#170

UILU-WRC-76-0107
RESEARCH REPORT NO. 107

Development of Criteria for Shore Protection Against Wind-Generated Waves for Lakes and Ponds in Illinois

By Nani G. Bhowmik

ILLINOIS STATE WATER SURVEY
Urbana, Illinois

UNIVERSITY OF ILLINOIS
AT URBANA-CHAMPAIGN
WATER RESOURCES
CENTER

JANUARY 1976

WRC RESEARCH REPORT NO. 107

DEVELOPMENT OF CRITERIA FOR SHORE PROTECTION AGAINST WIND-GENERATED WAVES FOR LAKES AND PONDS IN ILLINOIS

by

Nani G. Bhowmik, Associate Engineer

Illinois State Water Survey
Urbana, Illinois

FINAL REPORT

Project No. A-069-ILL

DEVELOPMENT OF CRITERIA FOR SHORE PROTECTION AGAINST WIND-GENERATED WAVES FOR LAKES AND PONDS IN ILLINOIS

July 1, 1974 - September 30, 1975

The work described in this publication was supported by funds provided by the
U.S. Department of the Interior as authorized under the Water Resources Research Act of 1964,
P.L. 88-379 Agreement No. 14-31-0001-5013

UNIVERSITY OF ILLINOIS
WATER RESOURCES CENTER
2535 Hydrosystems Laboratory
Urbana, Illinois 61801

September 1975

ABSTRACT

DEVELOPMENT OF CRITERIA FOR SHORE PROTECTION AGAINST WIND-GENERATED WAVES FOR LAKES AND PONDS IN ILLINOIS

A practical design criteria is proposed to stabilize lake shores against wind waves. The wind waves in inland waterways such as rivers, canals, or lakes can and will cause substantial erosional damage if the waterway shore does not have natural or artificial protection. The orientation of the main watersheds and the general wind movement during part of the year in Illinois are conducive to the generation of damaging waves in lakes. Extensive lake shore erosion caused by wind waves is present in Illinois.

A methodology was developed for estimating the height of wind waves in any lake for a given wind condition. Maximum wind speeds from five climatological stations in and around Illinois for the period of 1950-1972 were analyzed and a table was prepared showing the maximum wind speed for various durations and return periods. Statistical analysis of wind wave data collected from Carlyle Lake indicated that Rayleigh distribution fitted the wave height distribution reasonably well and that the nondimensional energy spectra followed $(f/f_m)^{-5}$ rule in the equilibrium range of frequencies. Boat-generated wave heights fitted Rayleigh distribution to some extent and a relationship was developed relating boat speed, maximum wave height, distance between the boat and the wave gage, and the draft of the boat.

From a consideration of various forces and physical properties of riprap particles and water, a relationship was developed to estimate the stable weight of riprap particles. Recommendations as to the proper selection of gradation, range of sizes, thickness of riprap particles and their size distribution, and gradation and thickness of filter materials were also incorporated. A design procedure was developed and one specific design problem was solved showing in detail all the steps involved in an actual design problem.

REFERENCE: Bhowmik, Nani G., **DEVELOPMENT OF CRITERIA FOR SHORE PROTECTION AGAINST WIND-GENERATED WAVES FOR LAKES AND PONDS IN ILLINOIS,** University of Illinois Water Resources Research Center Research Report No. 107.

KEY WORDS: Bank stabilization, boating, design, erosion, filters, gravel, lakes, riprap, shore protection, stability, waves, wind (meteorology).

CONTENTS

DEVELOPMENT OF CRITERIA FOR SHORE PROTECTION AGAINST WIND-GENERATED WAVES FOR LAKES AND PONDS IN ILLINOIS

by Nani G. Bhowmik

INTRODUCTION

Waves produced by winds blowing over the water surface are called "wind waves." They may vary in size from ripples to some larger size in lakes and to giant waves on the ocean. Wind waves are defined by their height, length, frequency, and energy spectrum. Wind wave characteristics are determined by wind speed, direction, and duration as well as by the length and width of the water surface over which the wind blows. The wind waves in inland waterways such as rivers, canals, or lakes will cause substantial erosional damage if the waterway shore does not have natural or artificial protection.

In a notice to the navigation interests in the Ohio River by the U.S. Army Corps of Engineers, Nichols (1975) stated: "It is known that erosion is caused by a variety of forces, including natural river currents, wind-generated waves, rapid rises and falls in river stage, and the surges, bow waves and propeller washes created by vessel traffic . . . ." Extensive lake shore erosion due to wind-generated waves exists in Illinois. The orientation of the main watersheds and the general wind movement during part of the year in Illinois are conducive to the generation of damaging waves in reservoirs (figure 1). The Mississippi River drains a major part of Illinois in a westerly, southwesterly to southerly direction. Any reservoir created on any one of the main streams will have its water surface length in the same direction. The orientation of the three large man-made lakes, Carlyle, Shelbyville, and Rend, substantiate this fact. The analysis of the long-term wind speed indicates that for the months of March through August, when the reservoirs are normally full from spring runoff, the wind generally blows from the west, southwest, or south. This natural coincidence of the wind direction and the maximum water surface length in reservoirs increases the potential of wind wave damage. Figure 2 shows such wind wave erosion in Carlyle Lake in the Hazlet State Park area.

The present research effort, aimed toward practical field application, had the following objectives:

1) To develop a theoretical relationship supported by field data needed to define wave characteristics and to estimate wave heights in lakes with known wind speed and direction
2) To analyze the long-term wind velocity in Illinois in order to estimate the wind speed for any specified location, duration, and return period
3) To collect field data on wind speed and direction and on wave heights for different wind characteristics
4) To collect data on size distribution of riprap materials on lake shores with protected banks
5) To make a statistical analysis of the wave history of the collected data to determine the energy spectrum and the characteristics of the wave heights

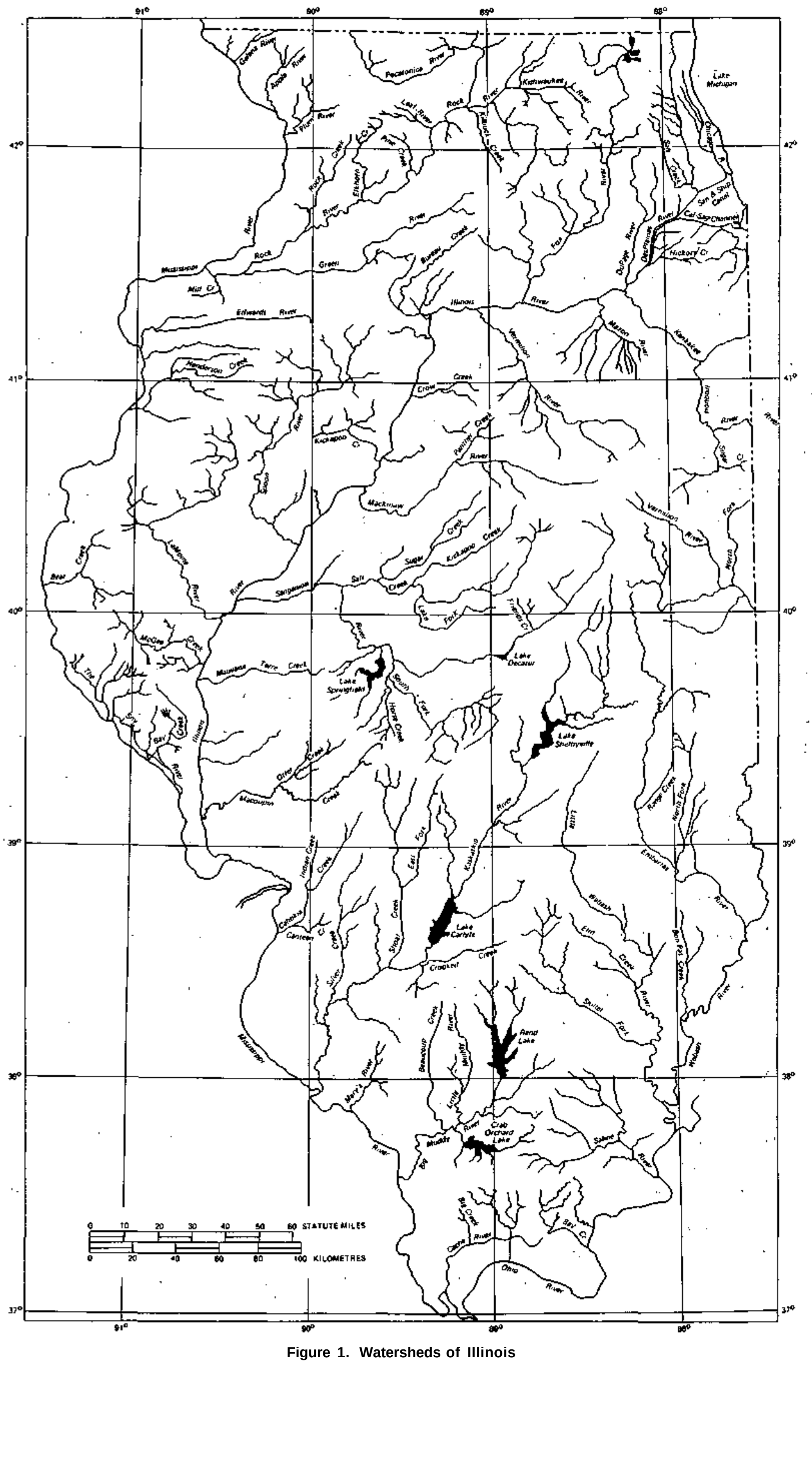


Figure 1. Watersheds of Illinois

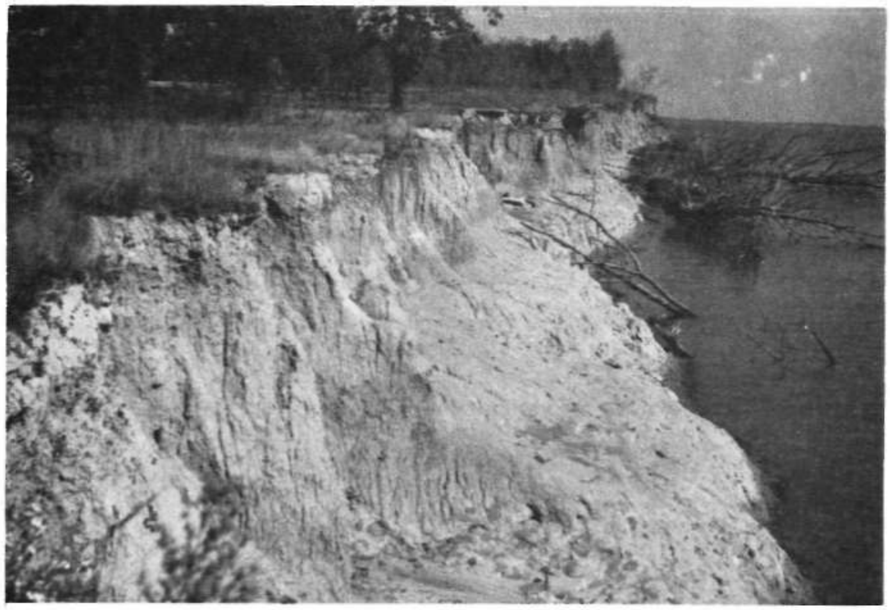


Figure 2. Wind-generated wave erosion in Carlyle Lake

6) To develop a stabilizing criteria for protecting lake shores by riprap materials against the destructive action of wind-generated waves

Some data on boat-generated waves collected from Carlyle Lake have also been analyzed and the results are included in this report.

Acknowledgments

This research has been carried out by the author as part of his regular work at the Illinois State Water Survey under the guidance of John B. Stall, Head of the Hydrology Section, and William C. Ackermann, Chief. The work upon which this research is based was supported, in part, by funds provided by the U.S. Department of the Interior as authorized under the Water Resources Act of 1964, Public Law 88-379, Project No. A-069-ILL. Ji-Ang Song, a graduate research assistant in the Civil Engineering Department of the University of Illinois, helped in the collection and analyses of the data. James C. Neill, Statistician, and Carl G. Lonnquist, Numerical Analyst, at the Water Survey assisted in the computer analysis of the raw data. Many discussions with Julius H. Dawes, Hydrologist at the Water Survey were very stimulating and helpful. Wilbur Debolt, Jr., of the Survey staff helped in the installation and maintenance of the wind set. Glenn E. Stout, Director of the Water Resources Center, was helpful in carrying out and reporting the research. Deep appreciation is expressed to A. L. LeGrand, Reservoir Manager of Carlyle Lake, for his extensive assistance with the data collection. John W. Brother, Jr., prepared the illustrations. Mrs. J. Loreena Ivens, Technical Editor, edited the report. IBM 360/75 computer at the University of Illinois was used for data analyses.

THEORETICAL BACKGROUND

Leonardo da Vinci, famous naturalist and scientist of the fifteenth century, made significant contributions to the understanding of the mechanism of wave motion (Titov, 1971). During the intervening centuries various investigators developed theoretical relationships, collected field data from oceans, lakes, and reservoirs, and postulated methods for estimating the wave heights from some known basic parameters. The present report is mainly concerned with the aftereffects of waves and how to prevent the erosional damages created by wind waves. Theories related to waves are given by Lamb (1932), Ippen (1966), Titov (1971), and many other researchers. Only the theories having direct correlation with the present investigation are discussed.

Wave Heights

Figure 3 shows a typical wind-generated wave profile. This type of series is termed a time series. Accepted methods of time series analyses can be performed on a series similar to the one shown and inferences can be made.

Autocorrelation Coefficients and Correlograms. The series shown in figure 3 is represented by a continuous process, $h(t)$. The digitizing of this wave record at interval Δt produces n values of equal spaced time series, $h(t), h(t + \Delta t) \ . \ . \ . \ h[t + (n - 1)\Delta t]$, where $h(t)$ is the surface displacement due to waves.

Autocorrelation coefficients are ordinary linear correlation coefficients between a time series and the same series at an interval of time later. In figure 3, $h(t)$ and $h(t + \tau)$ are two values of the time series which are τ lag apart. The autocorrelation coefficient $R(\tau)$ is defined as (Panofsky and Brier, 1958; Yevjevich, 1972)

$$R(\tau) = \mathrm{Cov}\,[h(t), h(t + \tau)]/\sigma^2 \qquad (1)$$

where σ^2 is the variance and Cov is the covariance of the time series realization represented by figure 3. A correlogram shows the autocorrelation coefficient $R(\tau)$ as a function of the lag time τ.

Fourier Coefficient and Periodogram. The longitudinal wave profile shown in figure 3 with T as the length of record may be fitted by trigonometric functions. A Fourier analysis is usually performed to decompose such a continuous series into regular sine and cosine components as follows:

$$h(t) = h + \Sigma[A_n(\cos 2\pi n/T)t + B_n(\sin 2\pi n/T)t] \qquad (2)$$

which can be reduced to

$$h(t) = \overline{h} + \Sigma[C_n(\cos 2\pi n/T)(t - t_n)] \qquad (3)$$

where $C_n^2 = A_n^2 + B_n^2$, C_n is the amplitude of the rath harmonic, and t_n is the time at which the nth harmonic has a maximum, and $\overline{h}$ is the average value of the time series. The value of C_n is given by

$$C_n = \int_{-\infty}^{\infty} h(t)\exp(-i2\pi ft)\,dt \qquad (4)$$

The coefficient C_n is referred to as the Fourier coefficient. More precisely, $C_n^2/2$ represents the contribution to the variance by the rath harmonic. A plot of C_n^2 versus f, where f is the frequency

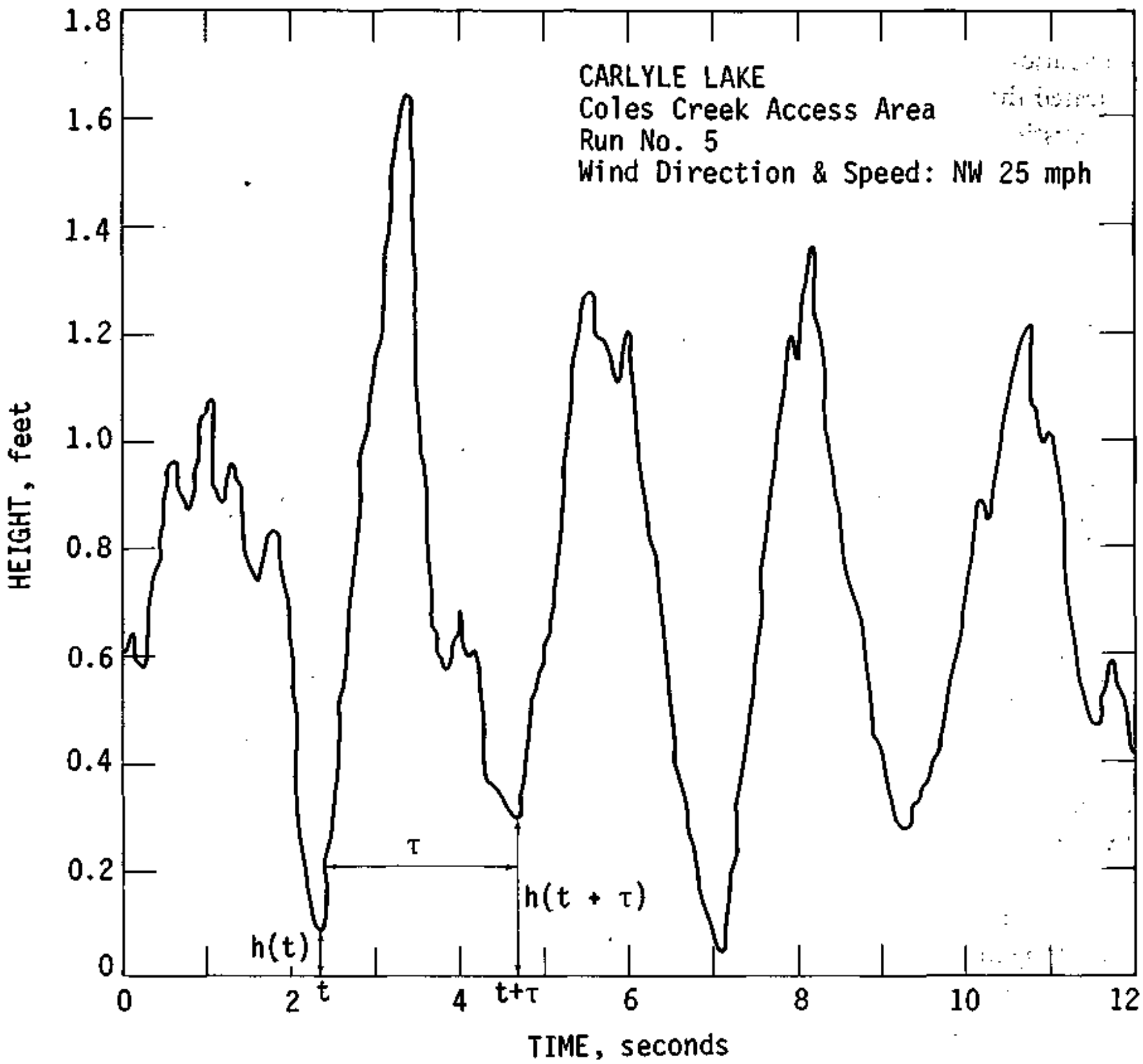


Figure 3. Typical wind-generated wave profile

(n/T), is called a periodogram and this shows the contribution of each harmonic to the total variance of *h(t).*

Spectral Density. The spectral density or power spectrum $\Phi(f)$ of a time series realization similar to one shown in figure 3 is generally defined as (Blackman and Tukey, 1958)

$$\Phi(f) = \int_{-\infty}^{\infty} [C(\tau)] \, (\cos 2\pi f \tau) d\tau \qquad (5)$$

where $C(\tau)$ is the autocovariance function.

Statistical Distribution of Wave Heights. Lonquet-Higgins (1952) has shown that the distribution of wind-generated wave heights is given by Rayleigh distribution. The cumulative Rayleigh distribution is defined as

$$p(H/\overline{H}) = p(\eta) = 1 - \exp\left[-(\pi/4)(\eta)^2\right] \qquad (6)$$

where H is the individual wave height and $\overline{H}$ is the average wave height in a time series realization of the wave heights. Bretschneider (1959) and Colonell and Perry (1968) indicated that wave height distribution fits well the Rayleigh distribution. However, Goodknight and Russell (1963) have indicated that, although the Rayleigh distribution can be utilized to estimate the wave heights, the χ^2 statistical tests did not indicate a good fit of the wave height data collected from the Bay Marchand area in the Gulf of Mexico.

Other statistical parameters such as standard deviation, σ, and coefficient of variation C_v can be utilized to describe wind wave characteristics.

Wave Energy. The total energy of a wave system can be divided into potential and kinetic energy (Ippen, 1966; Lamb, 1932). The wave energy is generally expressed in terms of total energy per unit of surface area (Ippen, 1966).

Potential energy *(PE)* and kinetic energy *(KE)* per unit of surface area is given by

$$E = PE + KE = (1/16)\Upsilon H^2 + (1/16)\Upsilon H^2 = (1/8)\Upsilon H^2 \tag{7}$$

where Υ is the unit weight of water, H is the height of a small amplitude wave, and E is the average energy per unit of surface area. Similarly, the energy per unit of surface area in the wave profile generated by a moving boat is given by (Das, 1969)

$$E_n = (1/2)\Upsilon C_n{}^2 \tag{8}$$

This indicates the energy per unit of surface area at each frequency f.

Significant Wave Height. Various investigators have proposed relationships and formulas for computing wave heights. The significant wave height is defined as the wave height where one-third of the waves in the wave profile is more than this wave height. Stevenson's formula was revised by Molitor who introduced wind velocity as a variable. This formula gives approximately 2.5 feet minimum wave height for the smallest fetch. However, the ASCE Subcommittee (1948) on slope protection indicated that the Molitor-Stevenson equation predicts a conservative higher value for wave heights.

Sibul (1955) studied the generation of wind waves in shallow water in a laboratory flume, and later supplemented the laboratory data with field measurements from various sources. The Sverdrup-Munk and Bretschneider relation (Sibul, 1955) of wave height as a function of wind velocity and fetch length was modified to include the effect of shallow water, and the following relation was presented:

$$gH_s/U^2 = 3.23 \times 10^{-3} (gF/U^2)^{0.435} \tag{9}$$

for $gF/U < 3 \times 10^4$

where F is the fetch length in feet, U is the wind velocity in fps, and H_s is the significant wave height in feet.

The above relation is applicable when the width of fetch is also of the same magnitude as the length. Because of this, the relation cannot be used for waterways or lakes with limited width. At this point, the correction proposed by Saville (1954) to estimate the effective fetch length, F_e, for a limited width can be utilized. Carlson and Sayre (1961) also utilized the correction proposed by Saville in order to compute the wave height for some irrigation canals of U.S. Bureau of Reclamation and concluded that the measured and computed wave heights are in close agreement. For a width to fetch, *W/F,* ratio of 0.05 to 0.6 the effective fetch, F_e, from Saville's data is given by

$$F_e = 1.054\, W^{0.6} F^{0.4} \tag{10}$$

When the wind direction is not coincident with the water surface alignment of the lake, the wind effectiveness can be assumed to vary with the cosine of the wind angle according to the formula

$$U_e = U \cos \Phi \tag{11}$$

where U_e is the effective wind velocity, U is the actual wind velocity, and Φ is the angle between wind direction and the water surface bearing. The above assumption appears to be reasonable for deflection angles less than 45 degrees.

With the above modifications, equation 9 can be rewritten as:

$$gH_s/U_e^2 = 3.23 \times 10^{-3} (gF_e/U_e^2)^{0.435} \tag{12}$$

When the wind velocity, direction, and fetch length and width are known, the significant wave height in a lake for any location can be estimated with equation 12.

Wind Tide

Wind blowing over the water surface of an enclosed body of water will exert a force and may pile up water on the leeward side of the lake with an associated lowering at the windward side. This is termed 'wind tide.' Many researchers have studied this phenomenon as reported by Saville et al. (1962) and the following relation is generally accepted as valid to estimate wind tide

$$S = (KU^2 F \cos \Phi)/D \tag{13}$$

where S is the wind tide, K is a coefficient, and D is the average depth of water in the lake.

Stability of the Lake Bank

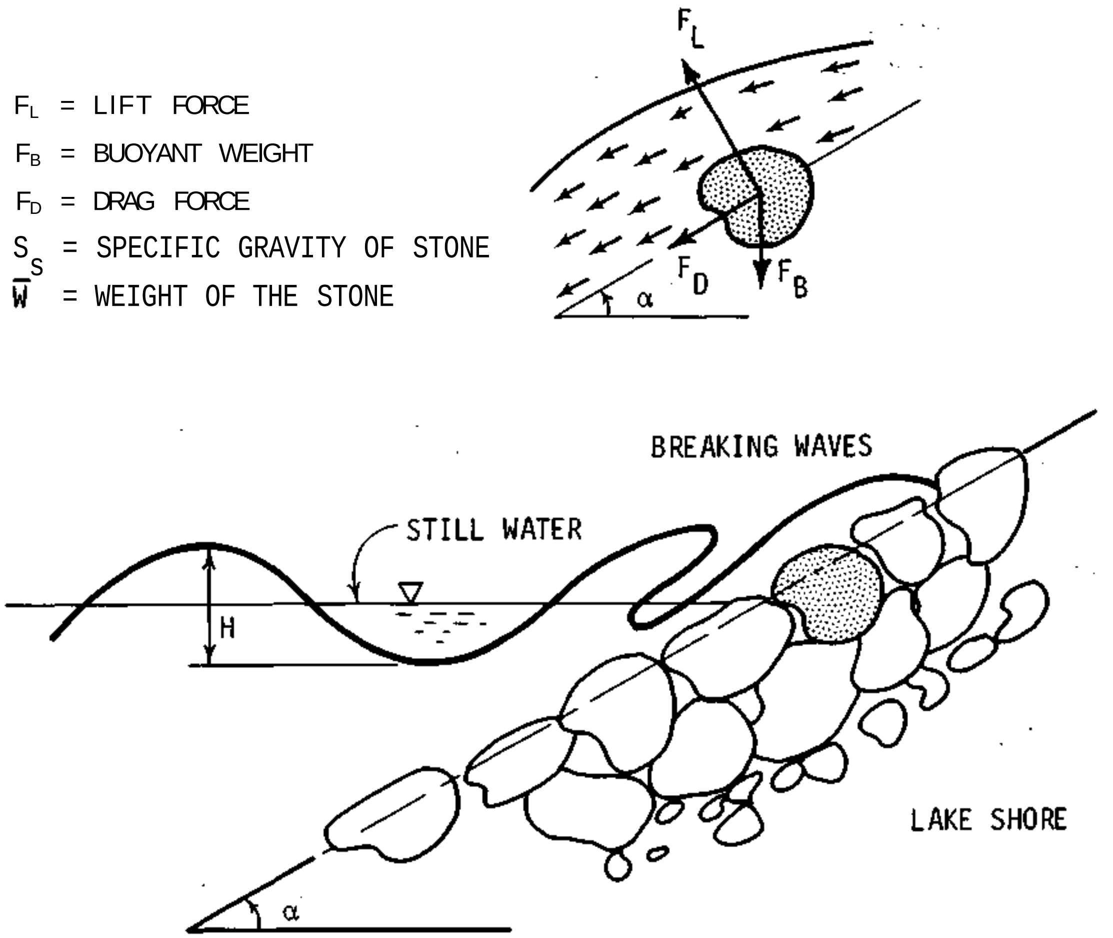
Lake shores are generally vulnerable to the assault of wind waves. As the wave approaches upward onto a sloping beach, the lower part of the wave is retarded by the friction and pressure of the beach, while the top part continues with almost its original velocity. After breaking against a shore, waves sometimes throw water high in the air depicting the tremendous amount of energy they contain. The breaking waves follow a downward path along the bank to the lake and may wash away the fine sands and start the failure of the bank.

The lake shores can be stabilized with stones or riprap. Riprap materials are esthetically pleasing and sometimes available at the location. Moreover, in the long run their use may be economical. The stable size of the riprap materials can be determined from a stability analysis of the bank for an anticipated wave condition.

Figure 4 shows the typical forces that act on a single stone in a bank when an oncoming wave breaks against the bank and follows a downward path. It was assumed that the depth of water at the breaking of the wave is equal to wave height, H (Hedar, 195 3).

A stability analysis will indicate that

$$\tan \bar{\Phi} = \frac{F_D + F_B \sin a}{F_B \cos a - F_L} \tag{14}$$



$$\overline{W} = \frac{\overline{K}H^3 S_s}{(S_s - 1)^3 (\cos\alpha - \sin\alpha)^3 \gamma^2}$$

Figure 4. Stability analysis of riprap particles

where $\overline{\Phi}$ is the submerged angle of repose of the riprap materials. With the assumption that $\overline{\Phi}$ equals 45 degrees, substitution of the expressions for different forces, and simplification, equation 14 becomes

$$\overline{W} = \overline{K} S_s H^3 / \Upsilon^2 \, [(S_s - 1)(\cos\alpha - \sin\alpha)]^3 \qquad (15)$$

where $\overline{W}$ is the weight of the stable riprap particle, $\overline{K}$ is an unknown coefficient, S_s is the specific gravity of the riprap materials and Υ is the specific weight of water. The coefficient $\overline{K}$ can be determined from published laboratory and field experimental data.

FIELD DATA COLLECTION

Data on wind velocity, wind-generated waves, boat-generated waves, size distribution of existing riprap particles, and other pertinent information were collected from Carlyle Lake (figure 5). This is the largest man-made lake in Illinois with 83 miles (134 km) of shoreline and 26,000 acres (10,522 hectares) of surface area at normal pool level. Carlyle Lake is located along the Kaskaskia River. The geology of the area is described by Lineback (1975) as follows: " . . . The Kaskaskia River is a major drainage way predating glaciation and served as a major melt-water channel for Kansan, Illinoian and Wisconsinan glaciations. . . . The old channel is filled with sand and gravel overlain by silts, or in places, till. . . . . . . The younger valley is underlain by Wisconsinan age outwash consisting of sand and fine gravel." The characteristics of the original bank materials can be seen in figure 2.

Wind Data. Data on wind velocity and direction were collected at two locations in Carlyle Lake as shown in figure 5. There were two recording wind gages at the Coles Creek Access Area at 10 feet (3.05 m) and 32.8 feet (10 m) above the ground level. One other recording wind gage was installed at Saddle Dam No. 2 at a height of 32.8 feet (10 m). All three wind sets were synchronized and data were collected continuously for the duration of the field experimentation.

Wind-Generated Wave Data. The wind-generated wave data were collected at the site shown on figure 5. Three staff gages were installed at a distance of about 80 feet (24.4 m) from the shoreline (figure 6). A super-8 mm movie camera equipped with a zoom lens was fixed on a tripod on the shore as shown in the picture. Whenever the wind and wave conditions were felt to be right, the wind-generated waves passing a fixed staff gage were photographed continuously, at the rate of 18 frames per second for a period of about 3 1/3 minutes. The whole procedure was repeated for various wind and wave conditions.

The developed film was projected at a rate of 2 frames per second on a chart moving at a constant rate of 5 inches (127 mm) per minute on a Sargent recorder. The chart was calibrated for each run to establish the scales for wave heights and the elapsed time. By varying the input voltage to the recorder from a variable voltage regulator, the inking pen was manipulated to follow continuously the water surface profile projected on the chart. This procedure yielded a continuous record of the wave history on the chart. The chart was then read by an Autotrol machine and the wave history data were subsequently digitized.

Riprap Size Distribution. Lake shores stabilized with riprap materials were selected to collect data on size distribution of these materials. Data were collected from both the stable and unstable segments of the banks. Since these stones are generally very large and almost impossible to bring to the laboratory for sieve analysis, the field technique described below was used to measure the sizes of the riprap materials.

A 10-foot by 10-foot (3.05 m) grid with 1 foot (0.305 m) squares made of 1/4 inch (6.35 mm) nylon line was spread on top of the designated area where riprap size distribution data were to be collected. The grid was made square by using wooden stakes at desirable locations. A total of 121 stones at the intersection of each grid point were selected and the longest, intermediate, and minor axes of the stones were measured. A similar technique was used by Wolman (1954) to measure the gravel sizes in mountainous streams. It was shown by Wolman that this type of statis-

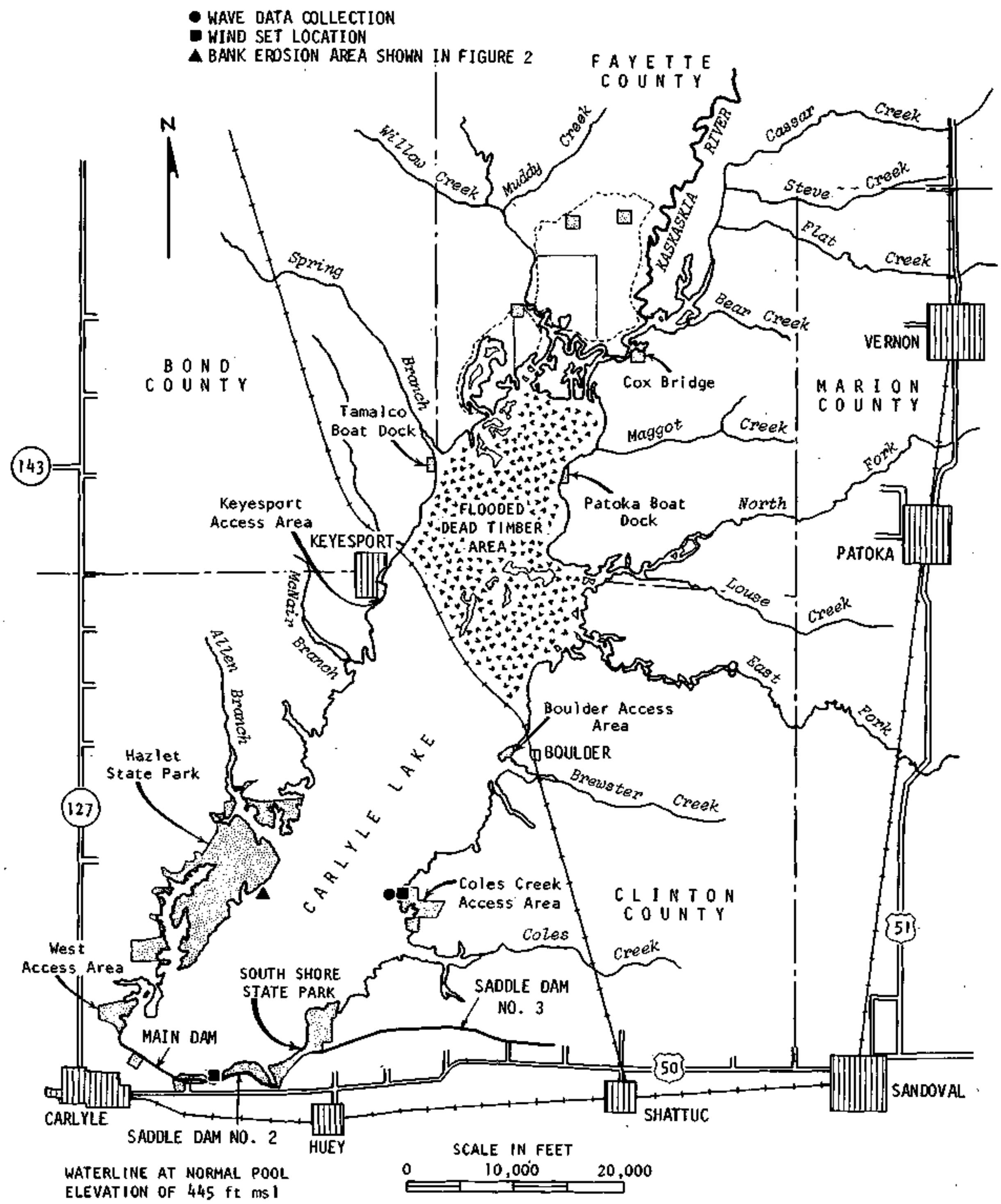


Figure 5. Carlyle Lake and the location of field experimentation

Figure 6. Arrangements utilized for wave data collection

tical data gave very reasonable results where the size distribution of the intermediate axes of all the stones were compared with the size distribution of the same gravels obtained from standard sieve analysis. In the present investigation it was also observed that this technique yielded very good results.

Boat-Generated Wave Data. Data on boat-generated waves were also collected from Carlyle Lake at the Coles Creek Access Area (figure 5). The general technique was the same as that described for the wind-generated wave data collection and shown in figure 6. The boat was 18 feet (5.49 m) long, 7.25 feet (2.21 m) wide, 3.25 feet (0.99 m) deep at midship and weighed about 2200 pounds (100 kg). The distance between the boat and the staff gage and the speed of the boat were varied, and the boat-generated waves passing a staff gage were photographed. The data were reduced by the same technique described before.

DISCUSSION AND ANALYSES OF THE DATA

Historical Wind Data

It was pointed out earlier that the waves produced in a lake are a function of wind velocity, direction, duration, and the length of the fetch. However, in order to estimate the wind characteristics in the vicinity of any lake or reservoir, long-term wind data from the closest climatological station should be analyzed. Five climatological stations in and near Illinois were selected and the wind data from 1950 to 1972 were gathered. Some of these data were listed as hourly, 3-hourly, or 6-hourly either on a daily basis or at every hour of the day for the entire month. Thus, considerable time and effort were required to synthesize these data, for a time series. The five climatological stations were Urbana, IL; Springfield, IL; Moline, IL; St. Louis, MO; and Evansville, IN. It was felt that these stations should give a very good coverage of the whole state. Data related to wind were published by the U.S. Department of Commerce, Weather Bureau.

Tentative examination of all wind data indicated that the selection of maximum wind speeds blowing more or less from the same direction for durations of 6 hours, 12 hours, and 24 hours should cover the ranges of prime interest. Wind data for 6 months of the year from January to June were checked, and the maximum wind velocities for the specified duration blowing approximately from the same direction were selected and the values tabulated. This procedure generated 23 values of wind speed and direction for each specified duration for each of the 6 months at each climatological station.

An analysis of these selected maximum wind velocities indicated that they plot as a straight line on log-normal probability paper. The plotting position was determined by the relation $m/N+1$, where m is the rank of the wind speed and N is the total number of observations in each set. The goodness of fit was tested by the χ^2 test, which indicated that the departures of the wind speed from the theoretical function were not significant and hence the log-normal function could be used to analyze the wind speed data.

Figure 7 shows typical CalComp plots of wind speed data for January and February at Moline for 1950-1972. Similar plots were developed for the other climatological stations. Maximum wind speed for a specified duration and return period can be estimated from plots similar to figure 7. However, instead of showing all the plots, the maximum wind speed data for the five stations are presented in table 1. The table shows the maximum speed for the three durations for return periods of 2 to 100 years. There are two wind speed values under each duration, the first being the best estimate from the log-normal plot and the second the upper 95 percent confidence limit. The prevailing wind directions are also given. Therefore, table 1 can be used to estimate a wind speed for a specified duration and return period for any area of interest in the state of Illinois.

Wind Data from Carlyle Reservoir

Figure 8 shows the wind velocity distribution at Carlyle Lake in the Coles Creek Access Area for the months of May and June 1974. These data were taken from the wind set installed at a height of 32.8 feet (10 m). Wind velocities and directions shown are daily averages. Generally wind blew from the south with daily average velocities of about 10 mph (16.09 km/hr).

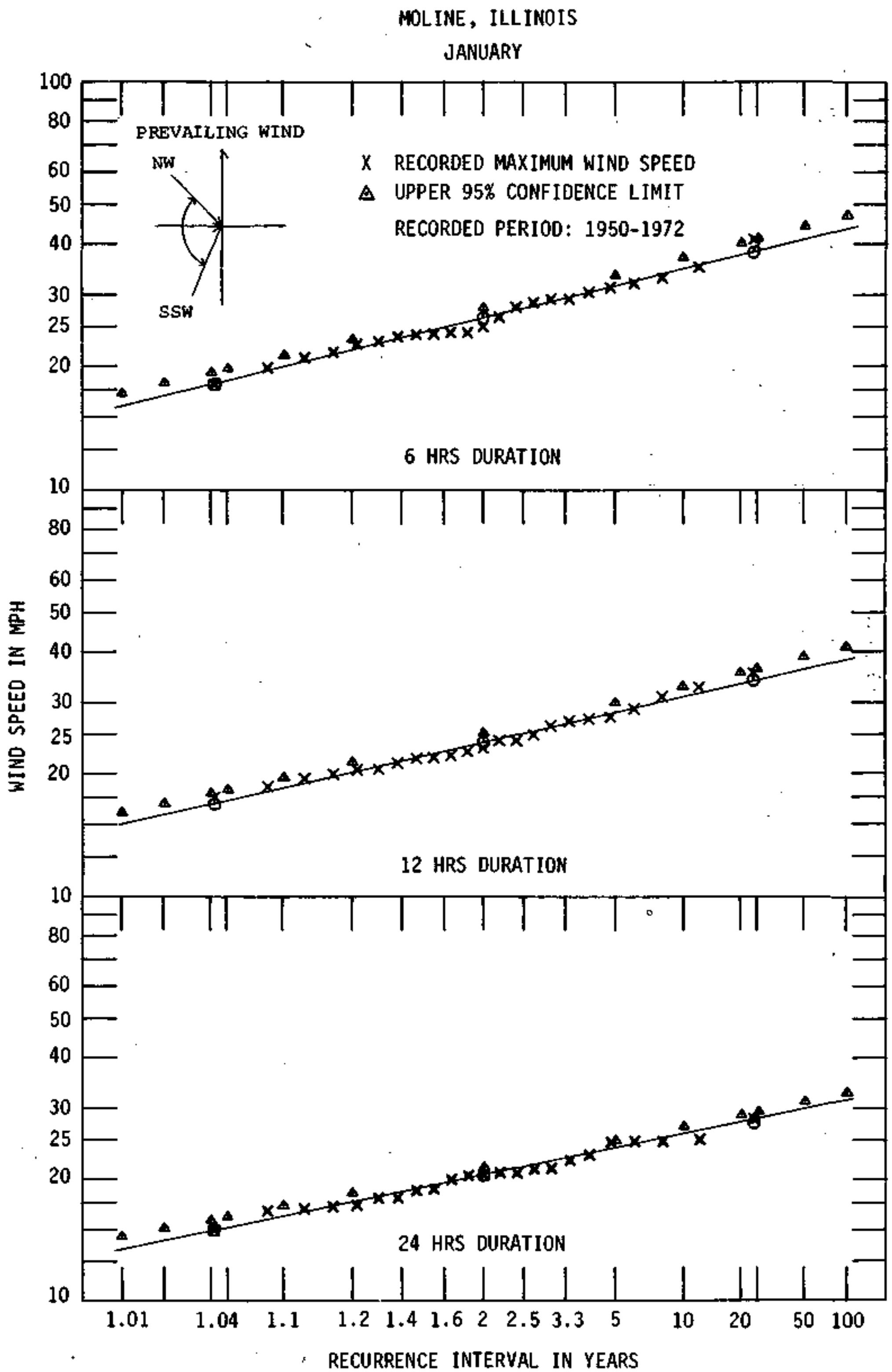


Figure 7. Long term wind velocity variations for Moline

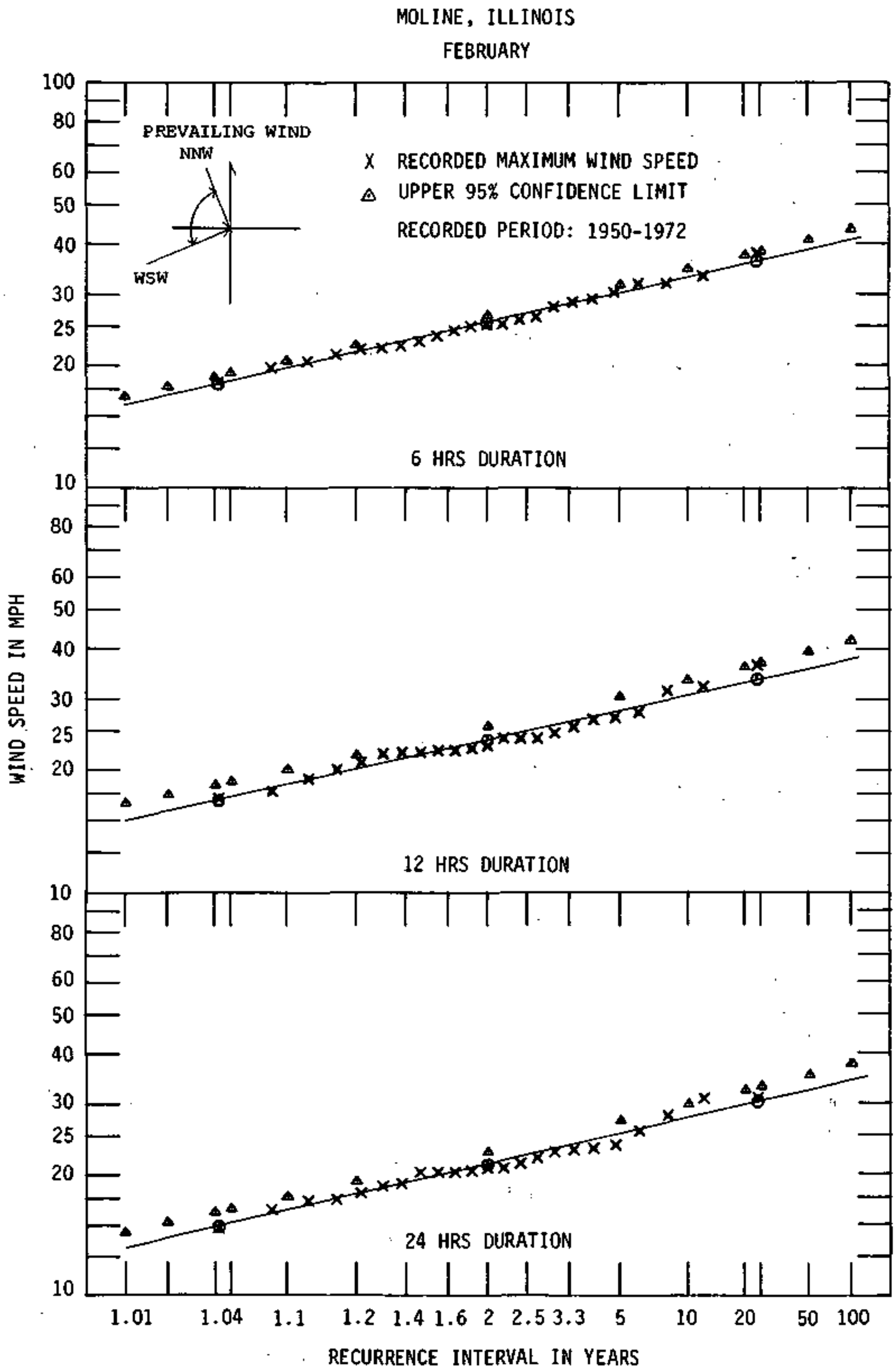
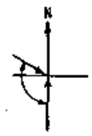
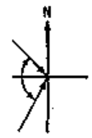
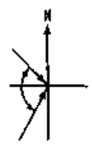


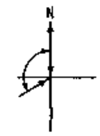
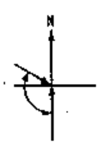
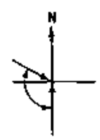
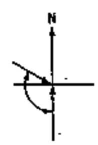
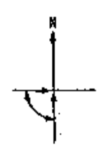
Figure 7. (Concluded)

Table 1. Maximum Wind Speeds in Miles per Hour, 1950-1972

SPRINGFIELD, ILLINOIS

RETURN PERIOD	6 HOUR		12 HOUR		24 HOUR		PREVAILING WIND
			JANUARY				
2	27.88	30.31	25.94	28.73	22.71	24.94	
5	31.96	34.75	29.84	33.17	25.99	28.53	
10	34.16	37.41	32.11	35.88	27.73	30.70	
20	36.19	39.82	34.11	38.33	29.35	32.67	
25	36.80	40.56	34.72	39.09	29.83	33.27	
50	38.61	42.77	36.51	41.37	31.28	35.08	
100	40.32	44.90	38.21	43.58	32.63	36.82	
			FEBRUARY				
2	29.04	31.31	26.19	28.05	22.45	23.81	
5	34.83	37.66	31.07	33.37	26.17	27.81	
10	38.30	41.57	33.97	36.62	28.35	30.21	
20	41.43	45.16	36.57	39.58	30.28	32.39	
25	42.39	46.27	37.37	40.49	30.87	33.05	
50	45.26	49.63	39.74	43.25	32.62	35.05	
100	48.09	52.89	42.01	45.91	34.27	36.97	
			MARCH				
2	32.33	36.88	29.61	33.48	25.72	28.00	
5	40.29	46.18	36.64	41.62	30.72	33.55	
10	45.20	52.15	40.95	46.81	33.72	36.98	
20	49.70	57.78	44.90	51.67	36.41	40.13	
25	51.09	59.55	46.11	53.20	37.23	41.10	
50	55.30	64.99	49.79	57.88	39.69	44.05	
100	59.39	70.38	53.34	62.50	42.05	46.92	
			APRIL				
2	29.51	33.70	26.56	30.46	23.33	26.67	
5	36.13	41.47	32.52	37.48	28.05	32.23	
10	40.16	46.40	36.15	41.95	30.89	35.73	
20	43.82	51.02	39.45	46.14	33.45	38.99	
25	44.95	52.47	40.46	47.46	34.24	40.01	
50	48.35	56.91	43.52	51.48	36.59	43.11	
100	51.63	61.28	46.47	55.45	38.84	46.16	
			MAY				
2	27.32	30.84	24.72	28.37	21.29	23.11	
5	32.86	37.28	28.85	33.29	25.05	27.27	
10	36.20	41.31	31.29	36.34	27.27	29.80	
20	39.20	45.05	33.45	39.16	29.25	32.12	
25	40.12	46.21	34.10	40.04	29.85	32.83	
50	42.88	49.77	36.06	42.69	31.65	34.99	
100	45.53	53.25	37.91	45.28	33.36	37.07	
			JUNE				
2	21.90	23.22	19.55	20.43	16.67	17.50	
5	25.27	26.86	22.16	23.19	18.92	19.91	
10	27.24	29.03	23.66	24.81	20.22	21.32	
20	28.98	30.99	24.97	26.26	21.35	22.58	
25	29.51	31.59	25.37	26.70	21.70	22.97	
50	31.07	33.38	26.54	28.00	22.71	24.11	
100	32.55	35.10	27.63	29.24	23.66	25.20	

URBANA, ILLINOIS

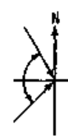
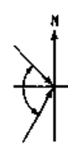
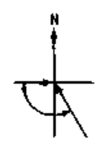
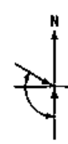
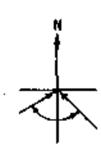
RETURN PERIOD	6 HOUR		12 HOUR		24 HOUR		PREVAILING WIND
			JANUARY				
2	17.66	19.32	16.57	18.01	14.81	15.97	
5	21.25	23.32	19.91	21.71	17.97	19.44	
10	23.40	25.81	21.91	23.99	19.88	21.59	
20	25.34	28.09	23.72	26.09	21.62	23.57	
25	25.94	28.80	24.27	26.74	22.15	24.18	
50	27.72	30.96	25.93	28.72	23.75	26.05	
100	29.44	33.06	27.52	30.64	25.28	27.87	
			FEBRUARY				
2	18.77	21.69	17.46	19.72	15.19	17.01	
5	22.49	26.14	21.19	24.05	18.59	20.91	
10	24.77	28.93	23.46	26.78	20.67	23.38	
20	26.73	31.54	25.50	29.32	22.55	25.68	
25	27.34	32.36	26.13	30.11	23.14	26.40	
50	29.18	34.85	28.03	32.53	24.88	28.59	
100	30.94	37.29	29.84	34.91	26.57	30.75	
			MARCH				
2	21.05	24.66	19.36	22.78	16.92	20.68	
5	25.70	30.29	23.69	28.05	20.53	25.29	
10	28.52	33.88	26.32	31.42	22.71	28.27	
20	31.09	37.27	28.72	34.60	24.69	31.08	
25	31.88	38.33	29.46	35.60	25.30	31.97	
50	34.25	41.59	31.68	38.67	27.13	34.71	
100	36.54	44.82	33.92	41.72	28.88	37.43	
			APRIL				
2	19.57	21.19	17.91	19.48	15.80	17.06	
5	23.86	25.91	21.80	23.78	19.24	20.83	
10	26.46	28.85	24.15	26.46	21.33	23.18	
20	28.82	31.57	26.28	28.93	23.22	25.34	
25	29.55	32.42	26.94	29.70	23.80	26.02	
50	31.74	34.99	28.91	32.04	25.55	28.06	
100	33.94	37.50	30.81	34.33	27.23	30.05	
			MAY				
2	17.76	19.65	16.51	18.66	14.03	15.70	
5	21.90	24.31	20.09	22.81	17.17	19.29	
10	24.43	27.26	22.26	25.43	19.07	21.56	
20	26.75	30.01	24.23	27.87	20.81	23.67	
25	27.46	30.87	24.84	28.64	21.34	24.33	
50	29.61	33.50	26.66	30.97	22.95	26.34	
100	31.69	36.08	28.41	33.26	24.50	28.32	
			JUNE				
2	14.18	15.44	13.13	14.35	11.29	12.01	
5	16.83	18.39	15.55	17.06	13.50	14.40	
10	18.42	20.20	16.99	18.72	14.82	15.86	
20	19.93	21.86	18.27	20.24	16.01	17.19	
25	20.27	22.38	18.67	20.71	16.37	17.60	
50	21.56	23.93	19.84	22.13	17.46	18.85	
100	22.79	25.44	20.96	23.51	18.50	20.05	

Note: The first number under each duration for every return period is the best estimate of wind velocity from Iognormal distribution and the second number is the tipper 95 percent confidence limit (1 mile = 1.609 km)

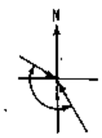
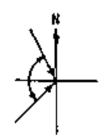
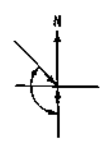
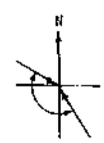
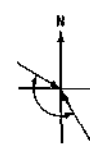
(continued on next page)

Table 1. (Continued)

EVANSVILLE, INDIANA

RETURN PERIOD	6 HOUR		12 HOUR		24 HOUR		PREVAILING WIND
			JANUARY				
2	22.67	23.36	20.57	21.47	18.29	18.86	
5	25.26	26.06	22.65	23.68	20.07	20.72	
10	26.73	27.63	23.83	24.97	21.08	21.79	
20	28.01	29.00	24.84	26.09	21.96	22.72	
25	28.39	29.41	25.15	26.43	22.20	23.01	
50	29.52	30.64	26.04	27.44	22.96	23.84	
100	30.58	31.80	26.86	28.39	23.66	24.62	
			FEBRUARY				
2	23.52	26.47	21.58	23.80	18.70	20.34	
5	28.19	31.87	25.00	27.67	22.07	24.08	
10	30.99	35.24	26.99	30.02	24.06	26.37	
20	33.51	38.37	28.76	32.16	25.85	28.46	
25	34.29	39.34	29.30	32.83	26.39	29.10	
50	36.60	42.30	30.89	34.82	28.01	31.05	
100	38.81	45.20	32.39	36.75	29.56	32.94	
			MARCH				
2	24.21	26.90	21.96	23.79	19.39	21.48	
5	28.26	31.52	25.71	27.93	22.80	25.35	
10	30.64	34.36	27.91	30.45	24.81	27.72	
20	32.76	36.96	29.88	32.74	26.60	29.90	
25	33.40	37.76	30.47	33.45	27.15	30.58	
50	35.32	40.18	32.25	35.57	28.77	32.62	
100	37.13	42.53	33.94	37.63	30.32	34.60	
			APRIL				
2	23.63	27.02	20.73	22.90	18.33	20.02	
5	28.57	32.83	24.24	26.76	21.38	23.43	
10	31.55	36.50	26.31	29.18	23.17	25.50	
20	34.24	39.93	28.14	31.40	24.76	27.39	
25	35.07	41.00	28.70	32.08	25.24	27.97	
50	37.55	44.27	30.37	34.14	26.68	29.73	
100	39.94	47.48	31.95	36.14	28.05	31.43	
			MAY				
2	21.60	22.72	18.48	19.64	15.79	16.81	
5	24.82	26.16	21.86	23.29	18.26	19.49	
10	26.69	28.20	23.87	25.51	19.70	21.10	
20	28.35	30.03	25.66	27.52	20.98	22.55	
25	28.85	30.60	26.21	28.14	21.37	22.99	
50	30.33	32.27	27.85	30.01	22.52	24.32	
100	31.73	33.87	29.41	31.82	23.61	25.60	
			JUNE				
2	18.81	21.20	16.38	19.11	13.62	15.06	
5	21.58	24.43	19.26	22.60	16.00	17.75	
10	23.19	26.40	20.96	24.79	17.40	19.40	
20	24.60	28.21	22.48	26.82	18.65	20.91	
25	25.03	28.77	22.94	27.45	19.03	21.38	
50	26.30	30.45	24.32	29.38	20.16	22.80	
100	27.50	32.08	25.63	31.27	21.24	24.17	

ST. LOUIS, MISSOURI

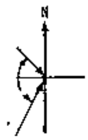
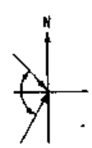
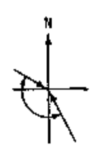
RETURN PERIOD	6 HOUR		12 HOUR		24 HOUR		PREVAILING WIND
			JANUARY				
2	22.74	24.16	20.93	21.43	18.22	19.30	
5	25.21	26.85	22.95	23.52	20.32	21.58	
10	26.61	28.43	24.08	24.71	21.52	22.91	
20	27.83	29.83	25.06	25.75	22.56	24.10	
25	28.19	30.25	25.35	26.06	22.87	24.46	
50	29.26	31.52	26.21	26.98	23.79	25.53	
100	30.26	32.72	27.00	27.84	24.65	26.55	
			FEBRUARY				
2	24.04	25.30	21.92	23.04	19.06	20.19	
5	27.96	29.38	25.34	26.68	22.09	23.45	
10	30.09	31.82	27.33	28.85	23.86	25.40	
20	32.07	34.01	29.09	30.80	25.43	27.16	
25	32.67	34.68	29.62	31.39	25.91	27.70	
50	34.45	36.68	31.21	33.18	27.32	29.32	
100	36.13	38.61	32.71	34.88	28.66	30.87	
			MARCH				
2	26.48	29.55	23.77	26.08	20.85	22.51	
5	31.41	35.21	28.87	31.79	24.74	26.79	
10	34.34	38.71	31.96	35.36	27.05	29.41	
20	36.97	41.93	34.76	38.65	29.13	31.80	
25	37.77	42.93	35.62	39.68	29.76	32.54	
50	40.17	45.97	38.20	42.80	31.65	34.77	
100	42.45	48.92	40.68	45.85	33.46	36.93	
			APRIL				
2	25.19	26.73	22.79	24.20	19.32	20.41	
5	28.78	30.61	26.34	28.04	22.38	23.69	
10	30.86	32.92	28.42	30.34	24.17	25.66	
20	32.69	34.90	30.25	32.41	25.76	27.42	
25	33.24	35.63	30.81	33.04	26.24	27.97	
50	34.88	37.52	32.46	34.94	27.66	29.59	
100	36.42	39.32	34.02	36.76	29.01	31.14	
			MAY				
2	22.27	25.10	19.69	21.17	16.67	17.98	
5	26.41	29.90	22.84	24.63	19.04	20.60	
10	28.87	32.88	24.69	26.72	20.41	22.17	
20	31.07	35.63	26.32	28.61	21.62	23.58	
25	31.74	36.49	26.82	29.19	21.98	24.01	
50	33.75	39.09	28.30	30.93	23.06	25.31	
100	35.67	41.62	29.69	32.61	24.08	26.55	
			JUNE				
2	17.80	18.70	16.15	17.36	14.33	15.11	
5	19.62	20.66	17.77	19.15	15.94	16.84	
10	20.65	21.79	18.67	20.20	16.86	17.85	
20	21.54	22.80	19.46	21.14	17.65	18.75	
25	21.90	23.10	19.69	21.42	17.89	19.02	
50	22.58	24.00	20.39	22.27	18.59	19.83	
100	23.31	24.85	21.02	23.08	19.24	20.60	

Note: The first number under each duration for every return period is the best estimate of wind velocity from lognormal distribution and the second number is the upper 95 percent confidence limit (1 mile = 1.609 km)

(concluded on next page)

Table 1. (Concluded)

MOLINE, ILLINOIS

RETURN PERIOD	6 HOUR		12 HOUR		24 HOUR		PREVAILING WIND
			JANUARY				
2	26.27	27.88	23.94	25.25	20.39	21.42	
5	31.52	33.54	28.37	29.99	23.65	24.89	
10	34.67	37.00	31.01	32.86	25.56	26.97	
20	37.51	40.17	33.37	35.47	27.25	28.83	
25	38.38	41.15	34.09	36.27	27.76	29.40	
50	40.99	44.10	36.24	38.69	29.29	31.11	
100	43.48	46.96	38.29	41.02	30.73	32.75	
			FEBRUARY				
2	25.55	26.65	23.76	25.68	21.15	22.74	
5	30.32	31.68	28.10	30.46	25.12	27.09	
10	33.16	34.73	30.69	33.39	27.49	29.75	
20	35.71	37.48	32.99	36.06	29.62	32.18	
25	36.48	38.33	33.70	36.89	30.26	32.94	
50	38.81	40.87	35.80	39.38	32.20	35.20	
100	41.02	43.32	37.80	41.79	34.05	37.40	
			MARCH				
2	28.35	31.25	26.35	28.66	23.17	24.73	
5	34.26	37.90	32.12	35.05	28.08	30.04	
10	37.82	42.05	35.62	39.03	31.04	33.32	
20	41.05	45.89	38.79	42.71	33.72	36.33	
25	42.04	47.09	39.77	43.86	34.54	37.26	
50	45.01	50.72	42.71	47.34	37.01	40.09	
100	47.85	54.26	45.54	50.75	39.38	42.84	
			APRIL				
2	28.49	31.41	26.20	28.70	22.73	24.45	
5	34.04	37.67	31.41	34.52	26.91	29.02	
10	37.36	41.54	34.53	38.12	29.38	31.81	
20	40.34	45.10	37.35	41.44	31.60	34.35	
25	41.75	46.21	38.21	42.47	32.28	35.14	
50	43.98	49.56	40.78	45.59	34.30	37.50	
100	46.58	52.83	43.25	48.63	36.22	39.79	
			MAY				
2	26.14	32.13	23.51	27.95	20.05	22.76	
5	32.50	40.26	28.61	34.73	24.36	27.79	
10	36.42	45.58	31.70	38.26	26.97	30.96	
20	40.01	50.67	34.50	42.05	29.34	33.93	
25	41.12	52.29	35.36	43.25	30.07	34.85	
50	44.47	57.28	37.95	46.91	32.25	37.68	
100	47.72	62.28	40.44	50.55	34.35	40.46	
			JUNE				
2	21.30	22.68	19.19	20.93	16.15	17.37	
5	24.45	26.09	22.04	24.12	18.34	19.78	
10	26.28	28.13	23.69	26.04	19.60	21.22	
20	27.99	29.96	25.16	27.79	20.71	22.51	
25	28.39	30.53	25.60	28.32	21.04	22.91	
50	29.83	32.20	26.91	29.93	22.03	24.09	
100	31.19	33.81	28.14	31.48	22.96	25.22	

Note: The first number under each duration for every return period is the best estimate of wind velocity from lognormal distribution and the second number is the upper 95 percent confidence limit (1 mile = 1.609 km)

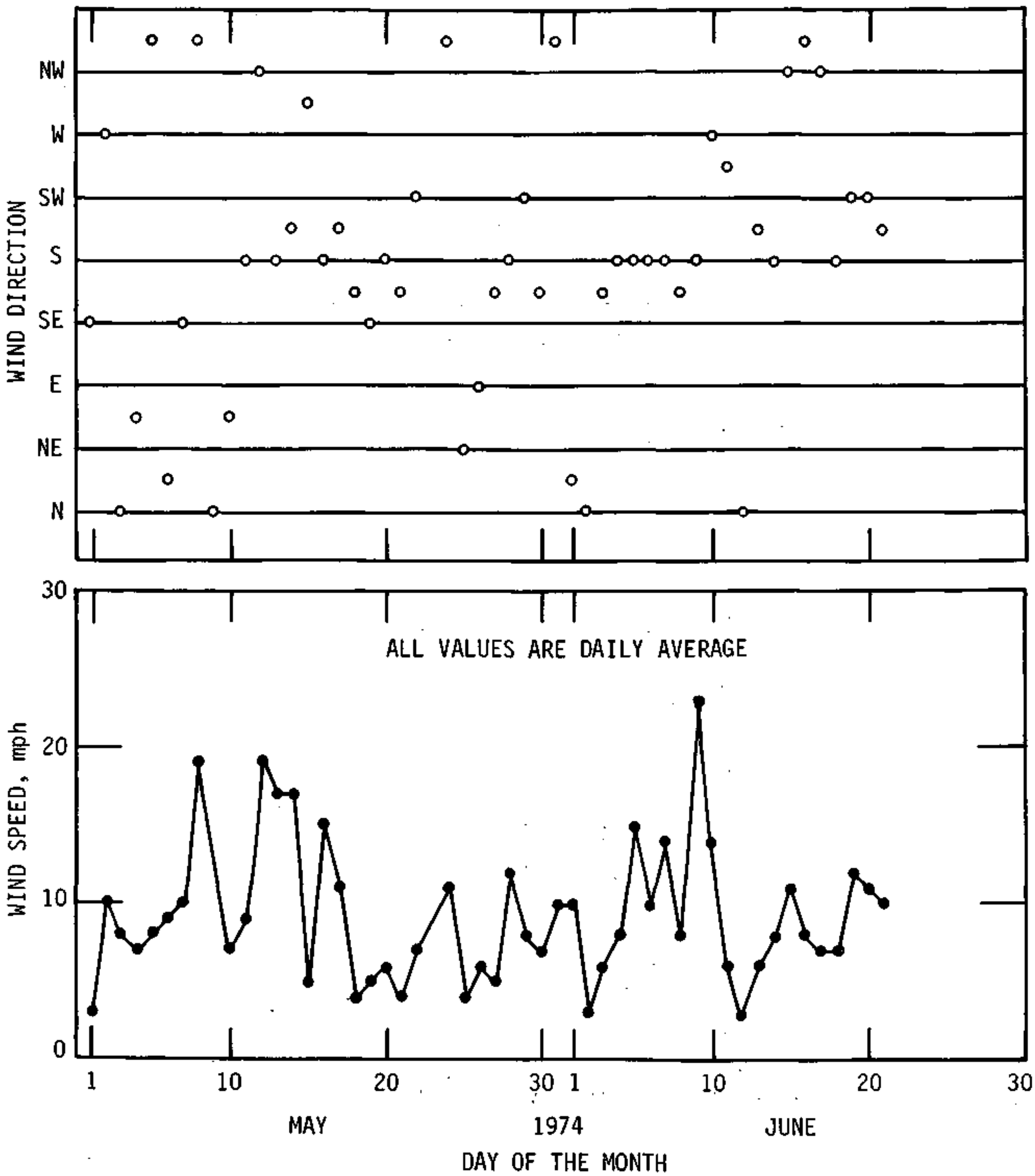


Figure 8. Wind records from Coles Creek Access Area in Carlyle Lake

Cermak and Koloseus (1953) in the Lake Hefner model study indicated that the vertical wind velocity profile over a lake surface follows a logarithmic distribution. Data collected from Coles Creek Area at 10 feet (3.05 m), U_{10} , and 32.8 feet (10 m), $U_{32.8}$, indicated that U_{10} was in the order of 60 to 99 percent of $U_{32.8}$. These variations remained the same for both steady and turbulent wind conditions. Since wind velocity data were collected at only two elevations, computations to test the validity of the logarithmic distribution could not be justified.

Braslavskii and Vikulina (1963) compared wind velocity data collected from a lake shore and over the reservoir to those collected in nearby meteorological stations. They have shown that, in general, the average wind velocity over a large reservoir is greater than that recorded on a shore station. Also it was reported that a correction factor should be included to account for the physical location of an existing wind speed measuring station. Wind data collected at Saddle Dam No. 2 and the Coles Creek Access Area (figure 5) were compared to check for any variations in the magnitudes of wind speeds. For this set of data for May and June of 1974 no significant variation was observed. However, it must be remembered that the location of the station (relief in and around the station) plays a very important role in the wind structure recorded at any station.

Wind-Generated Waves

The digitized wind-generated wave data were analyzed statistically to determine and interpret various wave characteristics.

Autocorrelation Coefficients and Correlograms. Autocorrelation coefficients were computed by using equation 1 for various lag times in seconds. The correlograms for runs 4, W4, 5, and W5 are shown in figure 9.

The time series realization of the wind-generated waves remained stationary in the mean. The correlograms shown in figure 9 indicate that wind-generated waves are positively correlated

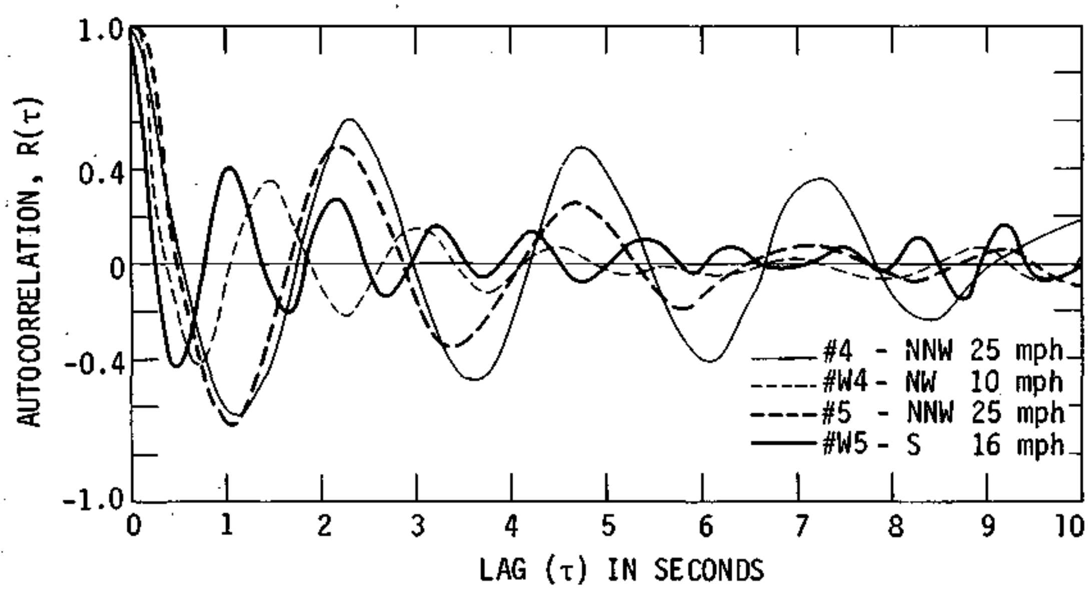


Figure 9. Correlograms of wind-generated waves

for lag of less than one-half second and that the autocorrelation coefficient changes sign with some periodicity. This indicates that the water surface fluctuates about a mean level without any trend. Figure 9 also indicates that for runs 4 and 5 with wind speed of 25 mph (40.2 km/hr) from the NW, the wind-generated waves have a fundamental period of about 2.5 seconds. Whereas, for run W4 with wind speed of 10 mph (16.09 km/hr) from the NW, the fundamental period decreases to about 1.5 seconds. Thus the fundamental period of wind-generated waves is at least a function of wind velocity and the fetch.

Energy Spectra. The spectral density function of wind-generated waves were computed with equation 5. Hamming weighting factors were used to smooth the raw spectra. The length of records varied from 340 to 925 sample points or 68 to 185 seconds.

The computed wave energy spectra versus frequency were plotted on log-log paper as shown in figure 10. The wind speed and direction for every run are also shown. In the higher frequency range, i.e., after the energy spectra attained the peak value, a relationship existed between $\Phi(f)$ and the frequency. These ranges of frequency are called equilibrium frequency after Phillips (1958). In the equilibrium range, $\Phi(f)$ is related to f as follows

$$\Phi(f) \propto f^{-5} \tag{16}$$

Figure 10 shows that in the equilibrium range, the energy spectra are approximately parallel to the f^{-5} line.

The other observations that can be made from figure 10 are: 1) the peak energy increases with an increase in wind velocity; 2) the peak energy corresponding to higher wind velocity occurs at a lower frequency whereas the peak energy corresponding to lower wind velocities occurs at a comparatively higher frequency. Therefore, in order to compare the wave energy spectra with published results, some sort of nondimensionalizing should be done.

Similarity Characteristics of Wave Spectra. Liu (1968) in a study of Lake Michigan wave data indicated that developing similarity criteria should be an important feature in the study of wave spectra. He used the normalization technique suggested by Hidy and Plate (1965) to nondimensionalize $\Phi(f)$ and f. The functional relationship between $\Phi(f)$ and f is expressed as

$$\Phi(f)\, f_m/\sigma^2 = F(f/f_m) \tag{17}$$

where f_m is the frequency at which $\Phi(f)$ is maximum and σ^2 is the variance of the water surface.

Nondimensional wave spectra and frequency were computed for six runs with varying wind speed and direction. These computed values are plotted in figure 11 on a log-log scale. The average line from Liu's (1968) data is also shown. As can be seen, in the equilibrium range the average line from Liu's data could also be taken as the average line from the present study. The slope of this average line is almost parallel to the typical $(f/f_m)^{-5}$ line indicating similarity characteristics of deep water waves (Liu, 1968).

Wave Heights. Wave heights, defined as the difference in elevation between two consecutive peaks and valleys in a time series realization similar to the one shown in figure 3, were digitized and frequency analyses were performed.

The nondimensional wave heights $\eta = H/\bar{H}$ and the probability of occurrence for three sets of data are plotted in figure 12. The solid line is the Rayleigh distribution (equation 6). Visual

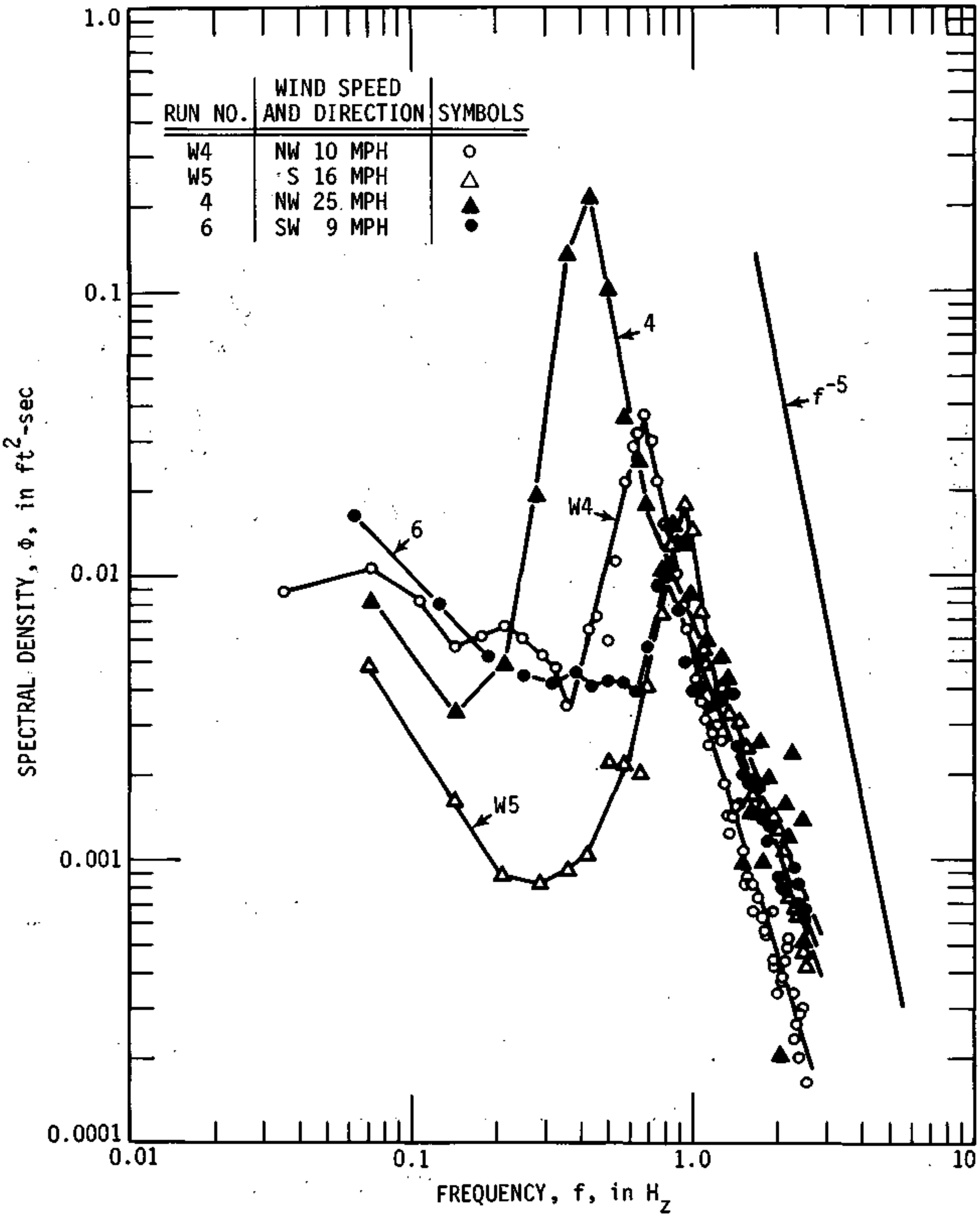


Figure 10. Wind-generated wave spectra

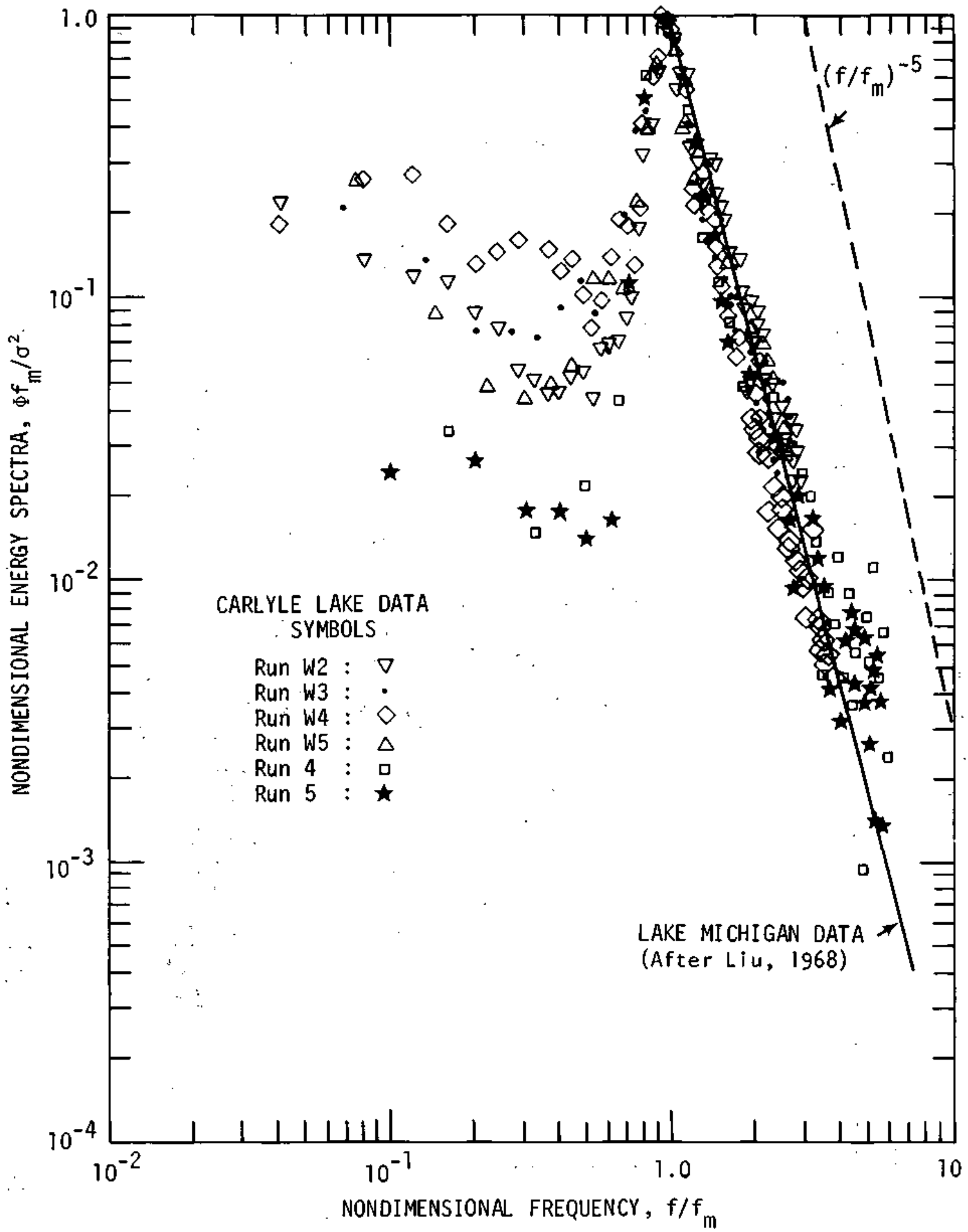


Figure 11. The similarity of wave energy spectra

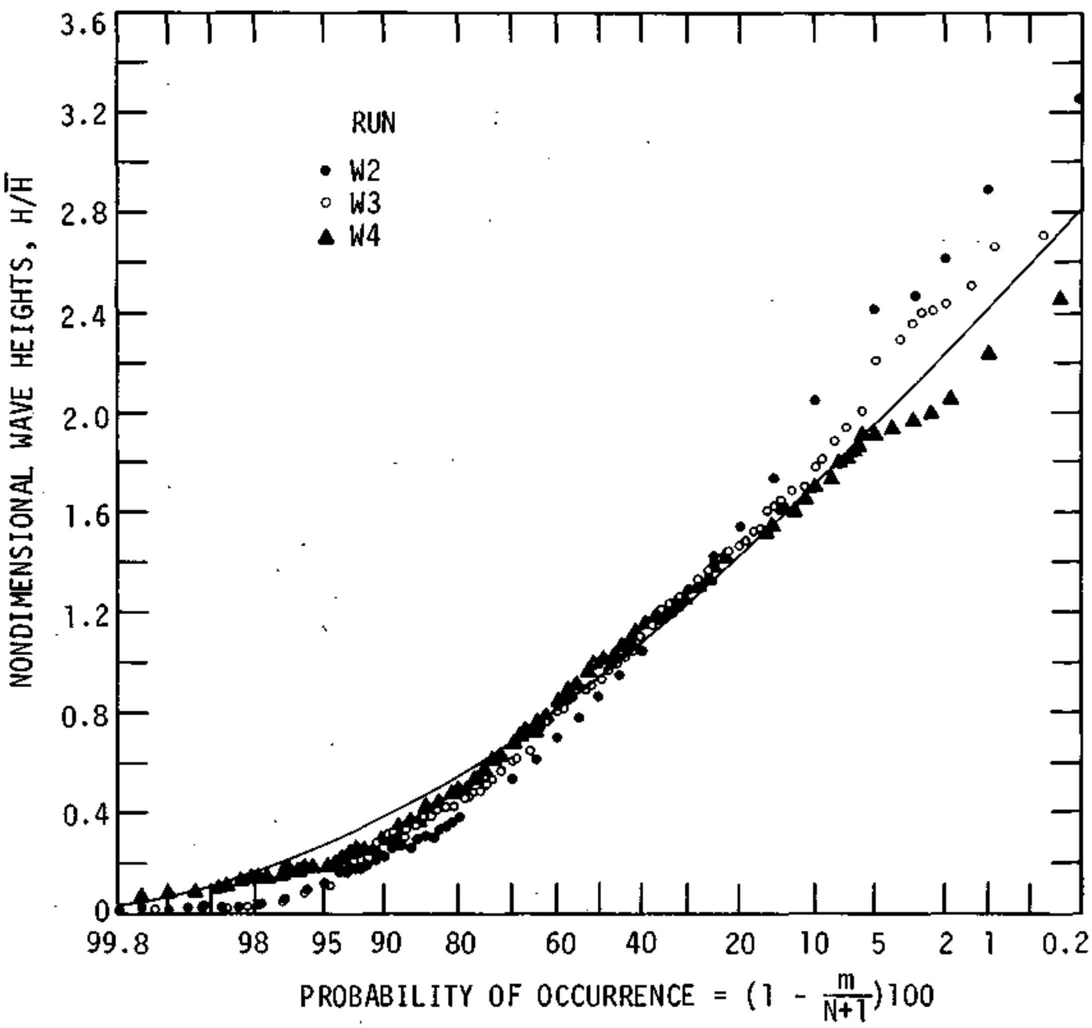


Figure 12. Frequency distribution of nondimensional wave heights

inspection indicates a good fit. A χ^2 test for goodness of fit indicated that at a 95 percent confidence limit the Rayleigh distribution fits these observed values of wave heights. A similar fit was also observed for other runs.

. The coefficient of variation, $\sigma/\bar{H}$, of the wave heights varied from 0.5 to 0.9 with an average value of 0.7 for all the runs.

Significant Wave Height. Equation 12 can be used to estimate the significant wave height from a known effective fetch. To facilitate the computational procedure, a nomograph was developed with equation 12 and is shown in figure 13. In the development of figure 13, it was assumed that the wind blows in the same direction of the fetch thus making cos Φ equal to 1.

Table 2 shows a comparison of the measured and estimated significant wave heights in Carlyle Lake for seven runs. Three different methods were used to compute the significant wave heights. Although none of the methods are very accurate the wave heights predicted by equation 12 compare closely with the maximum wave heights measured in the field.

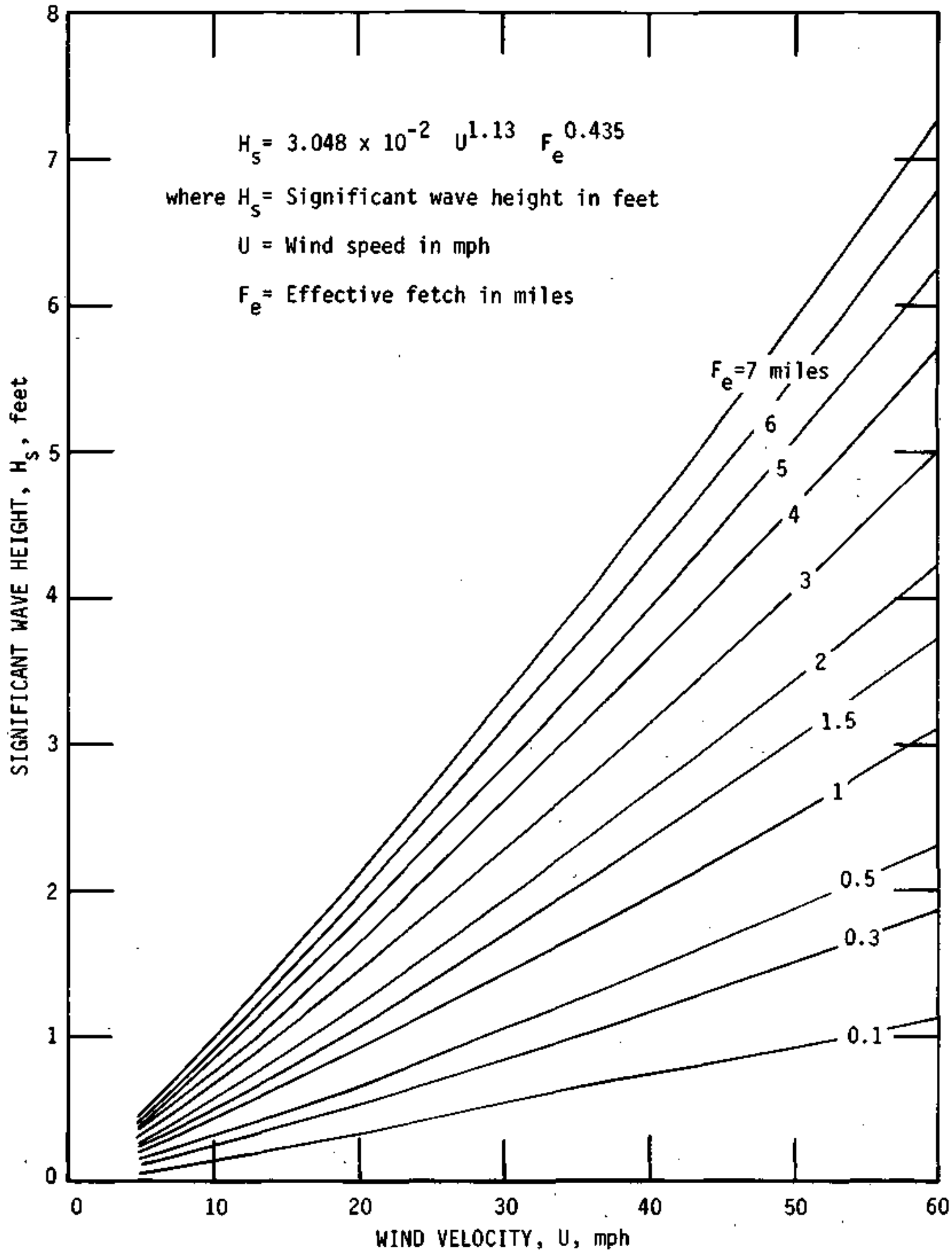


Figure 13. Relationship between significant wave height, wind velocity, and effective fetch

Table 2. Measured and Estimated Significant Wave Heights

1 mile = 1.609 km, 1 foot = 0.305 m

Run	*Wind speed (mph)*	*Wind direction*	*Measured wave height* *Average (ft)*	*Significant (ft)*	*Maximum (ft)*	*Computed significant wave height (ft) by method* *1**	*2*	*3*
W2	16	S	0.232	0.284	0.755	0.730	2.31	0.90
W3	16	S	0.228	0.287	0.621	0.730	2.19	0.70
W4	10	NW	0.346	0.432	0.855	0.598	2.04	0.43
W5	16	S	0.268	0.332	0.686	0.730	2.31	0.90
4	25	NW	0.443	0.497	1.756	1.684	2.67	1.70
5	25	NW	0.585	0.704	2.301	1.684	2.57	1.60
6	9	SW	0.234	0.287	0.683	0.598	2.01	0.40

*Method 1 — by equation 12
Method 2 — Molitor-Stevenson equation ASCE (1948)
Method 3 — following U.S. Army Corps of Engineers (1966)

Table 3. Wind Tide in Carlyle Lake with SW or SSW Wind and Fetch of 9.15 Miles

1 mile = 1.609 km, 1 foot = 0.305 m

Wind speed, U (mph)	*Wind tide, S (feet)*
10	0.073
15	0.163
20	0.291
30	0.652
40	1.160
50	2.810

Wind Tide. Data on wind tide were not collected from the field. However, a sample computation was performed to demonstrate the significance of wind tide in a large lake similar to Carlyle Lake. The wind tide S was computed by equation 13, with the coefficient K equal to 1/1400 (Saville et al., 1962), F in miles, U in mph, and S in feet. Table 3 shows the computed values of S for different wind speeds in Carlyle Lake.

Boat-Generated Waves. Data on boat-generated waves were analyzed by the same procedure outlined for wind-generated waves.

Autocorrelation coefficients were computed for various lag times in seconds and correlograms were developed for all runs. An examination of the time-series realization and the test for dependency of the mean and covariance on the positions along the time series indicated that boat-generated waves are nonstationary in nature. No periodic variation in the waves was discernible from the cprrelograms.

The Fourier coefficient C_n^2 was computed by using the SOUPAC program.

Figure 14 shows the relationship between C_n^2 and frequency f in Hertz for runs 1, 2, and 3. Since the value of $C_n^2/2$ for any particular frequency f gives the contribution of variance by the nth harmonic, it is clear that for run 1, the main contribution of variance is from frequencies in the range of 0.3 to 0.7 Hz. The variance spectra for runs 2 and 3 are very similar in magnitude and shape and do not contain a sharp peak similar to that for run 1.

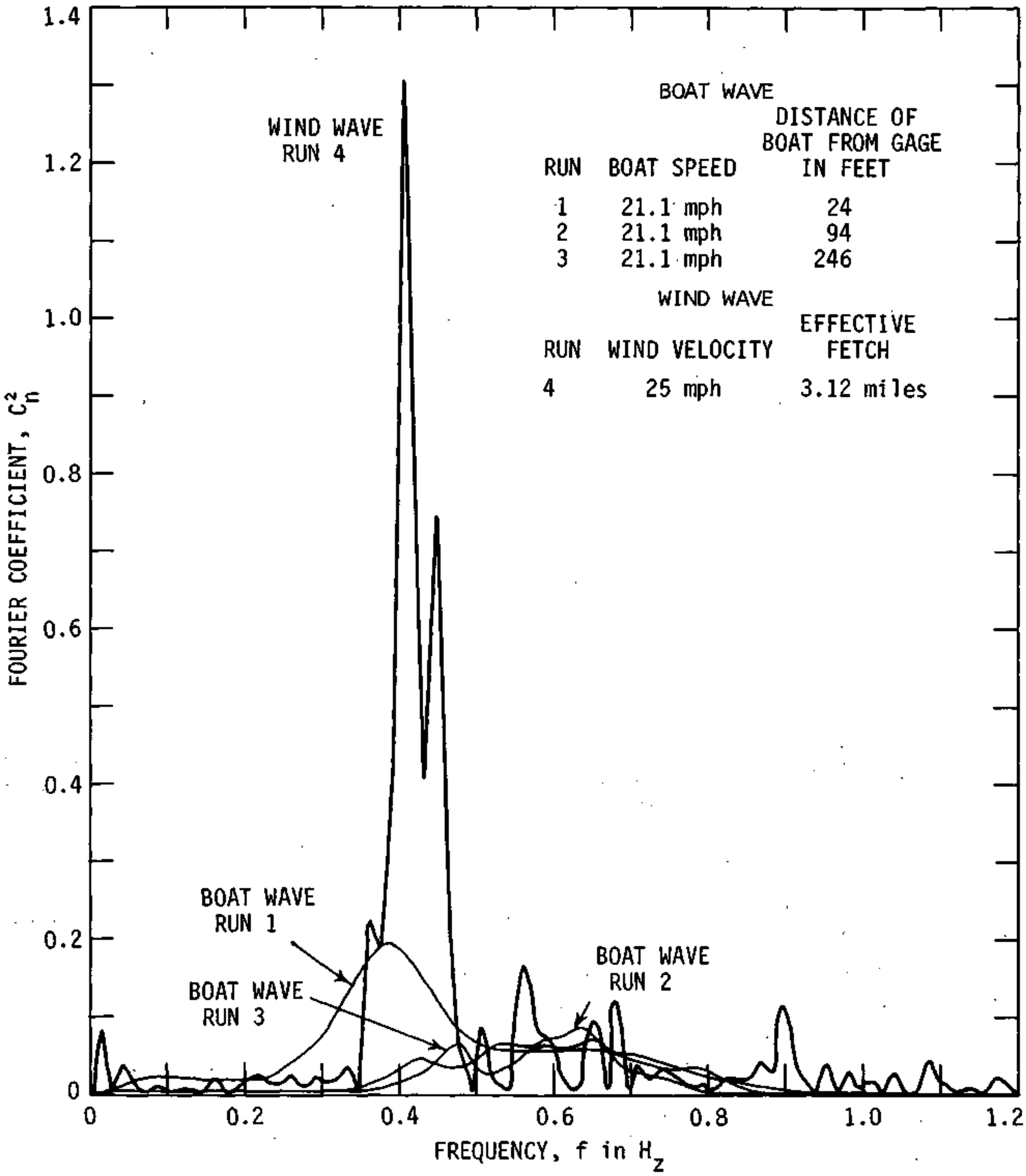


Figure 14. Fourier coefficient versus frequency for boat-generated waves

Figure 14 also shows, for comparison to boat-generated waves, one plot of variance spectra for wind-generated waves in Carlyle Lake. The peak energy for this condition (run 4) with a wind velocity of 25 mph occurs at a frequency of about 0.4 Hz, which corresponds closely to the occurrence of peak energy for boat waves of run 1 at 0.39 Hz. Waves produced by the boat at a close distance of 24 feet (7.32 m) for run 1 had a very sharp peak preceded and followed by small amplitude waves. This might account for the sharp peak in the energy diagram for run 1. On the other hand, because of the increased distance between the boat and the staff gages for runs 2 and 3, the waves reaching the staff gage were modified and smoothed, and the amplitudes were reduced by frictional resistance with an associated increase in the length of record. It is reasonable to expect that the lake shore must dissipate a major amount of wave energy in a shorter period of time whenever the boat is running close to the shore. Therefore, it might be advisable to ban any high-speed motor boat, say, within 100 feet (30.5 m) of the shore line.

Das (1969) has plotted H_m^2 versus the distance from the sailing line for cruiser model boat-generated waves in shallow water, where H_m is the maximum wave height in the time-series realization, and H_m^2 is proportional to the wave energy. Figure 15 shows the relationship between $(H_m/d_s)^2$ and X/L. The results of the present investigation were plotted with some of the results reported by Das. Here, d_s is the draft of the boat, X is the distance between boat and wave gage, and L is the length of the boat. The draft of the boat was used as the length parameter to compute the Froude numbers shown in figure 15. An examination of the plotted points from the field experiment show some systematic variation with Froude number. Straight lines can be drawn for the data of the present study. However, the data reported by Das (1969) appear to be nonlinear in this log-log plot. Das conducted his experiment in a laboratory flume with constant water depth. The depth of water in his experiments was twice that of the draft of the model boat. The field data presented were collected from a lake where the depth of water varied from 6 to 10 times the draft of the boat. Moreover, in field experiments all unknown variables cannot be accounted for as most of them can be in laboratory experiments. This might explain the overlappings of the field and laboratory data for different Froude numbers.

An equation was developed for the field data and is given by

$$(H_m/d_s)^2 = (3.45) \times (10^{-2}) V^{1.174} (X/L)^{-0.915} \tag{18}$$

where V is the boat speed in miles per hour.

Frequency analyses of the wave heights were performed. Figure 16 shows dimensionless wave heights 17 versus the percent of wave heights equal to or less than η. For comparative purposes, the Rayleigh distribution is also plotted. Visual inspection indicates that except for higher values of η in run 1, the remaining values of 17 closely follow the Rayleigh distribution. A χ^2 test for goodness of fit indicated that at a 95 percent confidence limit the Rayleigh distribution fits these observed values of wave heights. However, the probability of the sample χ^2 for run 1 is high compared with corresponding values for runs 2 and 3.

Riprap Size Distribution

Data on riprap size distributions for ten samples were collected from Carlyle and Rend Lakes. One of the samples from Carlyle Lake is for filter or bedding materials. A size analysis was done

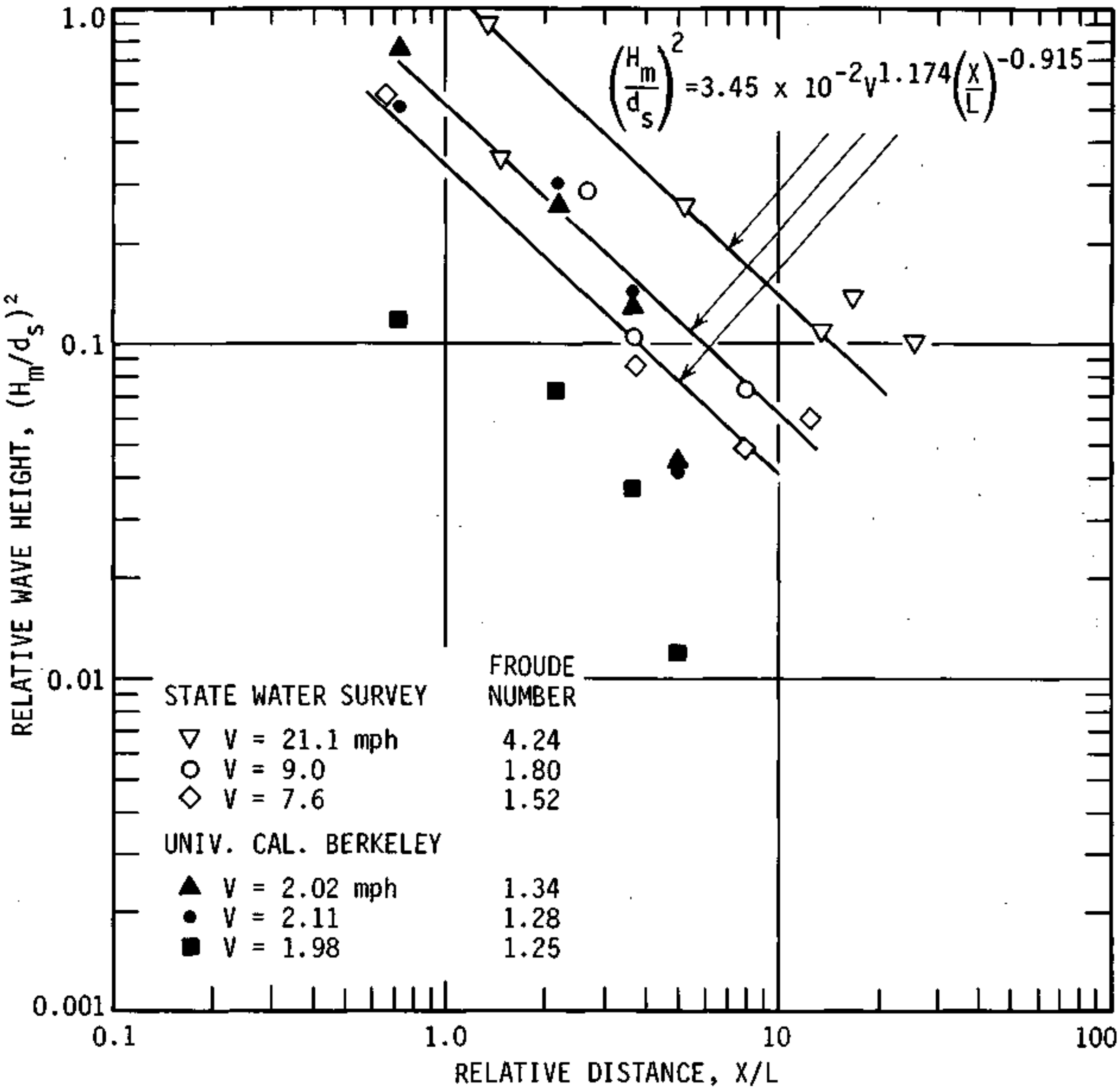


Figure 15. Relative wave height versus relative distance (boat-generated waves, 1 mph = 1.609 km/hr)

for each sample with intermediate axis of each stone as the representative size of that particular stone. Figure 17 shows the size distribution of all the samples. The d_{50} size of the riprap materials varies from 3 to 11 inches (76.2 to 279.4 mm) with the maximum size up to 40 inches (1.016 m). Some of these samples were collected from unstable banks where bank failure was either imminent or failure had already occurred. The d_{50} size of the filter material is equal to 1.38 inches (35.05 mm] The uniformity coefficient of all the riprap particles varied from 1.78 to 3.96 with an average value of 2.50. The standard deviation varied from 1.50 to 2.45 with an average value of 1.80.

The shape factors of all 1210 particles were computed using the relation

$$SF = c/(a/b)^{1/2} \tag{19}$$

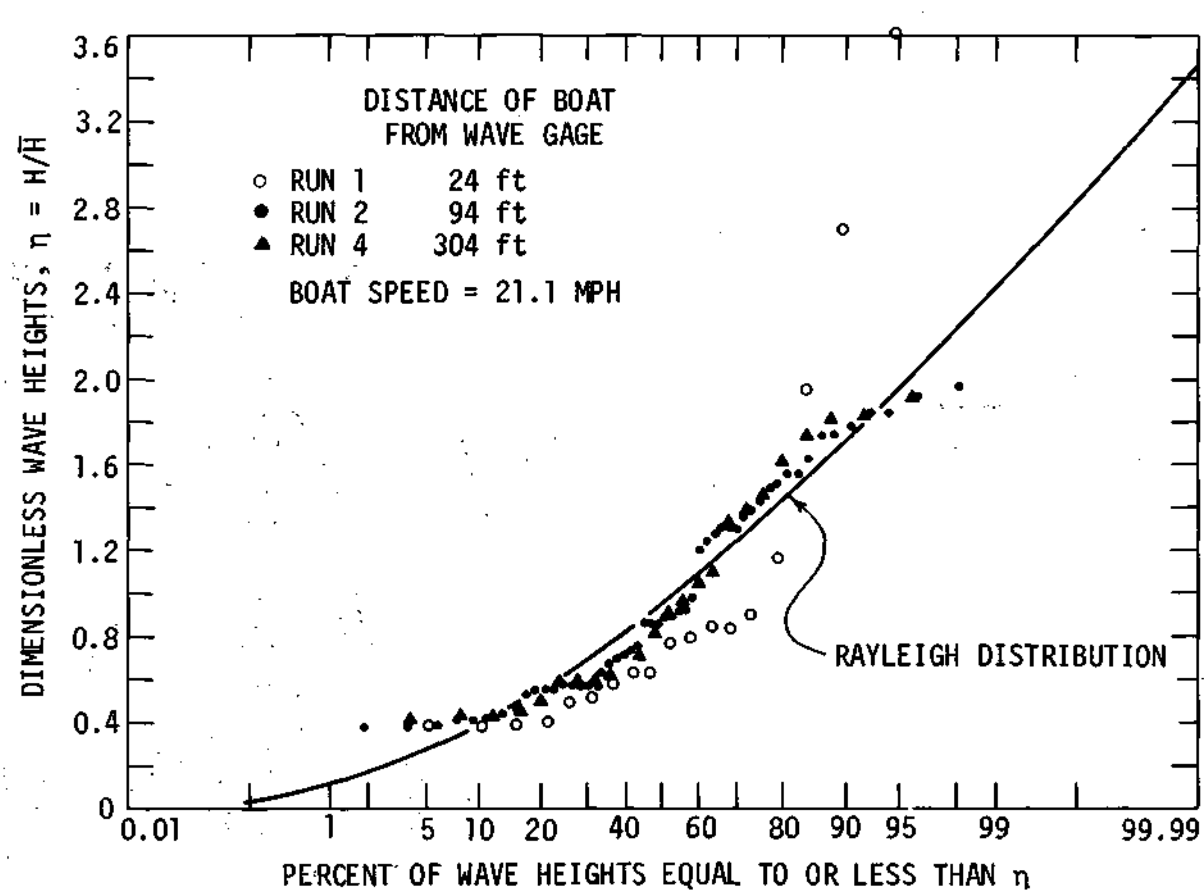


Figure 16. Dimensionless wave heights η versus percent of wave heights equal to or less than TJ (1 ft = 0.305 m)

where *a, b,* and *c* are the longest, intermediate and shortest axes of the three mutually perpendicular axes of the particle. For all these riprap particles including the filter materials, the *SF* varied from 0.505 to 0555 with an average value of 0.524. The standard deviation of *SF* varied from 0.158 to 0.196 with an average value of 0.167. With a sphere having a *SF* of 1.0 and well rounded gravel in stream beds having a *SF* of about 0.7, these riprap particles are undoubtedly angular in shape. Angular particles are more stable than rounded or flat particles. Therefore, in the stabilization of lake shores with riprap materials, angular particles should be given preference over rounded or flat particles.

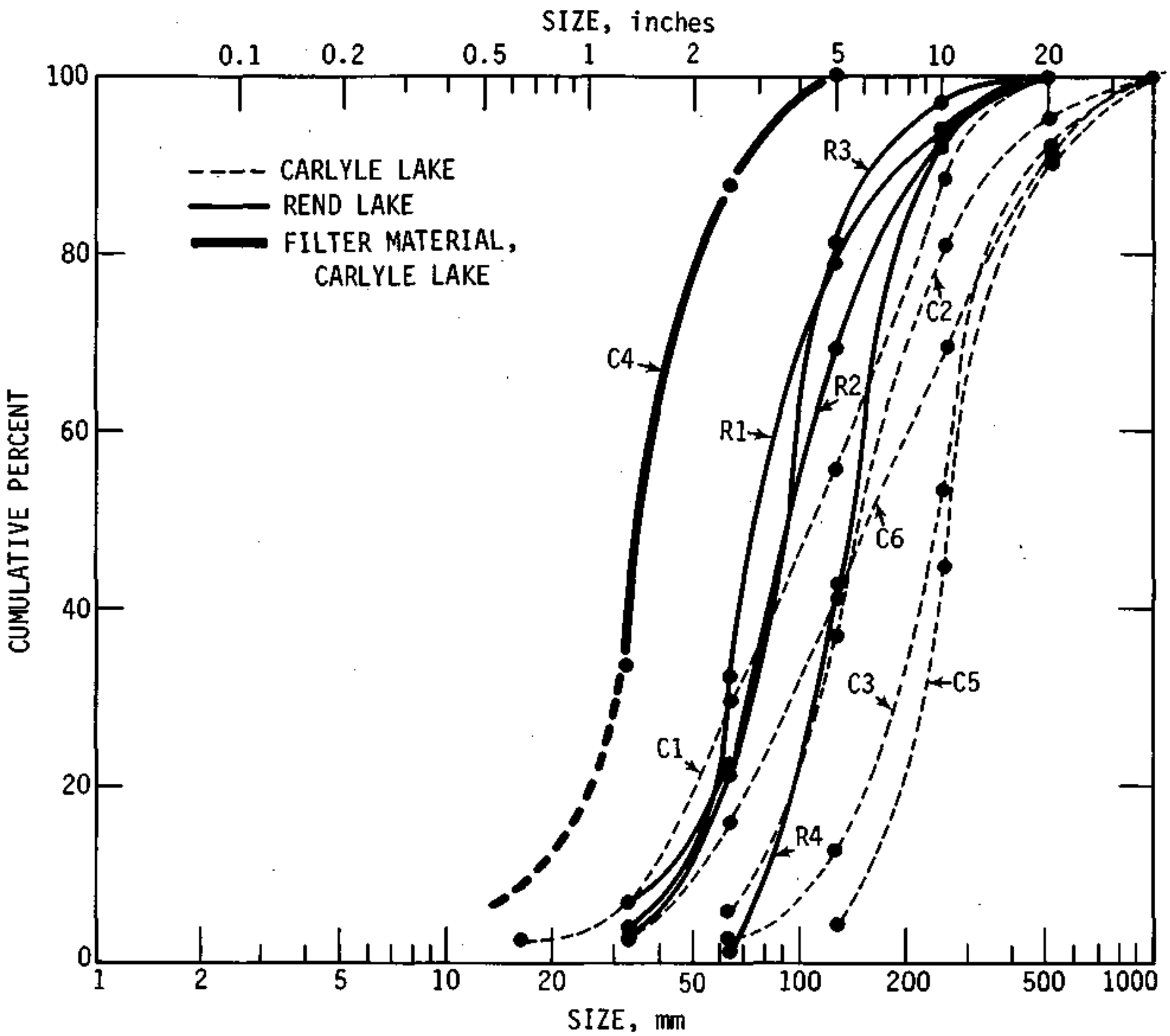


Figure 17. Size distribution of riprap and filter materials

STABILIZATION OF LAKE SHORES

Selection of Riprap Particles

It was already pointed out that riprap materials can be utilized to stabilize lake shores against wave action. The stable weight of the median size of the riprap particle can be computed by using equation 15 once the value of $\overline{K}/\Upsilon^2$ is known. This unknown coefficient was calculated from available laboratory and field data (Hedar, 1953; Saville, 1967). The computed values of $\overline{K}/\Upsilon^2$ varied from 0.367 to 0.454 pounds per cubic feet with an average value of 0.388 pounds per cubic feet. A nomograph was developed using equation 15 substituting the average value of $\overline{K}/\Upsilon^2$ equal to 0.388 pounds per cubic feet. This nomograph is shown in figure 18.

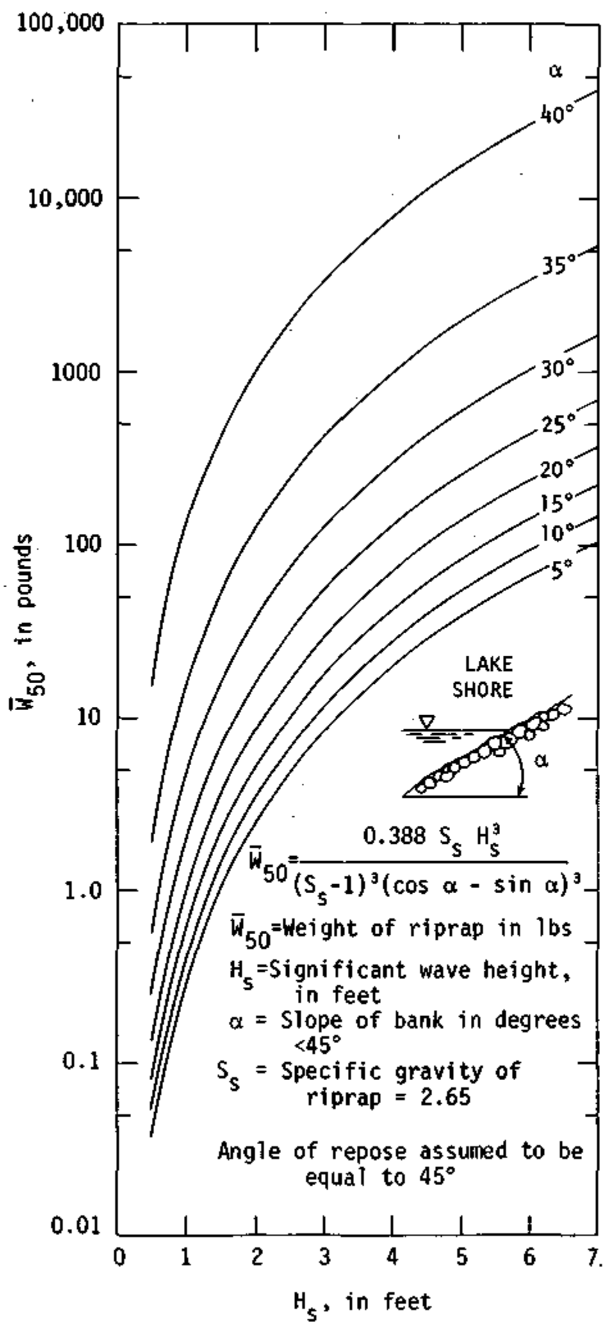


Figure 18. Nomograph for estimating $\overline{W}_{50}$ in pounds

The validity of equation 15 (figure 18) for field conditions was checked by utilizing the data collected from Carlyle and Rend Lakes. The maximum wind velocity of a 50-year return period was taken from table 1 for the nearest climatological station. Table 4 shows the computed and measured d_{50} sizes of the riprap particles at different locations in the Carlyle and Rend Lakes for varying lake shore conditions. The correlation between the estimated and measured d_{50} sizes is excellent. In all the unstable reaches the estimated d_{50} sizes are larger than those measured in the field indicating an unstable condition. However, in case of the reaches with stable banks, the variations between measured and estimated d_{50} sizes are very small. Therefore, it is quite reasonable to expect that the d_{50} size estimated from figure 18 for an expected wind condition should be satisfactory for the anticipated condition.

Gradation of Riprap Particles

The gradation of riprap particles is also important to the stability of the lake shores. Uniform particles are comparatively unstable compared to well graded particles. The U.S. Army Corps of Engineers in their Engineering Manual (1971) recommends the

Table 4. Computed and Measured Riprap Sizes in Carlyle and Rend Lakes

1 mile = 1.609 km; 1 foot = 0.305 m; 1 inch = 25.4 mm

Sample Number	Wind direction	Max. wind velocity 50-yr return period in mph (table 1)	Climatological sta. (table 1)	Effective fetch by eq. 10 in miles	Computed wave height by eq. 12 in feet	Computed W_{50} size in lbs. by eq. 15 or fig. 18	Equivalent diameter of a sphere in inches	Measured d_{50} size in inches	Remarks
Carlyle Lake									
C1	WNW ~S	40.17	St. Louis	3.25	3.19	11.61	6.14	4.33	generally stable occasional failure
C2	"	40.17	" "	2.71	3.06	16.36	6.88	5.79	stable
C3	"	"	" "	2.97	3.18	111.89	13.07	9.72	newly placed riprap
C5	"	"	" "		3.06	24.19	7.84	10.31	" " "
C6	NNW	31.15	" "	4.52	3.21	17.35	6.96	6.22	stable
Rend Lake									
R1	WNW ~SSE	38.86	Evansvllle &St. Louis	3.03	3.09	26.93	8.13	2.95	unstable, failure evident
R2	NNW ~SW	35.53	" "	3.25	2.88	21.23	7.51	3.66	" " " "
R3	"	35.53	" "	"	"	"	"	"	" " " "
R1	WNW ~SSE	38.86	" "	3.01	3.09	21.91	7.39	5.55	stable

maximum size of riprap to be equal to 4 d_{50} and the minimum size to be equal to 0.125 d_{50} with a thickness equal to 1.5 d_{50} with a minimum of 12 inches. The maximum and minimum sizes of riprap particles were recommended to be equal to 3.6 d_{50} and 0.22 d_{50} respectively, by the U.S. Army Coastal Engineering Research Center (1966). The U.S. Bureau of Reclamation (USBR, 1973) recommends the maximum and minimum sizes to be equal to about 2 d_{50} and 0.05 d_{50} , respectively, on a bank slope of 3:1. The thickness of riprap particles was recommended to be equal to 36 inches (0.915 m).

Bedding or Filter Materials

The other important consideration for the stability of banks is the selection of proper bedding or filter materials. Generally, the original bank materials consist of sand, silt, or clay or a combination of these materials. Repeated wave action will wash away these fine materials through the riprap blankets if not prevented in some way. Filter materials placed in between the original bank and the riprap particles prevents the wash out of these original bank materials. Therefore, the sizes of the filter materials must be larger than the original bank materials and smaller than the riprap particles. It must be emphasized here that the proper selection of the filter materials is as important as that of the riprap particles. A bank can and will fail by sloughing if the bank materials are allowed to wash away, even though the riprap particles have been properly selected.

The USBR (1973) recommends a blanket of crushed stone or gravel graded from 3/16 inches (4.76 mm) to 3 1/2 inches (88.9 mm) with a thickness equal to one-half the thickness of the riprap particles but not less than 12 inches (304.8 mm). The American Society of Civil Engineers Subcommittee on Slope Protection (ASCE, 1948) cites the work of Terzaghi and recommends that the ratio of d_{15} of riprap particles to d_{85} of filter particles should be less than 5. As an example, consider riprap samples C5 and C4, or C3 and C4 from Carlyle Lake in table 4 and figure 17. For samples C5 and C4, the ratio d_{15} of C5 to d_{85} of C4 is equal to 3.02 and similarly d_{15} of C3 to d_{85} of C4 is equal to 2.34. This computation indicates that the filter materials utilized at Carlyle Lake at sample locations C3, C4 and C5 satisfies the above criteria and most probably will prevent the wash out of the original bank materials. The U.S. Army Corps of Engineers (1971) recommends that filter materials be a minimum of 8 inches thick.

DESIGN PROCEDURE AND EXAMPLE

One of the main objectives of this project is to develop and suggest a design procedure to stabilize lake shores with stones or riprap against an anticipated wave action. The methodology already described contains enough materials to suggest such a procedure.

The equations and figures presented in this report should be valid not only for application in the state of Illinois but also for similar conditions in other states. The main difference for other states will be to develop plots or tables to estimate the wind velocity for any specified duration and return period.

In the course of field inspection it was repeatedly noted by the author that the quality of riprap particles played an important role in the stabilization of lake shores. Poor quality or soft ripraps were pulverized by heavy wave action in a number of different places. Therefore, even though the size of the riprap, bedding or filter materials, and bank slope might have been selected with care, the poor quality of the stones caused the banks to fail.

The following design procedure is suggested:

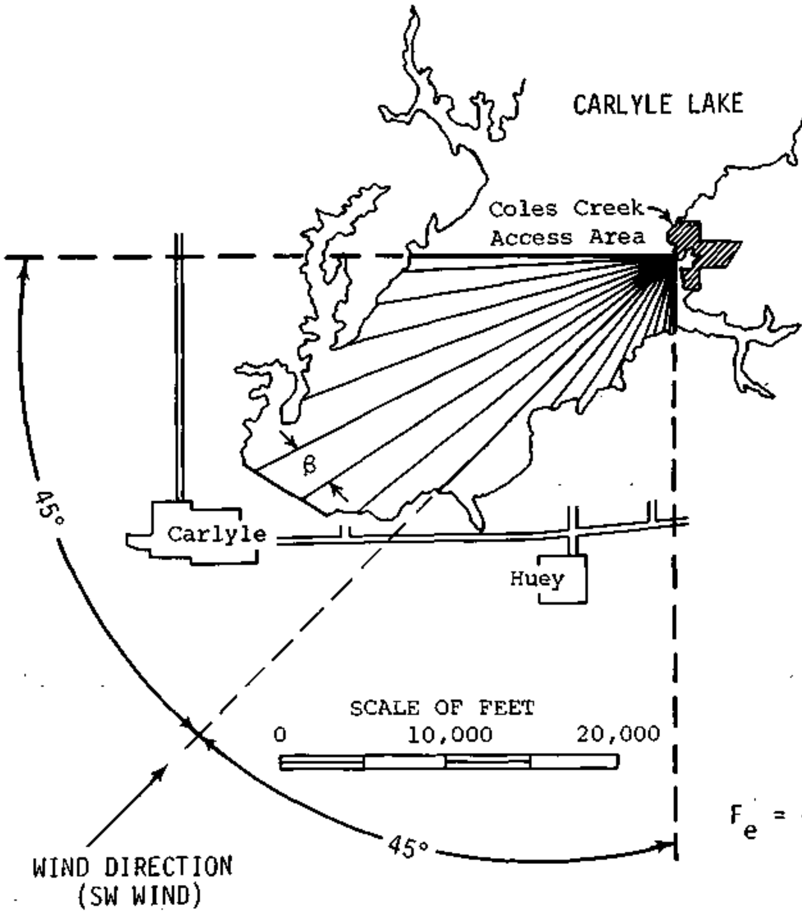
1. Select the location where stabilization is needed
2. Locate the closest climatological station for which wind data have already been analyzed and included in this report
3. Select design wind duration, return period, and wind velocity
4. Determine the maximum fetch of the water surface based on direction of the maximum wind velocity
5. If the shape of the lake is more or less rectangular, use equation 10 to estimate the effective fetch, F_e
   Equation 10 was developed for a width-fetch ratio of 0.05 to 0.6 for rectangular shaped water surfaces. These ranges of *W/F* ratio are within the general limits of large and small lakes in Illinois for winds blowing from south or southwest. Table 5 shows the width-fetch ratio of 26 representative lakes in the state. Surface areas vary up to 26,000 acres (1.522×10^8 square meters). In general, the *W/F* ratios are smaller than 0.6 except for a few cases when the wind blows from the S or SW. Equation 10 can therefore be utilized to estimate the length of the effective fetch. However, if the lake shore is very irregular in shape, the following procedure should be used to estimate F_e.

   The method suggested by Saville et al. (1962) proposes constructing 15 radials from a given point on the bank at intervals of 6 degrees out to an angle of 45 degrees on either side of the wind direction. These radials are extended until they intersect the shoreline. The component of length of each radial in a direction parallel to the wind direction is computed as shown in figure 19. The effective fetch F_e is computed by the relationship shown in figure 19. For the example in figure 19, F_e equals 15,133 feet. Since Carlyle Lake can be approximated by a rectangle, the computed value of F_e by equation 10 equals 16,400 feet. These two values are very close in magnitude.

6. From the known F_e, compute significant wave height by using equation 12 or figure 13. In equation 12, *U* is given in feet per second, whereas in figure 13, *U is* given in miles per hour.

Table 5. Width-Fetch Ratio of 26 Lakes in Illinois

Lake and County		Surface Area at Normal Pool Level in acres	Storage at Normal Pool Level in Acre -Ft	W/F Ratio for Wind Direction of			
				S or N	SW or NE	W or E	NW or SE
Carlyle Lake	(Clinton)	26,000	233,000	0.60	0.37	1.59	1.59
Lake Shelbyville	(Shelby)	11,100	210,000	0.17	0.14	0.29	0.28
Rend Lake	(Jefferson)	18,900	180,000	0.30	0.37	1.45	1.01
Springfield L.	(Sangamon)	4,234	55,362	0.25	0.22	0.19	0.55
Lake of Egypt	(Williamson)	2,250	41,500	0.13	0.38	0.16	0.17
Crab Orchard L.	(Williamson)	10,000	25,000	0.79	0.44	0.43	0.20
Kincaid Lake	(Jackson)			0.15	0.25	0.16	0.53
Pox Lake	(Lake)	1,550	16,520	0.72	0.33	0.63	1.36
Chautauqua Lake	(Mason)	3,562	l4,248	1.1)3	0.21	0.62	3.57
Lake Decatur	(Macon)	2,805	12,532	0.15	0.07	0.26	0.17
Lake Mattoon	(Coles)	1,210	10,220	0.21	0.17	0.97	0.85
Evergreen L.	(McLean)			0.41	0.50	0.61	0.38
Lake Bloomington	(McLean)	487	7,963	0.29	0.13	0.62	0.30
Devils Kitchen L.	(Williamson)	810	6,1)80	0.24	0.17	0.1)8	0.06
Lew Yaeger L.	(Montgomery)			0.16	0.26	0.90	0.17
Lake Vermilion	(Vermilion)	700	5,343	0.24	0.47	1.17	0.67
Lake Petersburg	(Menard)	191	4,303	0.18	0.1)3	0.19	0.53
Turner Lake	(Putnam)	300	1,800	2.40	0.38	0.23	0.29
Lake Thunderbird	(Putnam)			0.45	0.60	1.80	0.19
Lake Wildwood	(Putnam)			0.25	1.87	0.28	0.14
Meyers Lake	(Tazewell)	76	304	0.19	0.71	2.75	0.83
Maraldo Lake	(Tazewell)			0.60	0.26	0.33	0.19
Negro Lake	(Mason)			0.92	0.24	0.24	0.77
Lake of the Woods	(Champaign)			0.38	1.17	0.24	0.36
Clear Lake	(Sangamon)	45	270	1.33	1.22	0.57	0.25
Monee Lake	(Will)	40	120	0.30	0.67	1.50	0.80



β	cos β	X_i	X_i cos β
42	0.743	2.30	1.71
36	0.809	2.60	2.10
30	0.866	2.85	2.47
24	0.914	3.43	3.14
18	0.951	4.20	3.99
12	0.978	4.00	3.91
6	0.995	3.67	3.65
0	1.000	2.50	2.50
6	0.995	1.78	1.77
12	0.978	1.52	1.49
18	0.951	1.35	1.28
24	0.914	1.00	0.91
30	0.866	0.80	0.69
36	0.809	0.72	0.58
42	0.743	0.70	0.52
Total	13.512	—	30.72

$$F_e = \frac{\Sigma X_i \cos \beta}{\Sigma \cos \beta} = \frac{30.72}{13.512} = 2.27 \text{ units}$$

where 1 unit = 6,666.67 ft

$\therefore F_e = 15{,}133$ ft

Figure 19. Method of computation of effective fetch for irregular shaped lakes

7. Select a bank slope based on geology, bank materials present, and the existing semi-stable or natural slopes in the region.
8. Estimate the stable weight of the median size riprap particle from equation 15 or figure 18. Once the weight of the median size particle is known, the median size of an equivalent sphere can be computed with known or estimated specific gravity of the riprap particles or stones.

   The gradation, maximum and minimum sizes or weights of the riprap particles, and type and size distribution of the bedding or filter materials can be estimated as suggested earlier in "Stabilization of Lakes Shores."
9. Estimate the length of the bank along the slope to be stabilized as follows: the bank must be stabilized from below the expected low water level to the highest water level plus free board (Saville et al., 1962). The freeboard is a combination of wave height, wave runup, and wind tide. If the banks are not protected to the highest expected water level, the wave action may precipitate the bank failure near the top of the bank.

The above procedure is suggested on the basis of the present investigation for protecting embankments against wind-generated waves. Variations and changes can be accommodated if necessary.

In the case of a narrow and heavily forested lake where boat-generated waves can damage the lake shore, the procedure outlined above can be used with some modification. For boat-generated waves, the height of the wave must be estimated. As a guide, figure 15 can be used bearing in mind all the limitations in the development of this figure. With an estimated wave height, steps 7 to 9 can be followed to design the proper protective measures.

Design Example. The design procedure just outlined above is illustrated by a design example. A segment of the bank in Rend Lake is a highway embankment which almost squarely divides the lake in two parts. Illinois Highway 183 crosses the lake in an east-west direction, thus exposing both banks to wave action.

Data on bank slopes and sizes of the existing riprap particles were collected from the highway embankments. The north embankment is exposed to north and north-northwest winds especially in January and February. The computations are:

1. The location is the north embankment of Illinois Highway 183 in Rend Lake between the state park and west bank of the lake.
2. Since Rend Lake is located about half way between Evansville, Indiana, and St. Louis, Missouri, the long term wind data from these two climatological stations will be used.
3. A 6-hour duration with a 50-year return period is selected from table 1, under Evansville, Indiana for the month of February. The best estimate of wind velocity for a 50-year return period is equal to 36.60 mph (58.89 km/hr). Similarly, for St. Louis for the month of February, the wind velocity is equal to 34.45 mph (55.43 km/hr). Therefore, the maximum average wind velocity is equal to 35.53 mph (57.16 km/hr) or 52.11 fps (15.89 m/s).
4. Maximum fetch of the water surface in the north-northwest direction is measured to be equal to 39,600 feet (12,078 m).
5. The effective fetch, F_e, is computed by equation 10 to be equal to 17,158 feet (3.25 miles for figure 13) (5.23 km) for an average width of water surface equal to 9,000 feet (2.76 km).
6. The design wave height is computed by equation 12 to be equal to 2.88 feet (0.878 m).
7. The present bank slope was measured to be approximately equal to 3:1 (18.4 degrees for use with figure 18). Riprap particles were once placed on this embankment, but field inspection during the spring of 1975 indicated that repair work was needed at this location.
8. The median weight of riprap particles, $\overline{W}_{50}$, is estimated to be equal to 21.23 pounds (9.63 kg) by equation 15. The equivalent diameter of a sphere becomes 7.51 inches (190.75 mm). However, from table 4 Sample No. R2, the measured d_{50} size is equal . to 3.66 inches (92.96 mm) which is less than the predicted value of 7.51 inches (190.75 mm).
9. The total length of the embankment that should be protected against wave action is estimated as follows. For Rend Lake, the difference between normal pool level and emergency spillway elevations is 10 feet. Allowing another 5 feet below normal pool level, the total length of the embankment that should be protected is equal to 45 feet on a 3:1 slope.
10. The gradation of the riprap particles is estimated as follows: the maximum size is esti-

mated to be equal to 4 d_{50} or 30 inches (762 mm) and minimum size to be equal to 0.125 d_{50} or 1 inch (25.4 mm). The thickness is estimated to be equal to 12 inches (304.8 mm).

11. With a uniformity coefficient of 2.5 and standard deviation of 1.8, the d_{15} size of the riprap particles is estimated to be equal to 3.54 inches (90 mm).
12. Therefore, based on the above information, the size distribution and thickness of filter materials can be estimated. The ratio of d_{15} of the riprap particles to d_{85} of filter materials should be less than 5. For this example it is assumed to be equal to 4.0, and the d_{85} of filter materials is equal to about 0.88 inch (22.5 mm). With a standard deviation of about 1.6 similar to filter sample C4 from Carlyle Lake (figure 17), the d_{15} size of the filter materials should be about 0.275 inch (7 mm). The thickness of the filter materials is estimated to be 8 inches (203.2 mm). It is generally recommended to use crushed stone for filter materials.

The above example illustrates the principles involved in the design procedure, method of solution, and the use of equations and graphs developed in this report. However, in any field problem, the hydraulic engineer must exercise due judgment supplemented with experience to design proper protective measures. Economically this concerns a staggering amount of money. In one problem area, about five hundred thousand dollars were spent to stabilize one segment of an embankment with riprap particles. Because of improper use of riprap particles, within one year the embankment was damaged by heavy wave action. It was estimated that an additional two hundred thousand dollars had to be spent to repair this damaged embankment. Therefore, considerable savings in money can be achieved by selecting the proper protective measures for an anticipated wave condition.

SUMMARY AND CONCLUSIONS

Waves produced by winds blowing over the water surface are called 'wind waves.' They may vary from ripples to some larger size in lakes and to giant waves on the ocean. Wind waves are defined by their height, length, frequency of occurrence, and energy spectrum. Wind wave characteristics are determined by wind speed, direction, duration, and the length and width of the water surface over which the wind blows. The wind waves in inland waterways such as rivers, canals, or lakes can and will cause substantial erosional damage if the waterway shore does not have natural or artificial protection. Extensive lake shore erosion caused by wind-generated waves is present in Illinois. The orientation of the main watersheds and the general wind movement during part of the year in Illinois are conducive to the generation of damaging waves in reservoirs.

The present research investigation was undertaken to develop design criteria to stabilize lake shores with stones or riprap particles against an anticipated wave condition. Field data on wind-generated waves, wind velocity, size distribution of existing riprap particles, and other pertinent information were collected from Carlyle and Rend Lakes in Illinois. The wind-generated wave data were collected by using a super-8 mm movie camera. The techniques used for wave data collection and reduction were simple, very inexpensive, and possibly innovative. They can also be used for other similar conditions.

Long term wind data (1950-1972) from five climatological stations in and around Illinois were analyzed to predict the historical wind velocity for any specified duration, direction, and return period. These historical wind data fitted the log-normal probability distribution with two parameters. The goodness of fit was tested by the χ^2 test which indicated that the departures of the wind speed from the theoretical distribution are not significant. A table is presented showing the maximum wind speed for three different durations corresponding to various return periods for all five climatological stations.

Analysis of wind data from Carlyle Lake for the months of May and June, 1974, indicated that wind generally blew from the south with a daily average velocity of about 10 mph (16.0 km/hr).

Wind-generated wave data were digitized and statistical analyses were performed. Wind-generated waves are positively correlated for lag of less than one-half second. The fundamental periods were found to be a function of wind velocity and fetch. Analysis of energy spectra indicated an increase in peak energy with an increase in wind velocity. The peak energy occurred at a lower frequency for higher wind velocity compared to the occurrence of peak energy for lower wind velocity. Nondimensional plot of energy spectra versus frequency indicated some similar characteristics to the deep water waves in the equilibrium range of frequencies. Nondimensional energy spectra followed the $(f/f_m)^5$ rule in this range of equilibrium frequencies.

Frequency analysis of wave heights indicated that the Rayleigh distribution fitted these data and the goodness of fit was tested by the χ^2 test.

A relationship was developed to estimate the significant wave heights in lakes for a known wind speed and direction by modifying the relationship proposed by Sverdrup-Munk Bretschneider. The effect of shallow depth as proposed by Sibul (1955) and the effect of limited width as suggested by Saville (1954) have been incorporated in this relationship. A nomograph is presented for easy estimation of wave heights.

A method is also presented for estimating wind tide in a lake.

Analysis of boat-generated wave heights indicated that these heights can be expressed to some extent by Rayleigh distribution. Serial correlation analysis indicated no periodic variation in boat-generated waves. Variance spectra analysis indicated that as the distance between the moving boat and the shore becomes very close, the exposed shore must dissipate a major portion of the wave energy in a very short period of time. A relationship is presented relating maximum wave height, boat speed, distance between the boat and the wave gage, and the draft of the boat.

Data on size distribution of existing riprap particles were collected by measuring the three axes of each stone. The d_{50} sizes of riprap particles varied from 3 to 11 inches (76.2 to 279.4 mm) with the maximum size up to 40 inches (1.016 m). All riprap particles are angular in shape and the shape factor varied from 0.5 to 0.54 with an average standard deviation of 0.17.

From a consideration of the basic relationships for drag force, lift force, submerged weight of riprap particles, and physical properties of riprap and water, a relationship was developed to estimate the stable weight of riprap particles. The unknown coefficient in the equation was computed from available laboratory and field data. A nomograph is presented for ease in estimating the median weight of riprap particles. Recommendations as to the selection of gradation, maximum and minimum sizes of riprap particles, thickness of riprap particles, and size distribution, gradation, and thickness of filter materials are also presented.

Finally, a design procedure is suggested for stabilizing lake shores with riprap particles against an anticipated wave condition. This procedure incorporated all the relevant plots, nomographs, etc., developed in this report. One specific design problem was solved and presented in this report showing in detail all the steps involved in an actual design problem.

The method, procedure, relationships and figures developed in this investigation are mainly to be utilized for stabilizing lake shores with riprap particles. However, if other methods of stabilization of lake shores appears to be economical and/or advantageous, the. results of this investigation can still be useful in estimating the design wave heights.

During the course of this investigation it was observed that a considerable amount of money is being spent to maintain and/or stabilize lake shores after the reservoir has been built and the lake shore has started to erode by wind-generated waves. Engineers, planners, and those in charge of designing, developing or constructing any dam or reservoir might consider the fact that the stabilization of lake shores against wind waves should hot be considered as a maintenance item; rather, it should be included in the original cost of the project. The areas and shores of a proposed reservoir that might experience heavy wind wave action can be delineated from the wind data analysis already done for the state of Illinois (table 1). However, for any other locations in the country, a similar analysis of long term wind data can be performed, and on the basis of analyzed wind data, the location of potential erosion prone areas in any proposed reservoir can be delineated.

REFERENCES

Blackman, R. B., and J. W. Tukey. 1958. *The measurement of power spectra.* Dover Publications, Inc., New York.

Braslavskii, A. P., and Z. A. Vikulina. 1963. *Evaporation norms from water reservoirs.* Translated from Russian and published for the U.S. Department of the Interior and the National Science Foundation by the Israel Program for Scientific Translations.

Bretschneider, C. L. 1959. *Wave variability and wave spectra for wind-generated gravity waves.* U.S. Army Corps of Engineers Beach Erosion Board Technical Memorandum Number 118.

Carlson, E. J., and W. W. Sayre. 1961. *Canal bank erosion due to wind-generated water waves.* U.S. Bureau of Reclamation Hydraulic Laboratory Report No. Hyd-465, Progress Report 1.

Cermak, J. E., and H. J. Koloseus. 1953. *Lake Hefner model studies of wind structure and evaporation.* Final Report No. 54 JEC 20, Colorado Agricultural and Mechanical College, Fort Collins.

Colonell, J.M., and B. Perry. 1968. *Laboratory simulation of sea waves.* ASCE Journal of the Waterways and Harbors Division, V. 94(WW2): 159-174.

Das, M. M. 1969. *Relative effect of waves generated by large ships and small boats in restricted waterways.* Hydraulic Engineering Report No. HEL-12-9, University of California, Berkeley.

Estuary and coastline hydrodynamics. Edited by A. T. Ippen. Engineering Societies Monographs, McGraw Hill Book Co., New York, pp. 49-54, 150-154.

Goodknight, R. C, and T. L. Russell. 1963. *Investigation of the statistics of wave heights.* ASCE Journal of the Waterways and Harbors Division, V. 89(WW2):29-52.

Hedar, P. A. 1953. *Design of rock-fill breakwaters.* Proceedings, Minnesota International Hydraulic Convention, Minneapolis, September 1-4.

Hidy, G. M., and E. J. Plate. 1965. *Frequency spectrum of wind-generated waves.* The Physics of Fluids, V. 8(7):1387-1389.

Lamb, H. 1932. *Hydrodynamics.* Dover Publications, New York, 6th edition, pp. 623-624.

Lineback, J. A. 1975. *Geology of the Carlyle Reservoir.* Special Note. Illinois State Geological Survey, Urbana.

Liu, P. C. 1968. *Spectral analysis of shallow water waves in Lake Michigan.* Proceedings, Eleventh Conference Great Lakes Research, Milwaukee, Wisconsin, April. pp. 412-423.

Lonquet-Higgins, M. S. 1952. *On the statistical distribution of the heights of sea waves.* Journal Marine Research, V. 11(3): 345-366.

Nichols, W. S. 1975. *Notice to navigation interests.* U.S. Army Corps of Engineers, Ohio River Division, Cincinnati.

Panofsky, H. A., and G. W. Brier. 1958. *Some application of statistics to meteorology.* Pennsylvania State University, University Park. pp. 126-161.

Phillips, O. M. 1958. *The equilibrium range in the spectrum of wind-generated waves.* Journal of Fluid Mechanics, V. 4(5):426-434.

Saville, T., Jr. 1954. *The effect of fetch width on wave generation.* U.S. Army Corps of Engineers Beach Erosion Board, Technical Memo No. 7., 9 p.

Saville, T., Jr., E. W. McClendon, and A. L. Cochran. 1962. *Freeboard allowances for waves in inland reservoirs.* ASCE Journal of the Waterways and Harbors Division, V. 88(WW2): 93-124.

Saville, T., Jr. 1967. *Rock movement in large-scale tests of riprap stability under wave action.* U.S. Army Coastal Engineering Research Center, Reprint R. 3-67.

Sibul, O. 1955. *Laboratory study of the generation of wind waves in shallow water.* U.S. Army Corps of Engineers Beach Erosion Board, Technical Memo No. 72, 35 p.

Sorenson, R. M. 1973. *Ship-generated waves. In* Advances in Hydroscience edited by V. T. Chow. Academic Press, New York, Volume 9, pp. 49-83.

Subcommittee on Slope Protection of the Committee on Earth Dams. 1948. *Review of slope protection methods.* ASCE Soil Mechanics and Foundations Division, V. 74:845-866.

Titov, L. F. 1971. *Wind-driven waves.* Translated from Russian and published for the U.S. Department of Commerce and the National Science Foundation by the Israel Program for Scientific Translations.

U.S. Army Coastal Engineering Research Center. 1966. *Shore protection, planning and design.* Technical Report No. 4, 3rd edition.

U.S. Army Corps of Engineers. 1971. *Earth and rock-fill dams: General design and construction considerations.* Engineering Manual, EM 1110-2-2300, Washington, D.C.

U.S. Bureau of Reclamation. 1973. *Design of small dams.* 2nd edition, Washington, D.C.

Wolman,M. G. 1954. *A method of sampling coarse river-bed material.* Transactions, American Geophysical Union, V. 35(6):951-956.

Yevjevich,V. 1972. *Stochastic processes in hydrology.* Water Resources Publications, Fort Collins, Colorado.

NOTATIONS

The following symbols are used in this report:

A_n = coefficient
a = longest axis of a riprap particle
B_n = coefficient
b = intermediate axis of a riprap particle
C_n = Fourier coefficient
Cov = covariance of a time series
C_v - coefficient of variation
c = shortest axis of a riprap particle
D = average depth of water in a lake
d_s = draft of the boat
d_{50} = sieve diameter when 50 percent of the riprap particles are finer than this diameter
E = wave energy per unit of surface area
E_n = boat-generated wave energy per unit of surface area
F = fetch
F_B = bouyant weight of a riprap particle
F_e = effective fetch
F_D = drag force
F_L = lift force
f = frequency
f_m = frequency at which $\Phi(f)$ is maximum
g = acceleration due to gravity
H = individual wave height
$\bar{H}$ = average wave height
H_m = maximum wave height generated by a moving boat
H_s = significant wave height
$h(t)$ = continuous time series
K, $\bar{K}$ = coefficients
KE = kinetic energy
L = length of the boat
p = probability
PE = potential energy
m = rank of wind speed
N = total number of wind speed observations for historical wind data
n = discrete number of values in the time series realization
$R(\tau)$ = autocorrelation coefficient
S = wind tide
Sp = shape factor
S_s = specific gravity of riprap particles
T = length of record
t = time
U = wind velocity

U_e = effective wind velocity
V = boat speed
W = width of water surface
$\bar{W}$ = weight of stable riprap particle
X = distance between boat and wave gage
Δt = interval of time
a = lake shore slope with horizontal
Υ = unit weight of water
$\eta = H/\bar{H}$ = nondimensional wave height
σ = standard deviation
σ^2 = variance
τ = lag time
Φ = angle between the wind direction and water surface bearing
$\Phi(f)$ = spectral density
$\bar{\Phi}$ = submerged angle of repose of the riprap particles