA     STD ERROR B        F       VARIABLE      BETA IN     PARTIAL   TOLERANCE          F
HSQ               0.00027      0.72368       0.00005     35.378       H           -10.45404    -0.22409     0.00025      2.115
T                 1.88750      0.17425       1.31795      2.051       TSQ          -0.21407    -0.00000     0.00000      0.000
(CONSTANT)     -140.16131

F-LEVEL OR TOLERANCE-LEVEL INSUFFICIENT FOR FURTHER COMPUTATION
```

D4

DURATION ANALYSIS

FILE NONAME (CREATION DATE = 01/16/86)

CORRELATION COEFFICIENTS

A VALUE OF 99.00000 IS PRINTED
IF A COEFFICIENT CANNOT BE COMPUTED.

	DUR	H	HSQ	T	TSQ
DUR	1.00000	0.66005	0.66311	-0.07732	-0.07732
H	0.66005	1.00000	0.99987	-0.34993	-0.34993
HSQ	0.66311	0.99987	1.00000	-0.34762	-0.34762
T	-0.07732	-0.34993	-0.34762	1.00000	1.00000
TSQ	-0.07732	-0.34993	-0.34762	1.00000	1.00000

DURATION ANALYSIS

FILE NONAME (CREATION DATE = 01/16/86)

* M U L T I P L E R E G R E S S I O N * * * * * * * * * * * * * VARIABLE LIST 1
REGRESSION LIST 1

DEPENDENT VARIABLE.. DUR DURATION

VARIABLE(S) ENTERED ON STEP NUMBER 1.. SEV LH**2

		ANALYSIS OF VARIANCE	DF	SUM OF SQUARES	MEAN SQUARE	F
MULTIPLE R	0.26068	REGRESSION	1.	347.93038	347.93038	3.06209
R SQUARE	0.06795	RESIDUAL	42.	4772.25144	113.62503	
ADJUSTED R SQUARE	0.04576					
STANDARD ERROR	10.65950					

------------------ VARIABLES IN THE EQUATION ------------------

VARIABLE	B	BETA	STD ERROR B	F
SEV	0.00000	0.26068	0.00000	3.062
(CONSTANT)	-12.03434			

-------------- VARIABLES NOT IN THE EQUATION --------------

VARIABLE	BETA IN	PARTIAL	TOLERANCE	F

MAXIMUM STEP REACHED

DURATION ANALYSIS

FILE NONAME (CREATION DATE = 01/16/86)

* M U L T I P L E   R E G R E S S I O N * * * * * * * * * * * * * VARIABLE LIST 1
REGRESSION LIST 1

DEPENDENT VARIABLE.. DUR DURATION

VARIABLE(S) ENTERED ON STEP NUMBER 1.. D PEAK DIR

MULTIPLE R	0.23862	ANALYSIS OF VARIANCE	DF	SUM OF SQUARES	MEAN SQUARE	F
R SQUARE	0.05694	REGRESSION	1.	291.53490	291.53490	2.53580
ADJUSTED R SQUARE	0.03448	RESIDUAL	42.	4828.64692	114.96778	
STANDARD ERROR	10.72230					

------------------ VARIABLES IN THE EQUATION ------------------

VARIABLE	B	BETA	STD ERROR B	F
D	0.49102	0.23862	0.30835	2.536
(CONSTANT)	-32.47339			

------------- VARIABLES NOT IN THE EQUATION -------------

VARIABLE	BETA IN	PARTIAL	TOLERANCE	F

MAXIMUM STEP REACHED

D5

DURATION ANALYSIS

FILE NONAME (CREATION DATE = 01/16/86)

* M U L T I P L E   R E G R E S S I O N * * * * * * * * * * * * * VARIABLE LIST 1
REGRESSION LIST 1

DEPENDENT VARIABLE.. DUR DURATION

VARIABLE(S) ENTERED ON STEP NUMBER 1.. STP H OVER GT**2

MULTIPLE R	0.20950	ANALYSIS OF VARIANCE	DF	SUM OF SQUARES	MEAN SQUARE	F
R SQUARE	0.04389	REGRESSION	1.	224.72807	224.72807	1.92803
ADJUSTED R SQUARE	0.02113	RESIDUAL	42.	4895.45375	116.55842	
STANDARD ERROR	10.79622					

------------------ VARIABLES IN THE EQUATION ------------------

VARIABLE	B	BETA	STD ERROR B	F
STP	4986.47292	0.20950	3591.17557	1.928
(CONSTANT)	-4.62940			

------------- VARIABLES NOT IN THE EQUATION -------------

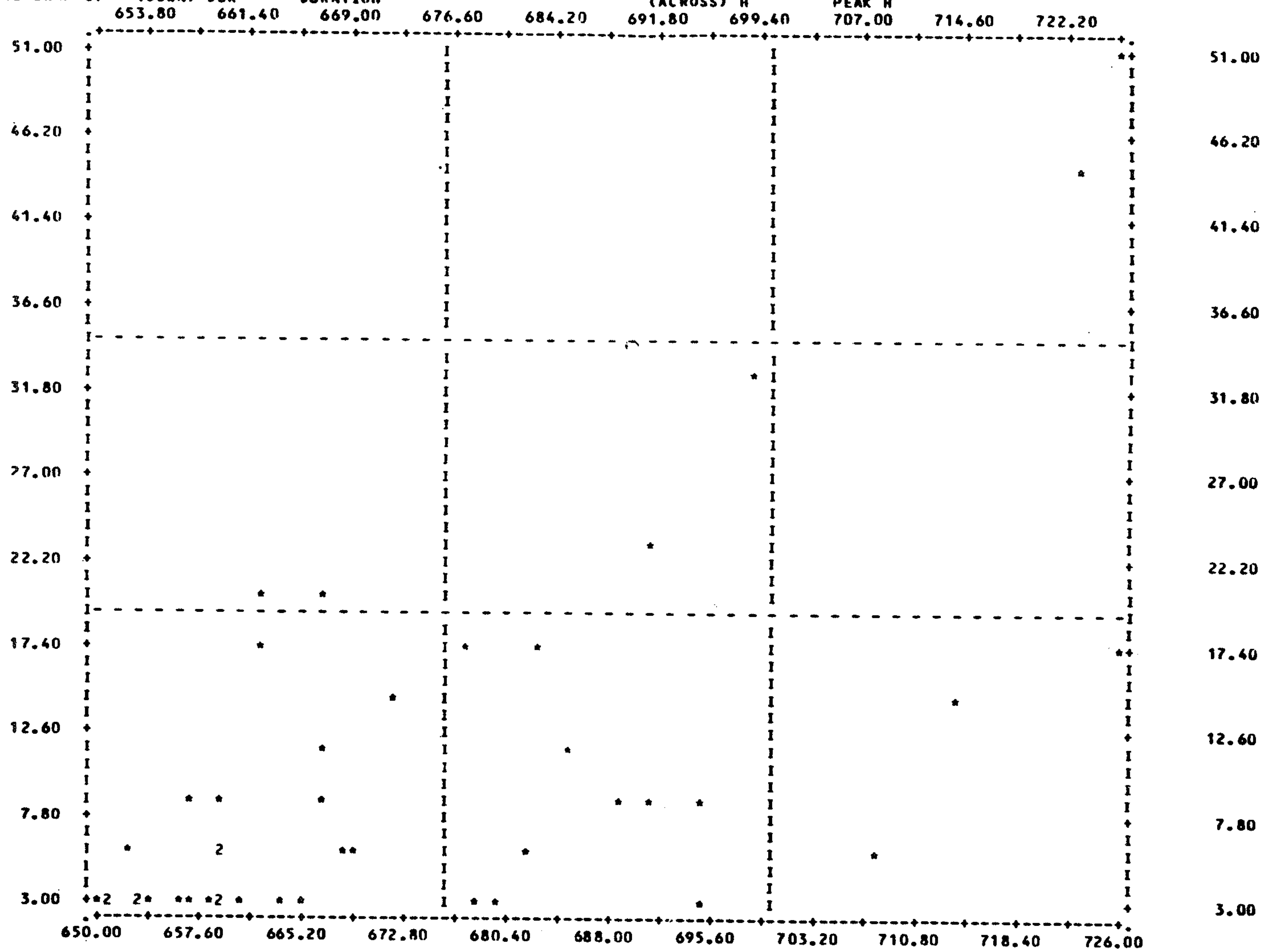
VARIABLE	BETA IN	PARTIAL	TOLERANCE	F

MAXIMUM STEP REACHED

D6

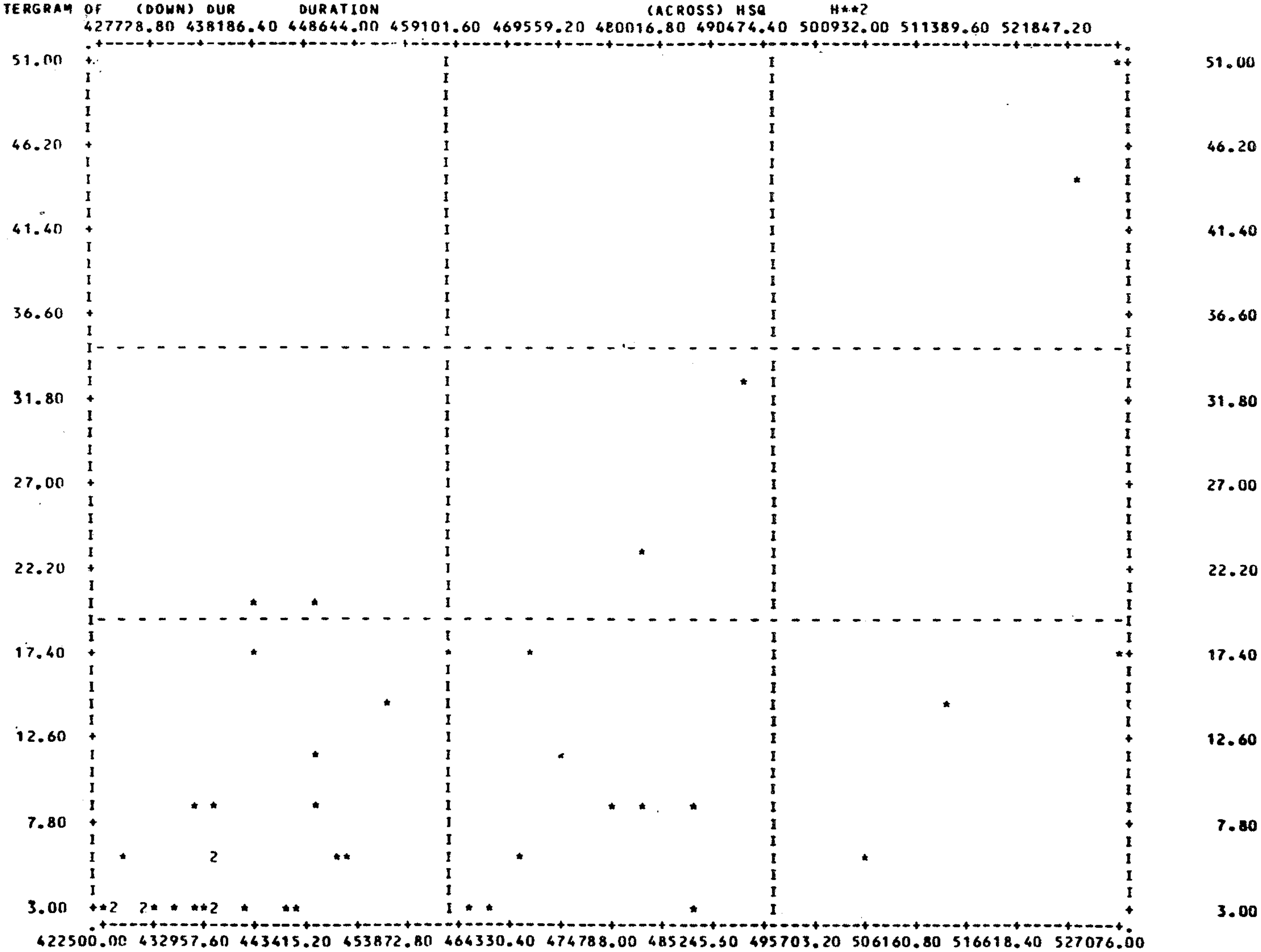
DURATION ANALYSIS

FILE NONAME (CREATION DATE = 0?/16/86)
SCATTERGRAM OF (DOWN) DUR DURATION (ACROSS) H PEAK H



DURATION ANALYSIS

FILE NONAME (CREATION DATE = 01/16/86)
SCATTERGRAM OF (DOWN) DUR DURATION (ACROSS) HSQ H**2



DJRATION ANALYSIS

FILE NONAME (CREATION DATE = 01/16/86)
SCATTERGRAM OF (DOWN) DUR DURATION (ACROSS) SEV LH**2