MICHIGAN SEA GRANT PROGRAM

THE MICHIGAN SHORE PROTECTION DEMONSTRATION PROJECT FINAL REPORT

Wave Histories by Means of a Computer Program

Some Conclusions About Low-Cost Protection

by
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Wave Histories by Means of a Computer Program

Some Conclusions about Low-Cost Protection

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1.0 INTRODUCTION

This project used field and laboratory investigations to evaluate low-cost shore protection methods. The project was initially financed by a grant of $300,000 to the Michigan Department of Natural Resources (DNR). The University of Michigan Coastal Zone Laboratory administered the project. The DNR provided major assistance and cooperation in selecting the sites and constructing the installations. The installations were designed by the project director. The eighteen field demonstration projects are listed in Table 1, and their locations are shown in Figure 1. The laboratory program was also the responsibility of the project director. The early observation program was financed by the original grant. Thereafter field observations were made with funds provided by the Michigan Sea Grant Program, the DNR, the U.S. Army Corps of Engineers and a small grant from a Rockefeller Foundation project centered at the University of Wisconsin. The results of the projects have been reported in a series of project reports (1,2,3,4), technical papers (5,6,7), and other publications planned specifically to provide "how to do it" information (8,9) for private property owners and public agencies who are faced with shore erosion problems. In other efforts to make the information available, workshops were presented throughout the state and an audio-visual presentation was made available.

The primary effort during the final project year was to develop a history of wave action at the demonstration sites as a basis for evaluating the effectiveness and durability of the field installations. Because of the large number of wind storms requiring analysis for the various sites, we developed a computer algorithm for estimating wave heights from continuous hourly wind velocities and directions. This computerized procedure was not only beneficial to this project but may be of value to others who must hindcast wave heights from wind data. The computer approach is presented in Section 2, and the results of its application to the various demonstration sites are shown in Section 3. Section 4 provides a series of conclusions regarding low-cost shore protection based on observations at the eighteen demonstration project sites.

2.0 DEVELOPMENT OF A COMPUTER PROGRAM TO DETERMINE WAVE CHARACTERISTICS FROM WIND

In accordance with the need to develop a method that would produce an expedient and reasonably accurate estimate of the wave height distribution during major storms at the demonstration sites, a computer solution of the

TABLE 1

MICHIGAN SHORE PROTECTION DEMONSTRATION PROJECTS

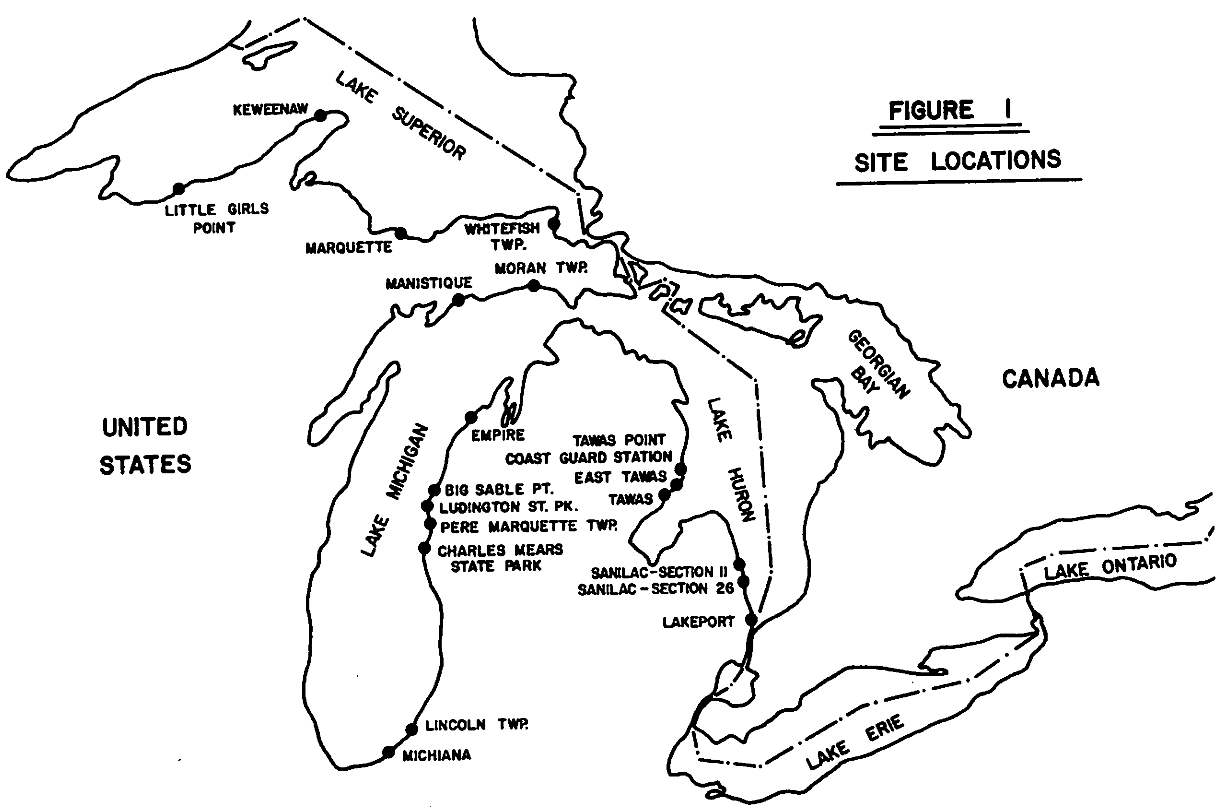
Project	Type of Protection
LAKE MICHIGAN	
Michiana	Toe protection, asphalt mastic and rock revetment, 300 ft long
Lincoln Township	Wooden groin, Longard tube groin
Mears State Park	Three groins, inner ends are gabions and outer ends are sand-filled bags
Pere Marquette Township	Off-shore breakwater, three segments of zig-zag concrete walls with 50 ft gaps
Ludington State Park	Two groins, steel
Big Sable Point	Steel wall, gabion cutoff groins
Empire	Toe protection, 40-inch Longard tube, 300 feet long
Moran Township	Toe protection, three 40-inch Longard tubes stacked one on two (300 ft), three layers of sand-filled bags (250 ft)
LAKE SUPERIOR	
Whitefish Township	Rock revetment with wooden groins extending to low bluff
Marquette	Sand nourishment, waste sand from local industry
Keweenaw	Revetment, waste rock from local mines
Little Girls Point	Revetment, Nami rings

Table 1 (Continued)

Project	Type of Protection
LAKE HURON	
Tawas Point Coast Guard Station	Revetment, part dumped rock and part placed in two layers
East Tawas	3000 yds^3 sand placed along 400 ft of open shoreline
Tawas City	4350 yds^3 sand placed between new wooden groin and existing pier along 400 ft of shoreline
Sanilac-Sect. 11	Toe protection, 69-inch Longard tube 400 ft long
Sanilac-Sect. 26	Six groins, two 40-inch Longard tubes, one 60-inch Longard tube, gabions, sand-filled bags, rock and asphalt mastic, timber crib
Lakeport	Off-shore barrier, 40-inch Longard tube placed on off-shore bar

FIGURE I

SITE LOCATIONS



revised Sverdrup-Munk-Bretschneider (SMB) equations was developed. The SMB method[10] is based on field and laboratory observations. The more sophisticated methods based on the solution of the equations developed by Miles and Phillips[11,12,13] were considered but not used, in part because of the large investment of time and computational resources that the method demands. Another important factor that made the use of the SMB method the only logical choice is the lack of detailed wind velocity data. The wind data available were satisfactory for the data requirements of the SMB method.

2.1 The SMB Equations

The revised versions of the SMB equations (10) in deep water in dimensionless form are given by:

$$\frac{g_c H_o}{U^2} = E \tanh \left[G(g_c F/U^2)^{.42}\right] \tag{1.a}$$

$$\frac{g_c T}{U} = M \tanh \left[N(g_c F/U^2)^{.25}\right] \tag{1.b}$$

$$\frac{g_c D_u}{U} = K\exp \quad \left[A(\mathrm{Ln}(g_c F/U^2))^2 - B\mathrm{Ln}(g_c F/U^2)+C\right]^{\frac{1}{2}} + D\mathrm{Ln}\ (g_c F/U^2) \tag{1.c}$$

where H_o is the deep water significant wave height in ft., g_c is the acceleration of gravity in naut. mi/hr^2, U is the wind velocity in knots, F is the wind fetch in nautical miles, T is the wave period in seconds, and D_u is the wind duration in hours. The values of the constants are:

A =	.0161	G =	.0125
B =	.3692	M =	27143.36
C =	2.2094	N =	.077
D =	.8798	K =	6.5882
E =	1720.77	g_c =	2131.42g

These equations have been used in their graphical form (13). The graphical procedure is convenient to determine the probable maximum wave height for average winds of various durations during a storm. To determine a more complete history of wave heights with periods of growth and decline, a method using graphs was developed.[10] However, when hindcasting is necessary for a large number of storms, as for this erosion control project, the graphical method becomes time-consuming. For this reason, we developed a computer algorithm for solving for wave height and wave period from a continuous wind velocity record.

Wind observations for intervals of one hour were available. The duration and velocity of the wind are taken as the controlling parameters but the corresponding fetch was checked at each interval to be sure that wave growth was not limited by the fetch. In addition, the growth is limited by the difference between the wave celerity and the wind velocity. When the wave celerity is larger than the surface wind velocity, no more energy is transferred from the wind to the water, and although the wind can be thought of actually slowing down the waves, any damping was neglected. Whenever this occurred, the wave characteristics equivalent to those one hour of wave travel upwind from the site were considered to prevail.

2.2 Numerical Solution

The data available consisted of hourly records of wind velocity and direction of the storm systems from Oct. 1974 to Nov. 1978 in the Great Lakes. A more detailed description of the compilation and analysis of the data is given in a later section. The solution consists of two parts. The first portion deals with estimating wave heights and periods in deep waters. The second portion deals with the changes in the wave characteristics as they move into the shallow water and reach the breaker locations at the various sites.

2.2.1 Deep Water Wave Characteristics. The numerical solution of equation (1) commenced by computing the value of the parameter $z = g_c F/U^2$ from equation 1c, knowing that $D_u = 1$ and the wind velocity U is given by the one recorded during the first time interval. Equation 2c can be arranged into the following form:

$$f_{Du}(z) = 1 - KU/g_c \exp(A(\ell nZ)^2 - B\ell n\ Z + C)^{\frac{1}{2}} + D\ell n\ Z = 0 \qquad (2)$$

The value of Z in equation 2 cannot be computed explicitly; therefore, a Newton-Raphson technique was used to compute the Eigen values. The derivative of this function with respect to Z, for constant U, becomes

$$f'_{Du} = - f_{D}\frac{1}{z} \{\frac{1}{2}[\Delta(\ell nz)^2 - B\ell n\ Z + C]^{-\frac{1}{2}}\ [2\Delta\ell n\ Z - B] + D\} \qquad (3)$$

The correct value of Z is obtained by iterating the expression

$$Z_i = Z_{i-1} - \Delta z \qquad (4)$$

in which

$$\Delta Z = Z\{\frac{1}{2}[\Delta(\ell n Z)^2 - B\ell n\ Z + C] \quad [2\Delta\ell n \quad Z - B] + D^{-1}\} \tag{4.a}$$

The value of Z so obtained is used in equations 1.a and 1.b to obtain the value of H_o and T for the first hour. For subsequent hours the procedure differs somewhat because the duration is no longer equal to one hour. If the wind is constant in intensity and direction, the duration (D_u) corresponding to subsequent intervals is obtained by increasing the duration of a previous step by an amount equal to the time intervals (ΔD_u), so that for every step:

$$D_{ui} = D_{ui-1} + \Delta D_u \tag{5}$$

Because the recorded wind data show velocity, and direction changes from hour to hour, it is necessary that an equivalent duration be computed and increased by the time interval (ΔD_u) to recompute the value of Z from equation 1.c and then to compute H_c and T from equations 1.a and 1.b respectively. The equivalent duration is defined as the number of hours that a wind blowing at a velocity U_2 would take to build up the same amount of wave energy present in the water at the end of the previous interval. In using wind data it was assumed that the velocity and direction remained constant from the middle of the interval to the middle of the next interval. At that time the conditions were assumed to change instantaneously to those for the next hour and the equivalent duration was determined by assuming that the wave energy was the same at the beginning of each hour as at end of the previous hour.

Since wave energy for unit width in deep water is given by

$$E_o = \frac{\gamma}{8} H_o^2 L_o$$

then

$$E_o = \frac{\gamma}{8} H_o^2 \frac{gT^2}{2\pi}$$

or

$$E_o = \alpha(H_o^2 T^2) \tag{6}$$

Expression 6 indicates that energy is proportional to the second power of the product of wave height and wave period. The proportionality constant, α, is assumed to be constant so that the value of $H_o^2T^2$ at the start of a new duration interval can be computed from the values of wave height and period at the end of the previous interval.

Thus, from equations 1.a and 1.b:

$$H_o = f_1(U^2, z)$$

and

$$T = f_2(U,z)$$

and

$$E_o = f_1(U^2,z) \quad f_2(U,z)^2 \qquad (7)$$

The value of the parameter Z must be computed from equation 7, where T and U are known and constant. This again, can be performed readily by a Newton-Raphson iterative procedure. Where:

$$E_o' = -2f_1 f_2 (f_1 f_2' + f_2 f_1') \qquad (8.a)$$

in which

$$f_1' = 722.72 \frac{U^2}{g_c} \frac{G}{Z} .58 \operatorname{sech}^2(Gz^{.42}) \qquad (8.b)$$

$$f' = 6785.84 \frac{UN}{g_c Z} .75 \operatorname{sech}^2(NZ^{.25}) \qquad (8.c)$$

Once the value of Z is obtained, it can be substituted in equation 1.c to obtain the equivalent duration D_u. This D_u is used in conjunction with equations 1.a, 1.b and 1.c to compute the wave characteristics.

It should be noted that there are cases where the procedure needs some intermediate considerations, some of which are:

a) $U < C$, that is when the wind velocity drops below the wave celerity or the wave period increases enough to cause the celerity to exceed the wind speed. In either case the wind is incapable of transmitting energy to the existing waves and these waves will move faster than the wind leaving the fetch area. Conditions at the site are then taken as those that existed one hour upwind one hour earlier.

b) $\Delta\delta > 25°$; where δ is the wind direction. When the value of δ changes more than 25° from one time step to the other, it is assumed that the effect of the pre-existing wave forms on the transfer of wind energy to the water, can be neglected. This criterion is derived from observations reported elsewhere (4) that wind energy is transmitted to water waves in a conical pattern with an approximate vertex angle of 45-50°.

c) $\Delta\delta < 25°$ when the change in δ is less than 25° the equivalent duration is computed from a corrected value of the wave energy given by:

$$E_{oc} = E_o \cdot \cos\Delta\delta$$

d) When a shift in wind direction produces a fetch (F) equal to zero (i.e. shift to offshore wind), during one or more time steps, the last fetch is corrected by a distance equal to the celerity times the time interval. The value of z is recomputed and used to determine H_o and T. This procedure is repeated for as long as the fetch remains zero.

2.2.2 Shallow Water Wave Characteristics. The height of the breaking waves in the surf zone is computed by means of a single step procedure based on the deep water wave height, the wave period and the angle between the bottom contours and the wave fronts.

As the wave train travels into shallow water the wave height at any depth is given by

$$H = K_r K_s H_o \tag{9.a}$$

where the refraction coefficient K_r, is given by the expression

$$K_r = \frac{\text{Cos } \alpha_0}{[\cos \sin^{-1}(\tanh Kd. \sin\alpha_0)]} \tag{9.b}$$

in which α_0 is the angle between the bottom contours and the wave fronts in deep water and $K = 2\pi/L$. The value of the shoaling coefficient K_s is computed by the expression

$$K_s = \tanh Kd \quad \left(1 + \frac{2Kd}{\sinh 2Kd}\right)^{-\frac{1}{2}} \tag{9.c}$$

The depth at which breaking occurs, d_b, and the height of the breaker H_b, are related according to the expression.

$$d_b = K_b H_b \tag{10}$$

Numerous laboratory and prototype studies have produced a variety of values and methods of obtaining K_b, but in general it is well accepted that for most practical problems K_b can be taken as 1.28.

Equation 9.c can be simplified and combined with equations 9.a, 9.b, and 10 to produce the breaking depth function given by

$$f_{Br} = d_b - 1.28 \sqrt{2}\, H_o \cosh Kd_b \cdot \frac{\cos\alpha}{A} \tag{11}$$

where

$$A = \cos\ [(\sin^{-1}\tanh Kd_b . \sin\alpha_0)] \quad . \quad \sin h\ 2Kd_b + 2Kd_b$$

In equation 11 the only unknown is the depth of the breaker d_b since K' is given by:

$$K' = \frac{4\pi}{gT^2} \cot Kd_b \tag{12}$$

and the values of T and H_o are known.

The breaking depth function (equation 11) was solved with a Newton-Raphson iterative procedure. It was decided to compute the value of the derivative of the breaking depth function numerically so that

$$f'_{Br} = [f_{Br}(d_b + \varepsilon) - f_{Br}(d_b)] / \varepsilon \tag{13}$$

where ε is a small perturbation. This method proved to be very efficient with solutions being produced with only two iterations on the average. Once the depth of the breaker is computed, the breaker height can be readily determined from equation 10.

2.3 Description of Program

The numerical solution described was implemented by means of a computer algorithm named MESH (Method for Estimating Significant Heights) using the Fortran IV language. MESH can be handled with any of the fast speed computers available today and the cost of processing storm data is obviously directly related to the storm length. For a single 48 hour storm, the cost of processing on the Amdahl machine available at the University of Michigan is 15¢. When several storms are processed at once, the cost can be reduced to less than half. The program also contains an option to compute and print the number of hours that the significant wave height exceeded various specified heights. Omittance of this feature can reduce processing costs. In addition, if the breaker height is not needed, and only deep water conditions are of interest, the cost given above can be reduced even further.

Another way of reducing processing costs is to manipulate the tolerance allowed for the numerical solutions and the maximum number of iterations specified. This feature will be presented in more detail in section 2.4.

2.4 Program Capabilities

The program can process storms as long as 100 hours. Most storms recorded so far fall short of this limit. When the wind shifts to a direction for which the fetch is zero, the pre-existing wave spectra are assumed to continue to travel to the downwind end of the fetch. An important feature of this program is that it allows the user to specify up to five different heights each one of which will be compared against the resulting wave height at every hour. After the hourly distribution of wave heights and periods is printed, the number of hours during which the waves were larger than each one of the specified heights is also printed in tabular form. This feature can be helpful in reducing the time spent compiling statistical data for a frequency and duration analysis. For user convenience the program also has the capability to process storms that last from one month to the next taking into consideration whether the number of days in the month is even or odd.

2.5 Input

Input to the program can be separated into two sets of records. The first set, (A), only needs to be specified once and contains the curve relating fetch to wind duration, tolerance requirements, number of iterations, processing options, shoreline azimuth, wave statistics requests and site name and site location. The second set of records, (B), contains the wind information on all storms to be processed. The number of storms that can be processed in one run is unlimited but each storm is processed individually.

Variables To Be Specified For Set A

IFET (I)	:	array of fetches in azimuth increments of 10
TOLR	:	tolerance
NIT	:	maximum number of iterations
IFRAN	:	used to select options (explained later)
SHAZ	:	azimuth of normal to shoreline
NHFR	:	array of heights for wave statistics (max = 5)
HFR (I)	:	array of heights for wave statistics (max = 5)
SITE (I)	:	site name and location

All the variable names defined above with the exception of NHFR and HFR, are specified in the first record through a free format statement under the generic name of FETS. The array (IFET 1) includes the values of all the fetches in nautical miles, starting at an azimuth of zero with increments of 10 up to 360. Care must be exercised so that the array contains a zero for all those directions along which there is no over-lake fetch. The values of TOLR and SHAZ must be specified in real form (F) and NIT and IFRAN in integer for (I). IFRAN only controls the computation and output of statistical information, and must be made equal to one if such computations are desired. Any other value will skip this feature. For practical reasons zero should be used. If IFRAN is made equal to one, the next record (A2) must contain NHFR and corresponding values of HFR. These use a fixed format. If IFRAN is different than one, record A2 should be left blank but not omitted. Record A3 is an alphabetic mode (A) and contains information about the site being investigated. This information will be printed as a heading at the start of each storm. It must be contained within a length of 36 characters left-justified.

The second set of records, (B), contains the information about all storms to be analyzed. Each storm must follow the same format and must have data for the following variables.

Variables To Be Specified For Set B

ID1	:	day the storm started (I2)
ID3		day the storm finished (I2)
MT	:	month the storm occured (A4)
NDMT	:	= 0 if month has 30 days (I1)
		= 1 if month has 31 days (I1)
IYR	:	year of event (I4)
IHRO	:	hour of the day the storm started (0-23); (I5)
IHRF	:	hour that the storm ended (I5)
SITE (I)	:	any relevant note regarding the storm at most 44 characters (11A)
TOL	:	tolerance for a particular storm. If not specified it defaults to TOLR (F7.5)
KIT	:	maximum number of iterations for a particular storm. If not specified it defaults to NIT (14).

Record B2 can be as long as 8 cards. It supplies the wind velocity, in knots, and azimuth at every hour. The array IDIR stores all the values of the directions and the array IVEL contains all the values of the wind velocities. The input is formatted in groups of 13 hours starting with the first hour of recorded data (IHRO). It is very common that the last group will have less than 13 hours of data, in which case the rest of the record is left blank. When more than one storm needs to be processed, only the second set of records (set B2) need be repeated.

2.6 Output

A typical output printout is shown in Fig. 2. The printout consists of three parts. The first part at the top of Fig. 2 is general information ending with the line which gives the units for the storm data. The second part, located in the middle of the page, is the tabulation of the storm data. Part three, at the bottom of the page, is a summary of the duration of waves of various heights.

The following storm data are tabulated in the middle portion of the printout.

Time in hours

Directions, Azimuths in degrees

Fetch in nautical miles

Wind velocity in knots

HO: Deep water wave height in feet

HB: Breaker height in feet

TO: Wave period in seconds

The storm data and the resulting wave characteristics are tabulated for every hour for 24-hour periods. The hour of the day is given by the first line of data labeled "TIME" on the leftmost column. In Fig. 2 the storm started at noon (1200 hrs) on January 1, 1978 and ended at 6:00 p.m. (1800 hrs.) on January 3. The rows labeled "TOL CHECK" are used to warn the user whenever the maximum number of iterations was reached without satisfying the tolerance requirements. If *2 appeared in this line it would indicate that the problem occurred during the computation of a new value of the parameter Z from a value of the equivalent duration greater than zero. An asterisk followed by a one (*1) would indicate that the problem occurred while computing Z from a value of the equivalent duration equal to zero.

Finally, at the bottom of the printout, statistics are printed giving the durations in hours during which the breaking waves were larger than the heights specified in record A2. In the example shown in Fig. 2 the breaking waves were larger than 4 ft. on four occasions lasting 10, 9, 1 and 2 hours respectively. Also the breaker was larger than 6 ft. during one period of 7 hours. Note that the most severe wave action occurred on the first day (January 1) at 10:00 p.m. (2200 hours) when the breaker reached 7.7 ft. with a period of 6.1 secs.

3.0 WAVE CONDITIONS AT THE DEMONSTRATION SITES

The principal purpose of the computer program described in the previous section was to determine the wave conditions at the 18 demonstration sites which have been constructed in this project. This section provides the results

Fig. 2 Sample Output

```
             SITE :  MICHIANA & STEVENSVILLE
       STORM DATE :  1- 3   JAN 1978
        TOLERANCE : 0.1000E 00
       ITERATIONS :    20
   NORMAL AZIMUTH : 310.
```

TIME(~~SECS~~ hrs.);DIREC.(DEG.AZ.);FETCH(NT.ML.);VELOC.(KNTS.);HO(FT.);HB(FT.);TO(SECS.)

TIME	0	100	200	300	400	500	600	700	800	900	1000	1100	1200	1300	1400	1500	1600	1700	1800	1900	2000	2100	2200	2300
DIREC	0	0	0	0	0	0	0	0	0	0	0	0	360	360	340	340	340	340	320	320	300	300	310	310
FETCH	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	213.	213.	128.	128.	128.	128.	72.	72.	56.	56.	62.	62.
VELOC	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	17.	16.	23.	26.	19.	26.	26.	26.	26.	26.	26.	23.
HO	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.4	1.9	2.9	4.0	4.3	5.0	5.7	6.3	6.7	7.2	7.6	6.6
HB	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.2	1.7	2.8	3.9	4.1	4.9	5.7	6.3	6.7	7.3	7.7	6.7
TO	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.5	3.0	3.7	4.3	4.6	4.9	5.2	5.5	5.7	5.9	6.1	5.7
TOL. CHECK																								

TIME	0	100	200	300	400	500	600	700	800	900	1000	1100	1200	1300	1400	1500	1600	1700	1800	1900	2000	2100	2200	2300
DIREC	300	300	270	260	260	250	240	240	240	190	220	240	270	270	270	260	270	270	260	270	270	280	280	270
FETCH	56.	56.	37.	33.	33.	31.	25.	25.	25.	0.	0.	25.	37.	37.	37.	33.	37.	37.	33.	37.	37.	41.	41.	37.
VELOC	23.	23.	23.	26.	24.	26.	24.	20.	26.	20.	20.	14.	26.	24.	24.	23.	20.	20.	23.	20.	21.	20.	21.	20.
HO	6.4	6.4	2.1	3.4	4.1	4.9	4.9	3.9	4.7	3.8	2.6	2.4	2.5	3.4	4.2	4.7	4.5	4.5	5.0	4.5	4.9	4.6	5.0	4.5
HB	6.4	6.4	2.0	2.9	3.6	3.8	3.3	2.6	3.2	2.6	1.8	1.6	2.3	3.2	3.9	4.1	4.2	4.2	4.3	4.2	4.5	4.5	4.8	4.2
TO	5.6	5.6	3.1	3.9	4.4	4.8	4.9	4.3	4.7	4.2	3.4	3.4	3.3	4.0	4.4	4.7	4.7	4.7	4.9	4.7	4.8	4.7	4.9	4.7
TOL. CHECK																								

TIME	0	100	200	300	400	500	600	700	800	900	1000	1100	1200	1300	1400	1500	1600	1700	1800	1900	2000	2100	2200	2300
DIREC	270	270	270	270	270	270	270	300	300	300	290	300	280	280	280	240	260	230	180	0	0	0	0	0
FETCH	37.	37.	37.	37.	37.	37.	37.	56.	56.	56.	49.	56.	41.	41.	41.	25.	33.	0.	0.	0.	0.	0.	0.	0.
VELOC	19.	19.	19.	16.	19.	19.	19.	14.	20.	20.	24.	24.	17.	20.	20.	16.	14.	14.	14.	0.	0.	0.	0.	0.
HO	4.2	4.2	4.2	3.5	3.8	4.1	4.2	0.0	1.7	2.5	3.4	4.1	3.9	4.2	4.5	1.3	1.6	1.2	0.4	0.0	0.0	0.0	0.0	0.0
HB	3.9	3.9	3.9	3.2	3.6	3.8	3.9	0.0	1.7	2.5	3.4	4.2	3.7	4.1	4.4	0.9	1.4	1.1	0.3	0.0	0.0	0.0	0.0	0.0
TO	4.5	4.5	4.5	4.1	4.3	4.5	4.5	0.0	2.8	3.4	4.0	4.4	4.3	4.5	4.7	2.4	2.9	2.4	1.3	0.0	0.0	0.0	0.0	0.0
TOL. CHECK																								

DURATION PERIODS IN HRS. DURING WHICH BREAKING
WAVES WERE LARGER THAN....

H(FT.)	DURATIONS(HRS.)
4.0	10, 9, 1, 2, 0,
6.0	7, 0,
8.0	0,
10.0	0,
12.0	0,

for the four year period which began with October 1974 and continued through November 1978. The summer months, June, July and August, are not included because wind data are not available. Fortunately, major storms occur less frequently during these months than during other months. The projects have been fully described in earlier reports (1,2,3,4). Wave histories for the period 1947-1976 were prepared previously (4) but the early storms were re-analyzed to take advantage of the greater accuracy that could be achieved with the computer analysis. The wind data were compiled from hourly reports from 13 weather stations. The data were adjusted according to the overall weather maps for that day and modified by a land-to-lake correction for the wind velocity. The records do not accurately define the average velocity over the entire fetch. However, we believe the estimated wave heights give a good picture of the wave history at the various sites.

The results of hindcasting analysis were summarized and are presented in Table 2. Shown in the table are the total number of hours of reported storm action, the number of storms available for each site, and the number of hours during which the height of the breaking waves was larger than 4, 6, 8, 10, and 12 ft. respectively.

Note that the Pere Marquette site on the Michigan west coast suffered the heaviest wave action. This can be expected since this site is exposed to long southwest and northwest fetches, and lacks any kind of natural protection. The sites at Charles Mears State Park and Ludington State Park which are in the same area as Pere Marquette are protected by Little Sable Point and Big Sable Point respectively and therefore are subjected to somewhat milder attack.

An analogous observation can be made for the sites at Stevensville and Michiana. These sites are near to each other and therefore one might assume they are subjected to the same storm conditions. However, Table 2 shows that the Stevensville site was subjected to over 200 more hours of waves larger than 4 feet than the site at Michiana. This occurred because the Stevensville site is exposed to longer fetches in the direction of the prevailing westerly winds than the Michiana site and also because at the Stevensville site the waves from the west reach the breaking point with less refraction than at the Michiana site.

The Lakeport and Sanilac sites are also close together near the southern end of Lake Huron. However, the wave climate is more severe at the Sanilac sites because the fetches from the southwest are considerably shorter at Lakeport. The duration of storm waves at both of these locations is very much smaller than at the Lake Michigan sites because easterly winds are much less frequent than westerly winds.

Brevort, although located on the northern shore of Lake Michigan, has the lowest ratio of waves larger than 4 ft. to the number of hours of storm action. This occurs because the site is well shielded from most southern storms by the shoals that extend west and by the group of islands and shoals that surround Beaver Island.

TABLE 2

Site	Hours of Storm Reported	No. Storms	No. of Hours with H_B Equal or Greater 4	6	8	10	12
Stevensville	1742	56	937	287	70	20	3
Michiana	1742	56	704	198	62	25	4
Ludington State Park	1991	62	778	257	51	14	1
Big Sable Point	1991	62	778	257	51	14	1
C. Mears State Park	1991	62	852	277	74	21	7
Pere Marquette	1991	62	1002	360	115	30	3
Empire (1)	Not Computed						
Brevort	613	26	210	59	9	-	-
Manistique (2)	Not Computed						
Eagle Harbor	798	27	364	76	5	-	-
Sanilac: 11 and 26	228	11	117	23	-	-	-
Lakeport	228	11	93	11	-	-	-
Tawas Point	163	8	23	-	-	-	-
Tawas City	163	8	-	-	-	-	-
East Tawas	163	8	12	7	-	-	-
Little Girls Point	38	2	12	7	-	-	-
Whitefish Bay	42	3	1	-	-	-	-
Marquette (3)	Not Computed						

(1) Installation failed during 1st year.

(2) Installation covered by other construction.

(3) Wave data are being prepared by other observers.

In the Tawas region there are three experimental sites, Tawas Point, East Tawas and Tawas City. Of these three sites only the one at Tawas Point (Coast Guard Station) is located outside of Tawas Bay. The waves at Tawas Point are given in Table 2. The other two sites are significantly protected by the bay. For waves approaching directly from the east, the diffraction coefficient for East Tawas is $K_d = .06$ and for Tawas City is $K_d = .11$. This severe diffraction plus refraction and bottom friction reduce the damage potential from waves coming from the east to negligible levels. It would appear that waves coming directly from the south could have the most damage potential for the East Tawas and Tawas City sites because the diffraction coefficients are 1.10 and 1.0 respectively, and the refraction coefficients are about 0.70. However, the fetch from this direction is only about 25 nautical miles and the average depth is only about 20 ft. In addition, the Charity Islands and Point Lookout are located about midway on the fetch and provide some protection. At present MESH is not programed to estimate wave growth in shallow water, but by using the charts of reference (10), the wave height caused by 20 knot winds at this location is estimated to be between 2 to 3 ft., with a wave period of about 3.5 seconds. Therefore, it can be safely assumed that when the influence of refraction and damping are brought into the final picture, the waves coming from the south are greatly reduced before reaching the experimental sites located inside Tawas Bay.

The amount of information available for the site at Little Girls Point is very small, perhaps due to an absence of any significant storms during the period covered by this study. It should be noted that this site is well shielded from storms approaching from the west by the group of islands off the coast of Point Detour, Wisconsin. Storms from the north and the northeast have considerably greater damage potential, but no large storms from these directions have been reported. Waves from the north would reach this site after minor refraction, but waves from the northeast would undergo a substantial amount of refraction, although the latter could still be a source of high waves due to a fetch of over 160 nautical miles.

The last site investigated is the one located in Whitefish Bay (Paradise) ~~where only 3 storms were reported. These storms had directions from the~~ northeast and southeast, which caused the fetch reaches to be confined within the Bay. As a result, no major surf action at the experimental site was recorded.

4.0 SUMMARY AND CONCLUSIONS

During 1978-79, the last year of a project to evaluate low-cost shore protection, we determined the wave history of eighteen field installations for the period October 1974 through November 1978. This was done by estimating wind over the lakes from wind observations on land. The deep water wave heights and periods were then estimated from the lake wind. Finally, the effects of shoaling and refraction were considered at each location in order to estimate the number of hours of breaker heights of various sizes. These computations were facilitated by developing a computer algorithm for the solution of the Sverdrup-Munk-Bretschneider equations relating deepwater wave heights and periods to the wind parameters. A detailed description of computing procedure and the computer output is presented in this report.

Wave histories were developed for the demonstration sites for the four year period 1974-1978. As shown in Table 2 the six sites on Lake Michigan which face to the west were subjected to breaking waves of four feet or greater for 700 to 1000 hours. For about one third of that time, waves were six feet or greater. The sites on Lake Huron which face to the east were subjected to only about 100 hours of four-foot waves. The difference is due to the prevalence of westerly winds in the Great Lakes region.

Altogether, the protective structures were observed for six years. Although this is not a sufficient period to make an economic feasibility evaluation, it provided some valuable information on the various types of protection installed, especially on procedures which displayed weaknesses. Indeed, even the shorter periods of wave action on the Lake Huron sites compared with the Lake Michigan sites produced several failures. Some specific conclusions are given in the following paragraphs.

Sand nourishment has been very effective at Tawas City, where the sand was placed between a groin and a short pier. However, sand placed on the open beach a few miles to the north, at East Tawas, was quickly moved away even though this area had not been exposed to large waves. A massive sand fill at Marquette is still quite effective although about 80 per cent of this fill has been displaced. The loss of sand is partly because the available sand had a very large percentage of material finer than the natural beach material. This beach is shielded to some extent by four small islands. At Ludington, annual sand fill along with two steel groins have provided protection in a very high energy location.

Our pair of groins in a high energy location (Stevensville) has created a protective beach which has been quite effective. One of the groins, constructed of wood sheeting, has remained intact. The other, consisting of two 40 inch Longard tubes, has settled considerably and has suffered some damage. The system of six groins at Sanilac, Sec. 26, has collected beach material. One groin, constructed of asphalt and stone, has suffered some damage and also a groin which was constructed of sand-filled bags has suffered considerable damage even though there have been only 23 hours of waves as large as six feet. One of the two steel groins used along with sand nourishment at Ludington was severely damaged and required partial replacement. The most satisfactory type of groin construction has been wood sheeting reinforced with wooden piles and a horizontal wale.

Rubble revetments have provided excellent protection at three locations despite the small stone used; small stone kept down the cost. In each case, a foundation layer of smaller stones was used and the toe was protected by a trench filled with stones. The revetment at Eagle Harbor on Lake Superior was subjected to 76 hours of waves having breaker heights of six feet or more. The other two, at Tawas Point and Whitefish Bay, have been subjected to six-foot waves for only 23 and 12 hours, respectively. The revetment at Whitefish Bay contains wooden groins extending landward from the revetment to protect the fill which was used to reclaim the park space which had been lost to erosion.

An offshore breakwater at Pere Marquette failed during large storms. It was constructed of one-foot thick concrete vertical zig-zag walls placed on the sand bottom. The breakwater was undermined by excessive turbulence at the face of the wall causing it to settle and break in places so that its crest sunk to the still water level or slightly below it. The over-topping waves were then able to attack the bluff and cause rapid erosion.

A number of structures were of the massive type which rest on the bottom and derive their strength from their own weight. Besides the offshore wall discussed above, these structures were sand-filled tubes or bags which were used for toe protection and in groins and gabions. In order to achieve low cost, these structures were placed on the bottom without a foundation or toe protection. As a result they have settled due to undermining or have failed by sliding. The sliding was greatest for devices which were made of smooth fabric or rested on a fabric surface.

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