CHAPTER 7

GREAT LAKES WAVE INFORMATION

By

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I. Introduction.

The lack of reliable Great Lakes wave information has long been a problem for the Corps of Engineers and others involved in planning and design along Great Lakes coasts. In recent years this need has been accentuated by increased water levels and increased demand for coastal land use. The Corps need for wave information became critical with the passage of the River and Harbor Act of 1970 (Public Law 91-611) Section 123 of this legislation authorized design and construction of contained spoil disposal facilities having a ten-year capacity to hold polluted dredged material.

The North Central Division of the Corps of Engineers (NCD) is responsible for dredging 117 navigation projects and connecting channels in the Great Lakes. Of these, 59 are considered polluted, necessitating construction of 41 diked disposal sites at an estimated cost of over $300,000,000. With a program of this magnitude, it was apparent that unnecessary conservatism in design had to be minimized through development of the best wave information base that the state-of-the-art could provide.

A project was initiated at the Corps of Engineers Waterways Experiment Station (WES) under the sponsorship of NCD to supply design wave information. After a review of all possible sources of available wave information and the potential for obtaining additional gage data, it was determined that a wave hindcast program might best meet the immediate needs of NCD. The actual study has been divided into four phases:

a. The estimation of over-lake winds,
b. The establishment of a wave hindcast technique,
c. The analysis of waves from model outputs, and
d. The evaluation of errors in Phases a, b, and c.

II. Winds Over A Lake.

Cole (1967) evaluated three methods for estimating winds over Lake Michigan. He compared estimates from these three methods to winds observed at a tower located about a mile offshore from Muskegon, Michigan. Table 1 gives the results of these comparisons to the three techniques in terms of correlation coefficients. The method based on the reduction of geostrophic wind speeds to surface wind speeds produced the highest

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TABLE 1
WIND SPEED COMPARISONS BY COLE (1967)

TYPE OF COMPARISON	NUMBER OF DATA PAIRS	CORRELATION COEFFICIENT
BRETSCHNEIDER WINDS VS 10-M WINDS	36	0.63
JACOBS' 7.5-M WINDS VS 7.5-M WINDS	43	0.55
JACOBS' 19.5-M WINDS VS 16-M WINDS	49	0.37
RICHARDS' WINDS VS 16-M WINDS	44	0.36
RICHARDS' WINDS VS 10-M WINDS	36	0.24

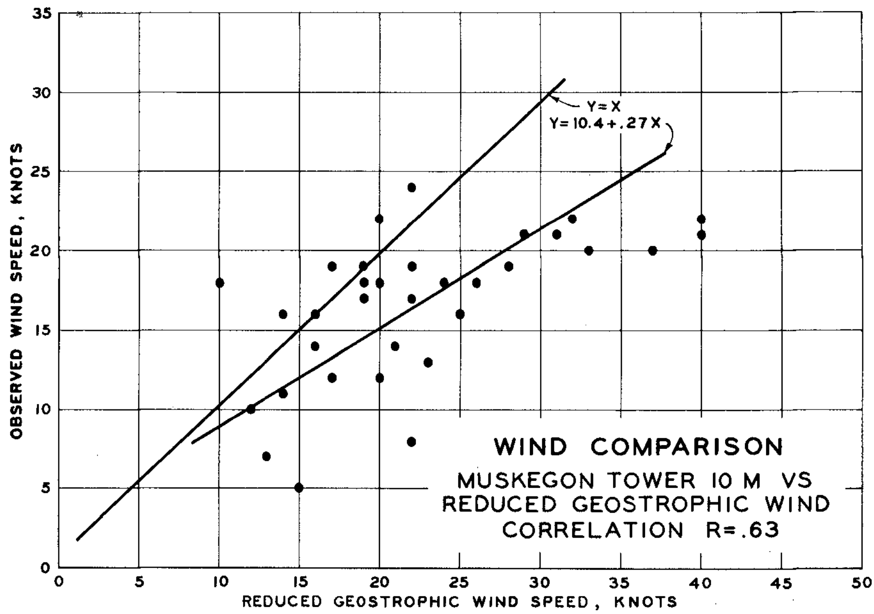


Figure 1 Comparison of wind speeds from Muskegon tower to wind speeds estimated by reduced geostrophic wind speed

correlation coefficient, 0.63. Methods based on the reduction of gradient winds to surface winds (Jacobs, 1965) and on the transformation of over-land wind speeds to over-water wind speeds produced substantially lower correlation coefficients. However, as shown in Figure 1, although the correlation coefficient is high for the geostrophic method there is a pronounced bias toward overestimation of wind speeds, particularly at high velocities. A detailed study of the relationship between geostrophic wind speeds and anemometer-level wind speeds over large lakes (Resio and Vincent, 1976) indicated that this bias is due to the lack of velocity-dependence in the constant used to reduce geostrophic velocities to surface velocities in this method.

During this study two methods were found to produce unbiased estimates of wind speeds over the Great Lakes--a method based on the transformation of geostrophic winds to surface winds using numerical techniques similar to those of Cardone (1969) and a method based on the transformation of wind speeds measured at land sites to over-water wind speeds. The latter of these methods was chosen for application to hindcasting due to its simplicity and the lack of long-term meterological records at a time and space scale suitable for obtaining accurate geostrophic level winds.

The derivation of the transformation coefficients is discussed in detail in an earlier paper (Resio and Vincent, 1976a). In this paper, the wind speed over water was related to the wind speed over land by

$$U_W = U_L \psi \Phi_n \overline{F} \tag{1}$$

where U_W is the over-water wind speed, U_L is the over-land wind speed, and ψ, Φ_n, and $\overline{F}$ are dimensionless conversion factors accounting for the effects of the velocity-dependence of lake surface roughness, air-sea temperature difference and over-water fetch, respectively. Figures 2 and 3 give the determined forms of Φ and Φ_n along with corroboratory evidence from other studyies for Lake Erie and Lake Ontario. The fetch factor was found to be approximated by

$$\overline{F} = \begin{cases} 0.411\ F^{0.297} & \text{for } F \leq 20 \text{ miles} \\ 1.0 & \text{for } F > 20 \text{ miles} \end{cases} \tag{2}$$

where F is the over-water fetch in miles. Whereas the relationships for ψ and Φ_n were based on more than 52,000 ships observations, the relationship for $\overline{F}$ is based on a limited number of observations tabulated by Richards et al. (1966) and should probably be regarded only as a first estimate.

Using Equation 1 to estimate winds over Lake Erie and Lake Ontario the rms error was found to be a function of velocity (Figure 4). As seen here, the error for wind speeds of importance to design and planning is beneath 5 knots.

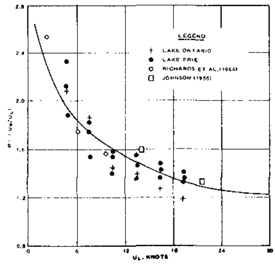


Figure 2. Variation in Ψ with observed land velocity

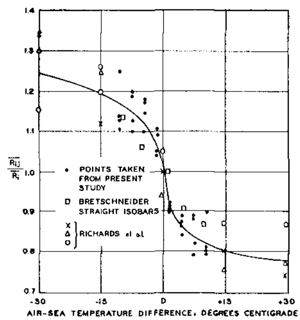


Figure 3. Variation in Φ_n with air-sea temperature difference

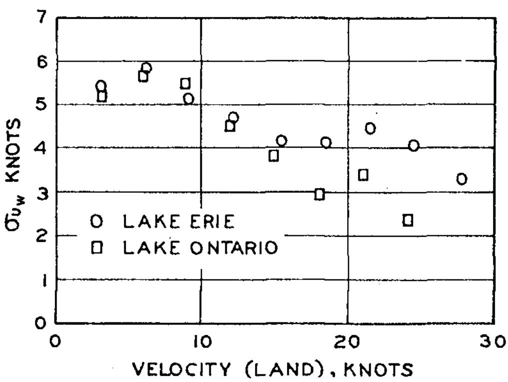


Figure 4. RMS error in wind speed estimate as a function of observed land wind speed

It can be shown (Resio, 1976) that the bias introduced by a random error in wind velocity is represented by

$$\frac{\hat{\bar{H}}}{\bar{H}} = \frac{(U^2 + \sigma_\varepsilon^2)}{U^2} \tag{3}$$

where $\hat{\bar{H}}$ is the estimated mean wave height for conditions with wind speed U, $\bar{H}$ is the actual mean wave height for conditions with wind speed U and σ_ε^2 is the variance of wind estimates around U. Figure 5 shows the value for $\hat{\bar{H}}/\bar{H}$ as a function of wind speed for an rms error of 5 knots. For wind speeds above 25 knots over the water, there is less than 5 percent bias due to the random error in wind estimates.

Further evidence that the wind transformation is reliable even during high wind conditions can be seen in Figures 6a, 6b, and 6c. These figures show comparisons of winds estimated from data obtained at Cleveland and Toledo airports during storms on Lake Erie and simultaneous observations of winds by ships in the lake.

III. The Wave Model.

Three hindcast techniques were evaluated for application to the Great Lakes. These are as follows:

a. The significant wave method contained in the Shore Protection Manual.

b. A numerical model of the growth and decay of wave spectra as formulated by Inoue (1966, 1967), Cardone (1969), and Pierson et al. (1966).

c. A numerical model of the growth and decay of wave spectra as formulated by Barnett (1966).

The first technique has been evaluated in previous studies by Cole (1967) and Brebner and Kennedy (1962) on Lake Michigan and Lake Ontario, respectively. Figure 7 shows the results of their comparisons along with some additional comparisons performed during this study for waves in Lake Ontario and Lake Erie. This figure indicates a pronounced tendency to over estimate wave heights in high wind conditions. Regression coefficients suggest that there is about a 35 percent bias in this method. However, since the winds used for hindcasting in this method appear to be too high (Figure 1), the bias might be more a function of errors in wind speed rather than a discrepancy in the wave hindcast curves. The standard deviation of the scatter about the regression line was 2.5 ft.

As indicated in the previous discussion, the lack of precise measure of wind speed over a fetch complicates the comparison of hindcast wave heights to observed wave heights. Therefore, instead of basing a decision regarding the optimal model for hindcasting in restricted fetches on such comparisons, the results of field efforts (Hasselman et al., 1973;

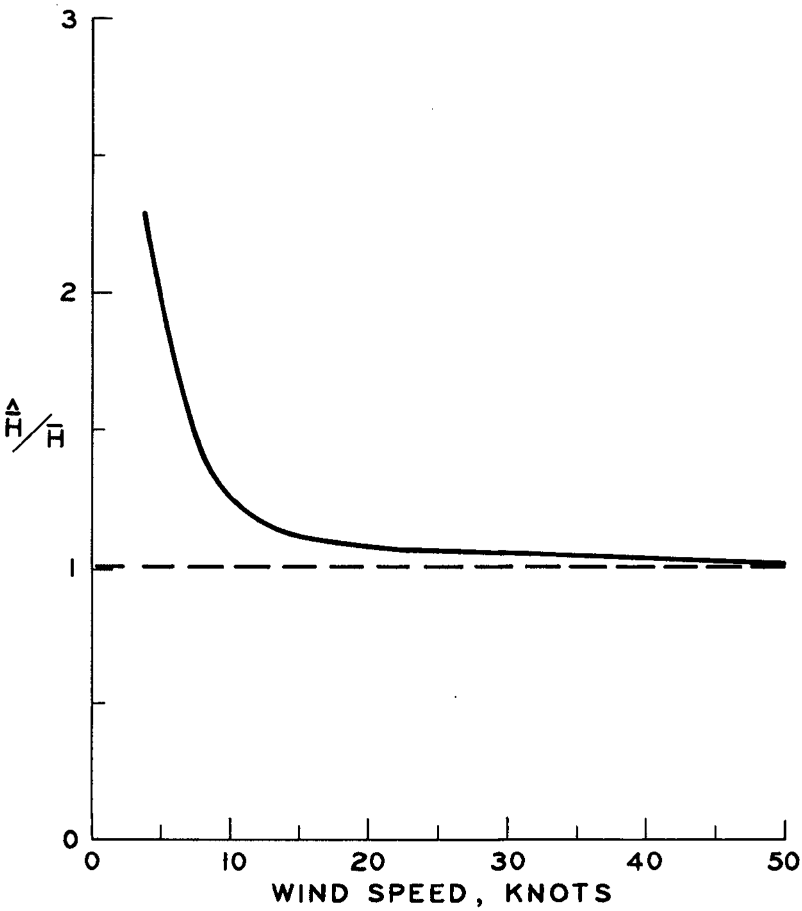
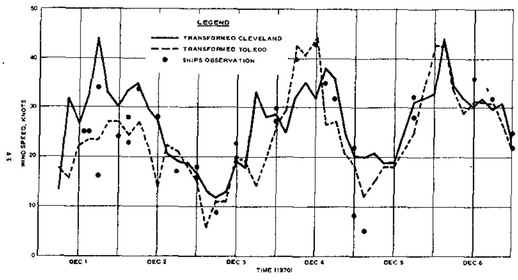
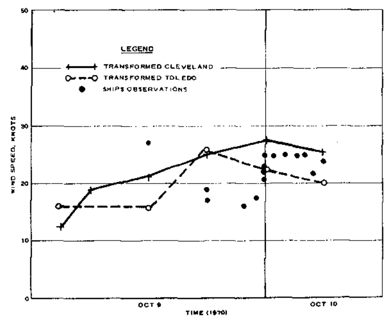


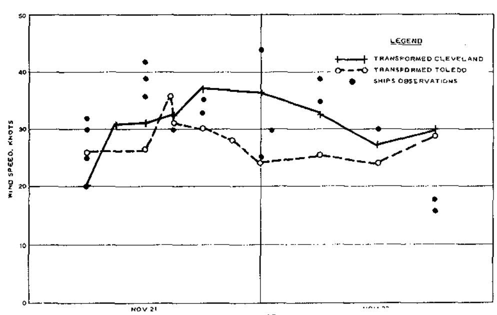
Figure 5. Estimate of bias due to random wind error



a.



b.



c.

Figure 6. Comparison of wind speeds by ships to **wind speeds** estimated by equation (1)

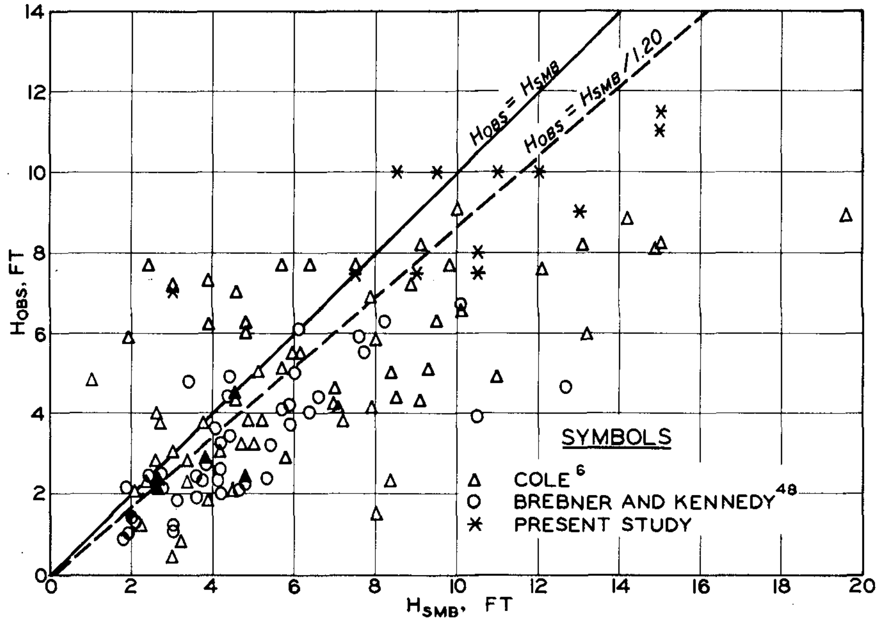


Figure 7. Composite chart showing results of comparison of observed wave heights and wave heights hindcast by SMB methods on the Great Lakes

Mitsuyasu, 1968), which formulated functional relationships between non-dimensional fetch and wave height parameters were used for comparisons. These studies have supported Kitaigorodskii's (1962) similarity theory for the development of wave spectra along a fetch. Figure 8 shows a plot of non-dimensional wave height, defined as

$$\tilde{H} = \frac{g\sqrt{E}}{U_*^2} \tag{4}$$

where U_* is the friction velocity of the wind, E is the total energy in the wave spectrum, and g is the acceleration due to gravity, versus nondimensional fetch defined as

$$\tilde{F} = \frac{gF}{U_*^2} \tag{5}$$

where F is the fetch measured in the same units as g and U_* . This figure indicates that none of the available models produce results in accord with recent observational evidence for limited fetches.

Of the three models, the Barnett model came closest to reproducing the desired relationship between non-dimensional fetch and non-dimensional wave height. Consequently, this model was chosen for modification in an effort to achieve a better fit between predicted and expected values. A review of the field studies by Mitsuyasu (1968) and Hasselmann et al. (1973) indicated that a major factor not considered in the Barnett model was the variation in the Phillips equilibrium constant with non-dimensional fetch. This constant was originally proposed by Phillips (1958) as a "universal constant" in the relationship between wave frequency and energy density in that portion of the surface waves dominated by wave breaking. On dimensional grounds, he hypothesized that such a relationship should be of the form

$$S(f) = Bg^2f^{-5} \tag{6}$$

where S is the energy density of waves with frequency f and B is a constant. Later work by Mitsuyasu (1966, 1968), Hasselmann et al. (1973), and DeLeonibus et al. (1974) have demonstrated that although the f^{-5} proportionality is almost always preserved in the equilibrium range, B varies systematically with non-dimensional fetch and non-dimensional wave height. Figure 9 shows that this variation can be quite important in the Great Lakes regions where non-dimensional fetches can range over several orders of magnitude. Consequently, an implicit formulation of B was programmed into the wave model to compensate for this effect. Figure 10 indicates that this brings the theoretical growth rates into much better agreement with observation.

In several tests of the modified Barnett model against deepwater gage observations in the Great Lakes, the model performed well. Figure 11

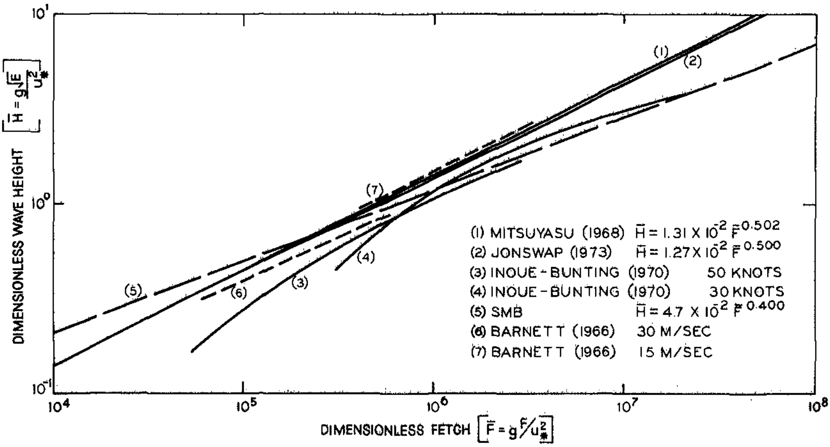


Figure 8. Comparison of wave height growths with fetch from various sources and field results of Mitsuyasu (1968) and Hasselmann et al. (1973)

SMB too large short fetch. too small large fetch

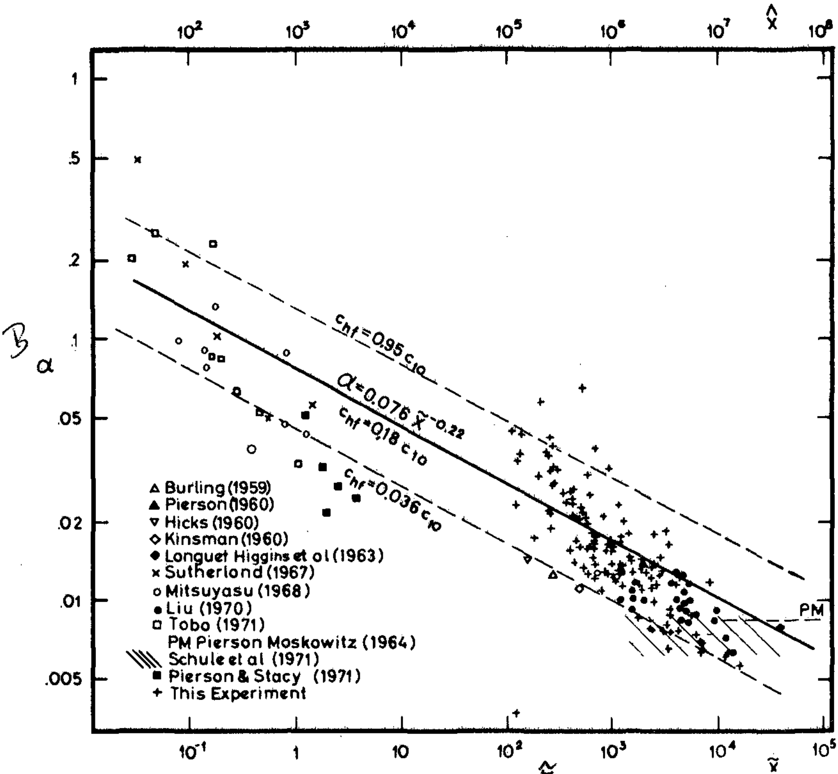


Figure 9. Variation in Phillips constant $\tilde{\beta}_{\alpha}$ with nondimensional fetch

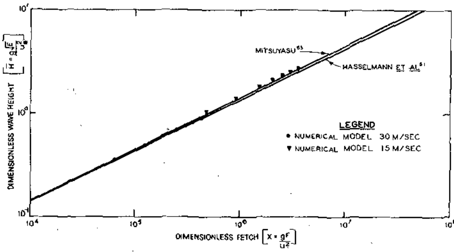


Figure 10. Comparison of rate of change of nondimensional wave height with nondimensional fetch with rates of change observed by Mitsuyasu (1968) and Hasselmann et al. (1973)

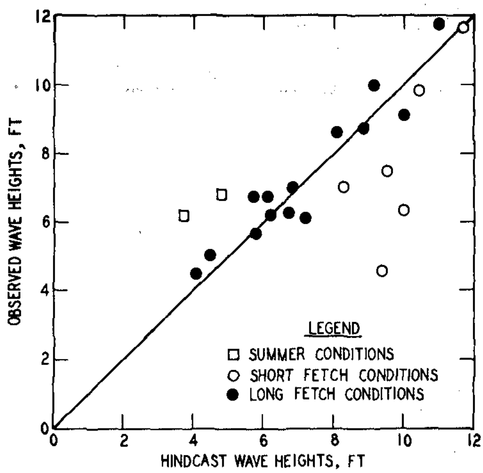


Figure 11. Comparison of maximum significant wave heights for individual storms as recorded by gages (Point Pelee, Lake Erie; and Toronto, Cobourg, and Main Duck, Lake Ontario), and significant wave heights hindcast by the numerical model using Barnett's parameterizations

is a compilation of the comparisons between observed peak significant wave heights and hindcast peak significant wave heights for storms on Lake Erie and Lake Ontario. The input to this model was taken from land winds transformed as described in the previous section. There was only a small positive bias in this sample ($\sim$4 percent), and the standard deviation of hindcasts around the best fit regression line was about 1.5 ft.

It should be noted that the wave heights calculated in the model are considered to be deepwater wave heights and must be transformed by refraction, shoaling, diffraction and bottom friction before a nearshore wave estimate can be made.

IV. Analyses Of Model Outputs.

Lake Erie, Lake Ontario, and Lake Michigan have now been completed under the present program at WES. Between 200-500 storms, covering a period of 69 years, have been hindcast for each lake. For each storm hindcast, a time series of two-dimensional spectra at 1-hour intervals was retained on magnetic tape for grid points located at about 10-mile intervals along the shoreline. These data permit an extensive set of analyses into the organization of the wave climate on these lakes. A typical set of data analyses is presented here for Cleveland, Ohio, which is on Lake Erie.

a. Analysis of extreme significant wave heights by season and direction of approach:

Since many harbors and design sites in the Great Lakes are protected by ice during the winter, the analysis of extremes was stratified into seasons defined as January-March (winter), April-June (spring), July-September (summer), and October-December (fall). Similarly, since the design of structures requires information on approach direction, the analysis separated the wave heights into direction classes as shown in Figure 12.

The largest significant wave height within each season-direction category was determined for each storm. It is assumed that the maximum significant wave height in one storm is uncorrelated with a maximum significant wave height in another storm. A preliminary study of the return periods of these wave heights indicated that they were adequately described by a Fisher-Tippett Type I distribution; and hence, the logarithm of the return period could be expected to be a linear function of wave height

$$\ell n T \approx a + bH \tag{7}$$

where T is the return period of the significant wave height H and a and b are two constants. Figure 13 shows that this is a reasonable assumption for deepwater wave heights in the Cleveland area. In this figure, the return periods were estimated by the USGS method (Dalrymple, 1960)

$$T = \frac{m+1}{n} \tag{8}$$

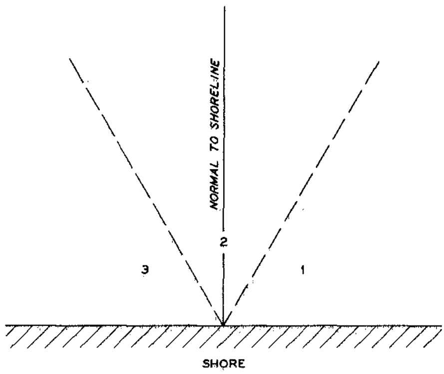


Figure 12. Definition of shoreline angles used in study

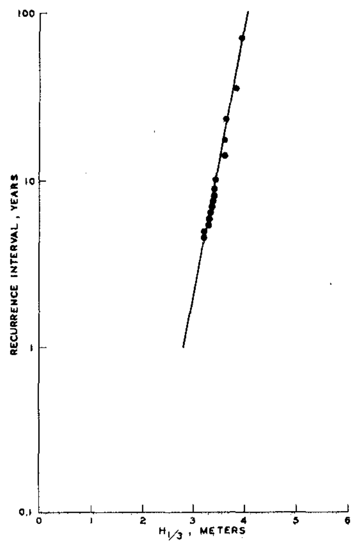


Figure 13. Linear variation of log T with wave height

where m is the total number of years in the sample and n is the rank of the magnitude (i.e., for the largest wave height n=1, for the second largest n=2, etc.).

Table 2 gives the return periods for deepwater significant wave heights in the Cleveland area for the four seasons and three angle classes previously defined. The return periods for all directions combined within each season are also given in this table along with the confidence bands for each of these estimates calculated from the relationship (Gringorten, 1961)

$$S_x = S\sqrt{\frac{1.1000y^2 + 1.1396y + 1}{N}} \tag{9}$$

where S is the standard deviation of the annual maxima of significant wave heights, S_x is the confidence band estimate (one standard deviation wide) and y is the reduced variate in the Fisher Tippett Type I distribution.

b. Recurrence intervals for wave heights within an arbitary time interval during a year.*

Although the stratification by seasons is adequate for some purposes, many planning and design criteria at sites around the Great Lakes require other intervals during the year for consideration. To obtain estimates of recurrence within arbitrary time intervals, all extremes were first categorized by 5-day increments within the year. Thirty-day overlapping periods were used to filter some of the irregularities from the distributions, and then the largest 10 significant wave heights in each time interval and direction class were plotted against recurrence intervals using the USGS method. Least squared error regression lines were calculated for each of these sets of data. Table 3 gives a matrix with the recurrence intervals for each 5-day period within the year calculated in this manner. The recurrence interval, T_k, for k 5-day periods can be calculated from the relationship

$$T_k = \frac{1}{1 - \prod_{i=1}^{k} (1-1/T_i)} \tag{10}$$

where T_i is the recurrence interval for a five-day period. As an example of the application of this table, the mean recurrence interval for a 10-ft wave in the angle class 2 in the month of July is 122 years.

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*The only constraint on this is that the time interval must be some multiple of 5 days.

TABLE 2

TABLE OF EXTREMES ESTIMATES
GRID LOCATION 8,15 LAT=43,37 LON=77,25 PULTNEYVILLE NY
SHORELINE GRID POINT 10

WINTER
ANGLE CLASSES

	1	2	3	ALL
5	6,9(1,3)	10,8(1,6)	13,8(0,6)	13,8(0,6)
10	9,2(1,7)	12,5(2,1)	14,4(0,7)	15,0(0,8)
20	9,8(2,1)	14,4(2,6)	15,1(0,9)	16,2(1,0)
50	10,8(2,6)	17,4(3,2)	16,1(1,1)	18,0(3,4)
100	12,5(3,0)	18,7(3,7)	17,4(1,3)	19,5(3,9)

SPRING
ANGLE CLASSES

	1	2	3	ALL
5	4,6(1,2)	5,6(1,3)	8,2(0,7)	8,2(0,7)
10	5,6(1,5)	7,5(1,7)	8,9(1,0)	9,4(1,0)
20	7,2(1,9)	8,5(2,1)	9,8(1,2)	10,6(1,3)
50	8,2(2,4)	9,5(2,7)	12,1(1,5)	12,2(1,5)
100	8,9(2,7)	10,8(3,1)	13,1(1,7)	13,5(1,8)

SUMMER
ANGLE CLASSES

	1	2	3	ALL
5	4,3(1,7)	4,6(1,2)	7,5(0,4)	7,9(0,5)
10	5,6(2,3)	5,6(1,5)	8,2(0,6)	8,7(0,6)
20	8,2(2,8)	7,2(1,9)	8,5(0,7)	9,6(0,8)
50	9,8(3,5)	8,2(2,4)	9,2(0,9)	11,1(4,0)
100	12,5(4,1)	8,9(2,7)	9,8(1.0)	12,7(4,1)

FALL
ANGLE CLASSES

	1	2	3	ALL
5	5,6(0,9)	12,1(0,9)	13,1(0,4)	13,2(0,4)
10	6,6(1,1)	13,1(1,1)	13,8(0,6)	14,2(0,6)
20	7,5(1,4)	14,1(1,4)	14,1(0,7)	15,2(1,5)
50	8,2(1,8)	17,1(1,8)	15,4(0,9)	16,9(1,7)
100	8,5(2,0)	18,0(2,0)	15,7(1,0)	18,3(2,1)

c. Significant period as a function of significant wave height and approach direction.

Two factors--significant wave height and over-water fetch--were found to explain the majority of the variations in significant periods. At each site, the over-water fetch is related directly to an angle of wave approach; consequently, mean significant period is well represented as a function of significant wave height within each previously defined angle class (Figure 14).

d. The expected duration of significant wave heights.

The amount of water transported over a dike during a storm is related not only to the peak significant wave height but also to the durations of significant wave heights above particular levels. A simple measure of this latter parameter is taken to be the mean duration of wave heights above any level H for all storms in which that wave height or higher occurs. Figure 15 shows the results of this calculation for the Cleveland data for the months of October through December.

e. Probabilities of higher order wave heights.

Estimates of the average wave heights for the highest 10 percent ($H_{1/10}$) or highest 5 percent of the waves in a spectrum are related directly to estimates of $H_{1/3}$ by a simple constant of proportionality. Consequently, recurrence intervals for these wave heights can be obtained by a transposition of the line for the recurrence intervals for $H_{1/3}$ (Figure 16). However, the statistics for the largest single wave in a spectrum also involve the duration of wave heights. As shown by Longuet-Higgens and Cartwright (1956), the probability of a particular wave height occurring is directly related to the wave height, the rms deviations of the surface elevation, and the duration of the sample. Since many storms have been hindcast in this study, there is no need to assume an average duration, but rather the probabilities can be calculated from the time histories.

V. Discussions and Conclusions.

An important question regarding the usefulness of hindcast wave data is the accuracy of the wave height distributions produced by the hindcasts. The answer to this question must be determined on an individual basis for each different type of probability estimate based on the hindcast data set. For example, as shown in Figure 5, the bias created by an error in wind estimates can create a significant bias at low wave heights. The importance of the type of relative bias demonstrated in Figure 5 is dependent on the magnitude of the bias created in the wave height estimates. Consequently, the extreme bias at winds under 10 knots is not significant in most cases, since the estimated wave heights for these winds are still less than a ft. On the other hand, if more accuracy is needed in this very low wave height range, an analysis similar to Figure 5 could provide a simple means of filtering the distribution of hindcast wave heights to obtain a better estimate of actual wave conditions.

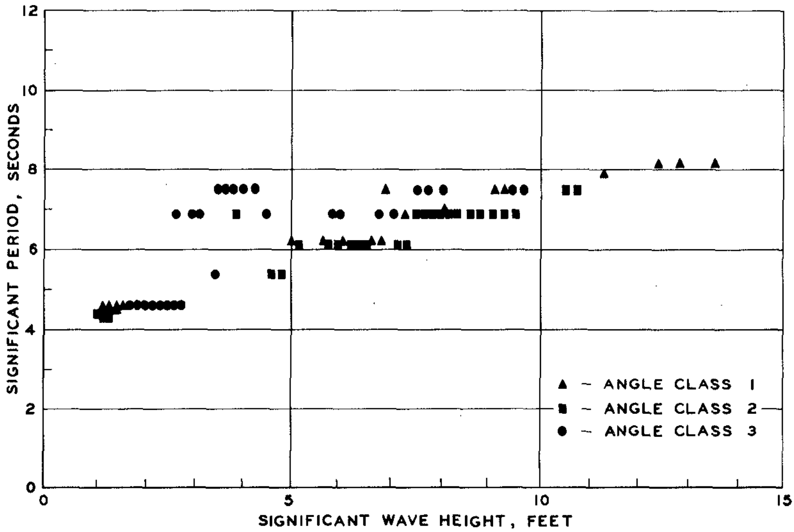

Figure 14. Significant wave period as a function of angle class and wave height

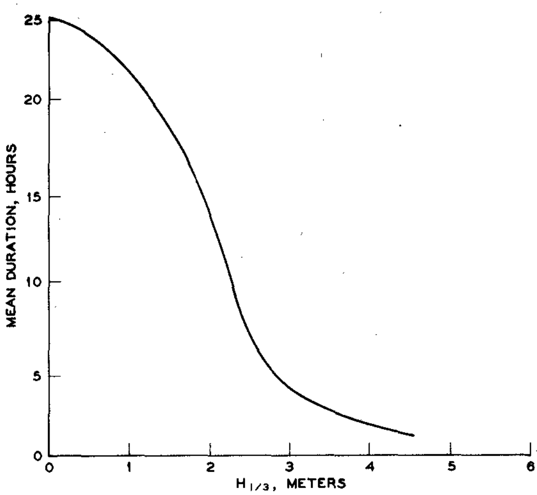

Figure 15. Mean duration of wave heights equal to or above H as a function of H

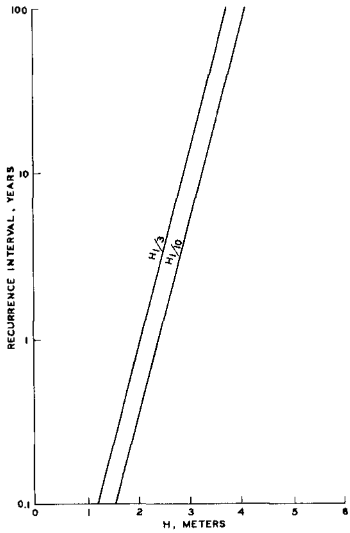

Figure 16. Linear shift in log T between $H_{1/3}$ and $H_{1/10}$

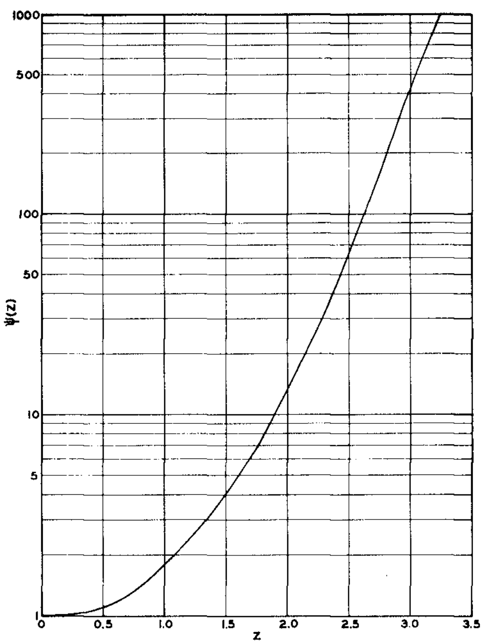

Figure 17. Relationship between $\Psi(Z)$ and Z

At the other end of the distribution, the extremes, the results shown in Figure 5 indicate very little bias; however, as shown in Resio and Vincent 1976b), an additional source of error encountered in this range comes from the effects of the convolution of the extremal distribution and a Graussian error distribution. This bias can be considered in terms of the ratio of mean recurrence interval estimated from the extremal distribution with the error term $\hat{T}$, to the mean recurrence interval for the extremal distribution alone, T . In Resio and Vincent (1976b) this was shown for a Fisher-Tippet Type 1 distribution to be given by

$$\frac{\hat{T}}{T} = \frac{1 - F(H)}{1 - F(H)\psi(z)} \tag{10}$$

where the form of $\psi(z)$ is shown in Figure 17 with z defined as

$$z = \frac{\sigma}{B} \quad ,$$

where σ is the rms error in the wave estimates and B is a coefficient of variation in the extremal distribution.

The importance of the above relationships is that it indicates that the possible bias can be quantified. For design wave calculations on Lake Ontario, this bias was shown (Resio and Vincent, 1976b) to be less than 10 percent. Additionally, the implication is that the condidence bands are not seriously affected by the random errors in individual hindcasts for these data. Thus, Equation 9 still provides a good estimate of the confidence bands when using hindcast wave heights, and the width of the confidence band still decreases as the square root of the record length.

This paper has demonstrated the utility of a numerical hindcast model in providing a set of data which can subsequently be analyzed to obtain a wide range of information on the organization of the wave climate at a site. Although there are errors in specific hindcasts, these errors do not appear to create significant bias in many of these statistical estimates. Furthermore, the ability to obtain a long-term, synthetic record of two-dimensional spectra permit an extensive analysis into even complex phenomena with higher confidence than could be obtained from an exact record obtained from exact measurements over a shorter period of time.

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