CHAPTER 29

ATTENUATION OF WIND WAVES BY A HYDRAULIC BREAKWATER

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ABSTRACT

Waves generated in a tank by air blowing over the water surface were subjected to a horizontal current of water created by horizontal water jets issuing from a manifold at the water surface (hydraulic breakwater). The energy spectra of the waves were computed for conditions before and after the hydraulic breakwater was turned on. It was found that the shorter, steeper wave components were attenuated to a much greater extent than were the longer wave components. Thus, although a large portion of the wave energy could get past such a breakwater, the waves in the lee of the breakwater looked considerably lower to the observer.

INTRODUCTION

The literature* on harbor protection contains a number of articles on an "air breakwater" or a "pneumatic breakwater." This type of breakwater consists of a pipe on the ocean bottom, supplied with compressed air which issues from the pipe through a series of ports. In some manne this results in a decrease in wave height in the lee of the breakwater. Th mechanism, or mechanisms, by which the waves are attenuated is in dispute, with apparent discrepancies between results of model and prototype studies. The most likely mechanism is the one suggested by Schijf (1940) and studied theoretically by Taylor (1955). The air bubbles formed when the air discharges through the ports mix with the water, and as the mixture is less dense than the surrounding water, it rises. The air bubbles escape to the atmosphere at the surface while the water turns through ninety degrees forming horizontal currents. The thickness of the current was found to be proportional to the one-third power of the volume rate of flow of air per foot of pipe. The claim that the bubbles themselves some how attenuate the waves has been shown to be incorrect both theoretically (Schiff, 1948a, 1948b) and experimentally (Carr, 1950) in studies of the effect of a bubble-water field one-half a wave length thick.

*See, for example, Green (1961) which contains an extensive list of references on the subject, and a discussion of this paper by Schijf (1961).

ATTENUATION OF WIND WAVES BY A HYDRAULIC BREAKWATER

Because it seemed that the surface current produced by the pneumatic breakwater had the main effect on waves, tests were made using a series of horizontal jets to generate a surface current, this device being called a hydraulic breakwater (Dilley, 1958; Horikawa, 1958; Snyder, 1959; Williams, 1960). It was found that a manifold placed at the mean water level, discharging water jets horizontally into the waves, generated a surface current similar to the current created by the pneumatic breakwater, and that this current had the same effect on the waves as in the case of a pneumatic breakwater.

Why should there be a discrepancy between the observations in various model studies and claims made for the prototype pneumatic breakwater? Many of these claims stem from early stories of experiments at a pier at El Segundo, California. However, it was concluded from these studies that the apparatus was of no utility or benefit and therefore abandoned (U.S. District Court, 1923), but these conclusions apparently were never published. Thus, part of the claims are not valid. It is believed that the reasons for the claims are in part real, and in part psychological. Waves that are still in the generating area are steep, many of them breaking due to their steepness, and many of them nearly breaking. Because of this, an opposing current, which will cause the waves to steepen, will force many of them to break and dissipate wave energy. In addition, waves in the ocean are irregular and for many purposes can be described by an energy spectrum. On the other hand most laboratory tests are performed with periodic waves of uniform height. For many purposes the laboratory waves can be, and have been, associated with the portion of the energy spectrum in the vicinity of the peak energy density which in turn is closely related to the significant wave (Wiegel, 1960). Now, suppose we can generate a surface current by a pneumatic breakwater, or some other means, that is neither thick enough nor fast enough to stop the longer component waves in the spectrum associated with the maximum energy density, but which can stop the shortest wave components and cause those wave components somewhat longer than the shortest waves to steepen and break. Most of the wave power will be transmitted into the lee of the breakwater, but it will look much smoother than the original wave system as the short steep wave components will have either been reflected by the current or greatly attenuated. This is the psychological part--the wave system no longer looks as high as it did before.

If irregular wave systems can be treated to a certain extent as a superposition of linear wave trains, then a current might be able to affect the wave components in the selective manner described above. The data obtained by Kurihara (1958) in his field tests suggested to the authors that this might be possible. In order to test this possibility laboratory experiments were performed using a hydraulic breakwater to generate the surface current, and blowing wind over the water surface to create the waves.

STATISTICAL ANALYSIS OF WIND-GENERATED WAVES

It has been recognized from analyses of wind-wave records that the ordinate-time history of the water surface may be represented for many practical purposes as a "stationary Gaussian process." This statistical model in turn implies a distribution of energy over a range of frequencies which is independent of time and a distribution of probability over a range of ordinates (i. e. ordinates of the time-surface elevation record) (Putz, 1954, Pierson, 1954, Bretschneider, 1959).

The most direct method of analyzing any given wind-wave record is to extract from it the apparent wave heights and periods--that is, to consider the individual "bumps" of the record to be waves themselves. The problem remains then to show that such "wave heights" and "wave periods" are statistically congruous with the stationary Gaussian process This has been done by Putz (1954). In regard to the probability distribution curve containing two parameters σ_0 , the root-mean-square ordinate, and ρ_0, the ratio of the number of zero crossings of the ordinate to the number of zero crossings of the first derivative of the ordinate (the number of wave maxima and minima). Zero crossing means the crossing of the time axis, this axis being through the mean of the ordinates. When $\rho_0 = 1$, this distribution function coincides with the Rayleigh distribution function. A comparison of this derived probability distribution with "wave heights" as extracted from a record of 20-minute length shows that the actual distribution coincides with a theoretical distribution curve where $\rho_0 = .92$. Thus, the actual distribution curve is approximately the Rayleigh distribution curve. This result is in agreement with the work of several others, namely, Bretschneider (1959), Longuet-Higgins (1959), and Miche (1952).

In regard to the energy distribution with respect to frequency, Putz employed the fact that the Fourier spectrum of the covariance of the stationary Gaussian process is the energy spectrum of the wave record. Further, the covariance is shown to depend on the zero crossings of the record. Consequently the apparent periods of the waves are related to the energy spectrum. These results apply to wave records of length no greater than about twenty minutes, as records of longer duration do not satisfy the time stationary requirement.

In view of the above it may be concluded that the "wave heights" and "wave periods" of the individual "bumps," as extracted from the wave record, are compatible with the stationary Gaussian process and may therefore serve as indications of the energy distribution with respect to frequency and of the probability distribution of the ordinates.

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EXPERIMENTAL APPARATUS AND PROCEDURE

The experiments were carried out in two different wind-wave tanks. The larger tank was 106' long by 3' deep by 1' wide, while the smaller tank was 60' long by 1/28' deep by 1' wide. The larger tank was located on the U. C. campus and will be referred to as the UCB tank, and all data taken from it will be noted as UCB data. The smaller tank was located at the University's Richmond Field Station and will be referred to as the RFS tank, and the data taken from it noted as the RFS data. The wind in the RFS tank was generated by a blower, while the wind in the UCB tank was generated by an exhaust fan. Wind speeds in both tanks were measured with a pitot tube and a draft gage.

The wind waves were recorded by parallel wire resistance probes connected to a Brush oscillograph. The flow through the breakwater for the UCB tank tests was measured by an orifice plate inserted in the breakwater supply line, and a water manometer. The breakwater flow rate in the RFS tank was measured volumetrically by noting the change in water level in the tank during a given run together with the run time. As the run times were short, the increase in water levels in the tank was not sufficient to affect the performance of the breakwater during the run.

The hydraulic breakwater used to generate the horizontal current is shown in Fig. 1. The designation $\lambda = 8$ on the drawing is to tie it in with the results of several other scale breakwaters used in model tests on the scale effect of hydraulic breakwaters (Williams, 1960). A performance curve for the breakwater is shown in Fig. 2. It shows the length of the longest wave that can be attenuated to only 5% of its original height for a given breakwater discharge. This curve pertains essentially to nearly deep water waves since the points which define the curve resulted from data where $1.77 \leq L/d \leq 4.92$. Figures 3 and 4 show the arrangement of the resistance probes, breakwater, pitot tube, etc., for the UCB and RFS tanks, respectively. The experimental procedure was similar in both tanks, except for measuring the breakwater discharge. First, the waves were recorded for a given wind speed without the breakwater in the tank; next, the waves were recorded after the breakwater was installed but before it was turned on, and, finally, the waves were recorded for several breakwater discharges while holding the wind speed constant. This procedure was repeated in the UCB tank for two water depths, 6 inches and 27 inches, using one wind speed at each depth, and in the RFS tank using one depth, 6 inches, and two wind speeds. In the UCB experiments only two resistance gages were used, one in front of and one behind the breakwater. In the RFS experiments three gages were used, two behind and one in front of the breakwater. The exact positions of these gages with respect to the breakwater are shown in Figs. 3 and 4.

ANALYSIS OF DATA

Two problems presented themselves: first, the method of taking the sample and second, the size of the sample. In view of the excessive amount of work required in extracting the "wave heights" and "wave peri ods" from a wave record, an attempt was made to determine the minimu length of record that would give a realistic account of the physical situati involved. A record from the UCB tank was analyzed using samples of 100 consecutive waves and 50 consecutives waves, and the cumulative dis tribution curves were plotted of the "heights" and the "periods" (see Figures 5a and 5b). Since there was no appreciable difference in these two curves, it was decided that about 50 waves could be selected as an adequate sample size.

There are several methods available for picking the "wave heights" ar "wave periods" from the record. Two of these methods are the zero-upcrossing method and the crest-to-trough or trough-to-crest method. In the zero-upcrossing method the periods are taken as the distance betwee the successive upcrossings of the wave record, f(t), with the axis througl the mean of the ordinates. This quantity is denoted as $\tilde{T}$. The wave height, H, corresponding to a given $\tilde{T}$ is taken as the vertical distance from the crest to the trough found on the interval $\tilde{T}$. The crest-to-trougl method defines the "period" as the distance between successive dominant crests. This period is indicated by T. The height is taken as the vertic distance from the first crest of T to that point which is lowest between th two crests. The trough-to-crest method defines T as the distance betwe successive troughs, and the height, H, as the distance between the first trough and the highest point on the record between the two troughs. Fro these definitions it is clear that $\bar{T} \geq \bar{\tilde{T}}$ where the bars indicate averages of a number (N) of waves. The equality sign holds in the limit as $N \to \infty$, provided there are only a finite number of small ripples which intersect the time axis in the unlimited record. These ripples are considered as "waves" in the zero-upcrossing method, but are neglected in the crest-to-trough method (see Pierson, 1954).

To illustrate the differences between these two methods a section of wave record was analyzed both ways, and the resulting cumulative distribution curves plotted in Figs. 5a and 5b. Figure 5a illustrates the difference between the two methods for a 100 wave-period sample, and the difference between a 50 wave-period sample and a 100 wave-period sample for the zero-upcrossing method. Figure 5b shows the difference

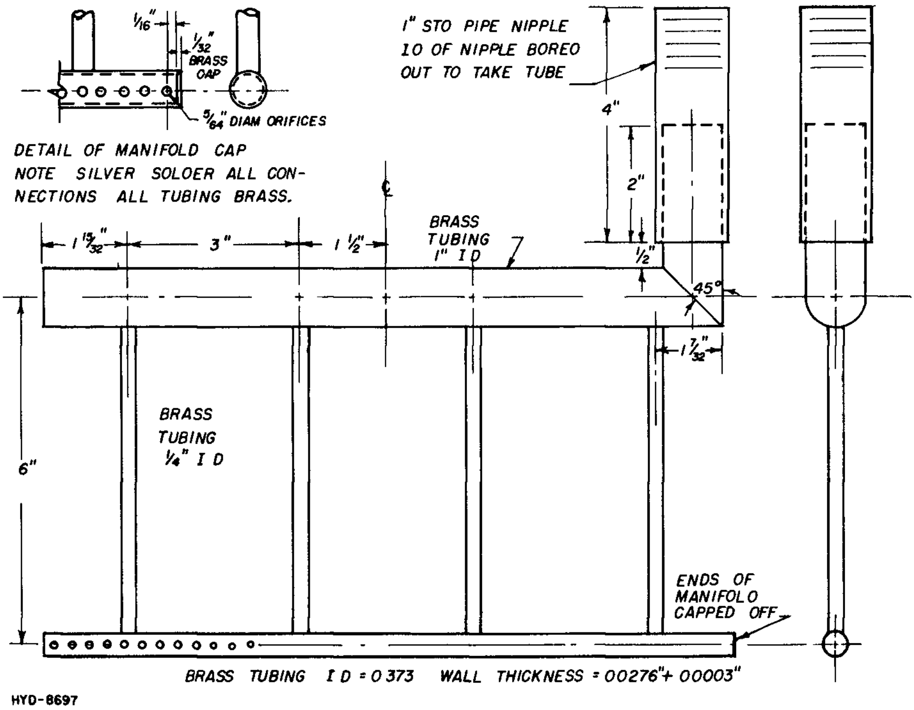

Fig. 1 - Model manifold hydraulic breakwater, λ= 8

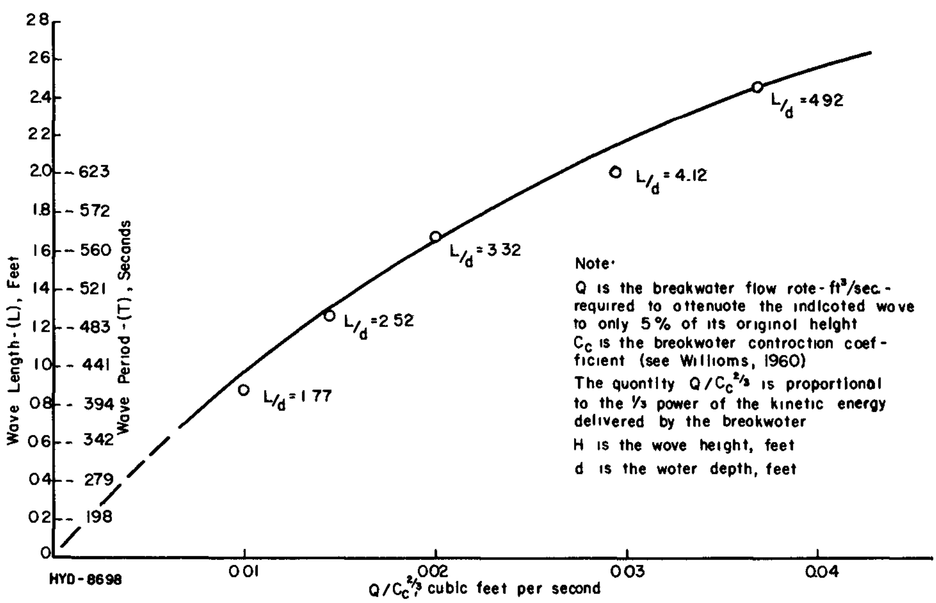

Fig. 2 - Wave length (period) vs $Q/C^{2/3}$ for λ = 8 model breakwater, wave steepness - H/L = .038

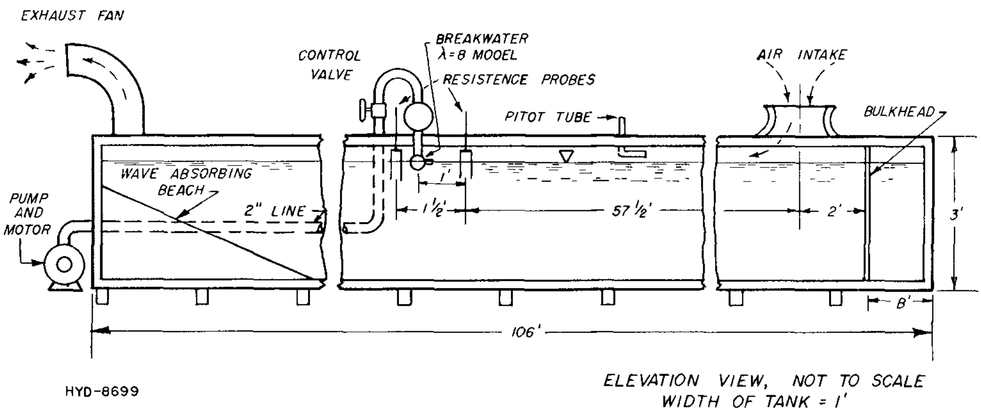

Fig. 3 - UCB wind-wave tank

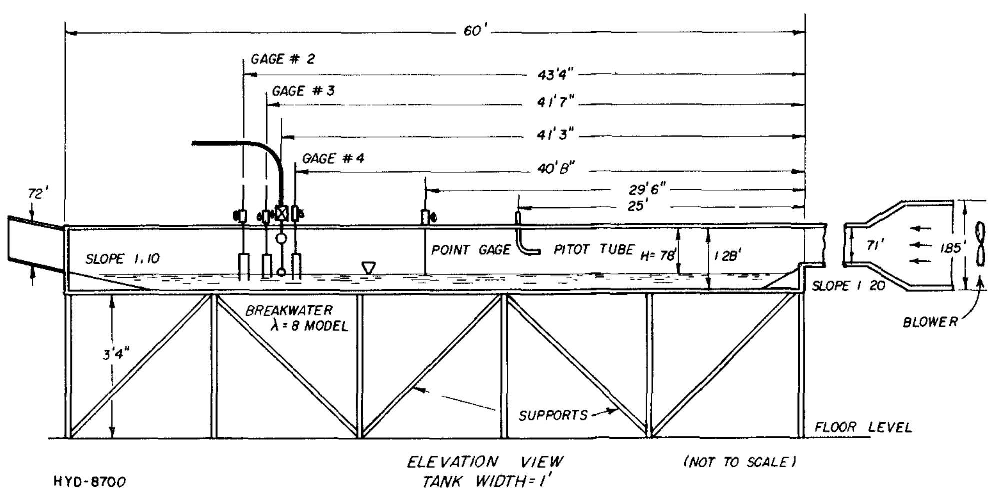

Fig. 4 - Richmond Field Station wind-wave tank

between a 50 wave-height sample and a 100 wave-height sample for the zero-upcrossing method. It is to be noted that the wave heights taken from the record by both of these methods will be identical provided there are no small ripples intersecting the time axis between two larger waves. In view of this fact and the results indicated in Fig. 5a, it was decided that the selection of the method could be based solely on its utility for the purpose at hand, and subsequently the zero-upcrossing method for the sample size of 50 plus waves was chosen. In these and subsequent figures the term "maximum wind speed" refers to maximum speed obtained from a velocity traverse from near the water surface to the top of the tank (see Fig. 6).

It should be emphasized that the sample size of 50, plus, waves pertained only to the wave record obtained without the breakwater in the tank. These 50 plus waves represented a time interval on the wave record of 25 to 35 seconds. It was this time interval that was kept constant throughout a set of breakwater discharges at a given wind speed, and consequently determined the sample sizes for each breakwater discharge. This was done so that the total energies of the wave spectrum could be compared realistically with one another before and during breakwater operation. Also, an effort was made to select as closely as possible the same set of "waves" at each gage location for a given run. An example of the records before and after the hydraulic breakwater was turned on is given in Fig. 7.

After the wave heights and periods were measured on records from the several wave gages for a given breakwater discharge and wind speed, the following quantities were calculated: $\overline{H}$, $\overline{T}$, σ_H, $\sigma_{\tilde{T}}$, $\overline{H^2}$, $\overline{H/\tilde{T}^2}$, $H_{1/3}$ and $H_{1/3}/\overline{H}$. Here the bars denote arithmetic averages, σ_H and $\sigma_{\tilde{T}}$ are the usual standard deviations of H and $\tilde{T}$ respectively, $\overline{H/T^2}$ is taken as being representative of the wave steepness, and $\overline{H^2}$ is representative of the wave energy per unit surface area. These quantities were calculated for the conditions of no breakwater in the tank, breakwater in tank with zero discharge, and at least two discharges. For the RFS tank two such sets of quantities were calculated, one for each wind speed used. These results are presented in Table 1. Similarly two sets of such quantities were calculated from the UCB tank records, one for each water depth used. These results are presented in Table 2.

Finally, joint frequency plots for H^2 and T, and frequency histograms for H/T^2 were plotted. These data appear in Figs. 8 through 11 for the RFS tests and in Figs. 12 through 15 for the UCB tests.

TABLE 1. SUMMARY OF RESULTS, RFS DATA

Gage No.	Number of Waves	$\bar{H}$, ft.	$\bar{T}$, sec.	σ_H, ft.	σ_T, sec.	$\overline{H^2}$, ft^2	$\overline{H/T^2}$	$H_{1/3}$, ft.	$H_{1/3}/\bar{H}$
Maximum wind speed = 29.5 ft/sec, water depth 0.5 ft.									
No breakwater in tunnel									
2	54	.081	.475	.0271	.0621	.0072	.358	.111	1.37
3	54	.079	.472	.0228	.0615	.0068	.361	.106	1.34
4	54	.081	.465	.0282	.0638	.0073	.387	.110	1.36
Breakwater in tunnel, no discharge									
2	55	.091	.480	.0253	.0719	.0090	.408	.119	1.31
3	55	.079	.480	.0257	.0748	.0069	.352	.109	1.38
4	55	.083	.478	.0230	.0731	.0075	.381	.108	1.30
Breakwater in tunnel, Q = .00775 cfs., C_c = 815*									
2	52	.077	.511	.0228	.0651	.0065	.305	.102	1.33
3	53	.059	.492	.0221	.0737	.0039	.253	.083	1.41
4	55	.092	.471	.0345	.0782	.0097	.417	.129	1.40
Breakwater in tunnel, Q = .0104 cfs. C_c = .700									
2	52	.064	.516	.0260	.0832	.0047	.244	.091	1.42
3	50	.057	.535	.0242	.0764	.0039	.205	.084	1.47
4	53	.111	.500	.0393	.0605	.0139	.437	.151	1.36
Maximum wind speed = 41.4 ft/sec., water depth = 0.5 ft.									
No breakwater in tunnel									
2	55	.129	.636	.0323	.0983	.0178	.335	.163	1.26
3	56	.134	.622	.0329	.112	.0190	.372	.167	1.24
4	56	.138	.627	.0396	.122	.0200	.369	.178	1.29
Maximum wind speed = 41.4 ft/sec., water depth = 0.5 ft.									
Breakwater in tunnel, no discharge									
2	55	.129	.635	.0300	.132	.0178	.347	.159	1.23
3	55	.137	.624	.0288	.128	.0197	.383	.169	1.23
4	55	.127	.630	.0291	.113	.0170	.343	.159	1.24
Breakwater in tunnel, Q = .0155 cfs., C_c = .640									
2	53	.112	.662	.0309	.104	.0134	.260	.144	1.30
3	51	.116	.677	.0329	.0908	.0152	.277	.156	1.34
4	53	.188	.654	.0427	.0868	.0371	.455	.232	1.23
Breakwater in tunnel, Q = .0190 cfs., C_c = .625									
2	52	.095	.671	.0316	.136	.0101	.223	.129	1.36
3	49	.112	.699	.0355	.123	.0128	.230	.149	1.33
4	54	.168	.645	.0536	.106	.0300	.423	.226	1.35

*C_c is the discharge coefficient of the orifice as determined experimentally.

TABLE 2. SUMMARY OF RESULTS, UCB DATA

Gage No.	Number of Waves	$\overline{H}$, ft.	$\overline{T}$, sec.	σ_H, ft.	σ_T, sec.	$\overline{H^2}$, ft^2	$\overline{H/T^2}$	$H_{1/3}$, ft.	$H_{1/3}/\overline{H}$
Maximum wind speed = 21.3 ft/sec, water depth = 0.5 ft.									
	No breakwater in tunnel								
2	54	.064	.478	.0203	.0394	.0044	.287	.0861	1.35
3	54	.065	.475	.0185	.0336	.0045	.289	.0823	1.27
	Breakwater in tunnel, no discharge								
2	53	.0614	.481	.0203	.0471	.0042	.273	.0844	1.37
3	53	.062	.477	.0193	.0405	.0042	.277	.0835	1.35
	Breakwater in tunnel, Q = .0059 cfs, C_c = .970								
2	53	.0607	.494	.0178	.0565	.0040	.251	.0793	1.31
3	53	.0800	.494	.0255	.0519	.0071	.338	.110	1.38
	Breakwater in tunnel, Q = .0071 cfs, C_c = .860								
2	52	.043	.510	.0157	.0466	.0021	.164	.0611	1.42
3	53	.086	.500	.0245	.0537	.0080	.351	.111	1.29
	Breakwater in tunnel, Q = .0079 cfs, C_c = .805								
2	49	.034	.544	.0122	.0502	.0014	.118	.0474	1.39
3	53	.094	.500	.0341	.0590	.0098	.390	.133	1.41
	Breakwater in tunnel, Q = .0089 cfs, C_c = .757								
2	48	.019	.550	.0081	.0788	.00041	.0645	.0271	1.43
3	53	.098	.500	.0276	.0602	.0104	.402	.127	1.30
Maximum wind speed = 31.9 ft/sec, water depth = 2.25 ft.									
	No breakwater in tunnel*								
2	52	.170	.532	.0395	.0758	.0305	.622	.212	1.25
3	54	.164	.506	.0387	.0810	.0283	.641	.200	1.22
Maximum wind speed = 31.9 ft/sec, water depth = 2.25 ft.									
	Breakwater in tunnel, no discharge								
2	55	.131	.486	.0360	.0647	.0186	.559	.171	1.31
3	55	.110	.489	.0323	.0582	.0132	.465	.144	1.31
	Breakwater in tunnel, Q = .0102 cfs, C_c = .713								
2	50	.095	.535	.0323	.0554	.0100	.330	.131	1.38
3	54	.123	.489	.0369	.0763	.0167	.520	.159	1.29
	Breakwater in tunnel, Q = .0120 cfs, C_c = .666								
2	51	.071	.530	.0285	.1120	.00590	.264	.104	1.46
3	51	.122	.536	.0456	.0808	.0170	.434	.173	1.42

*Note The results recorded for the condition "no breakwater in tunnel" were reduced from data which was not taken at the same time as the rest of the data for the above wind speed and water depth.

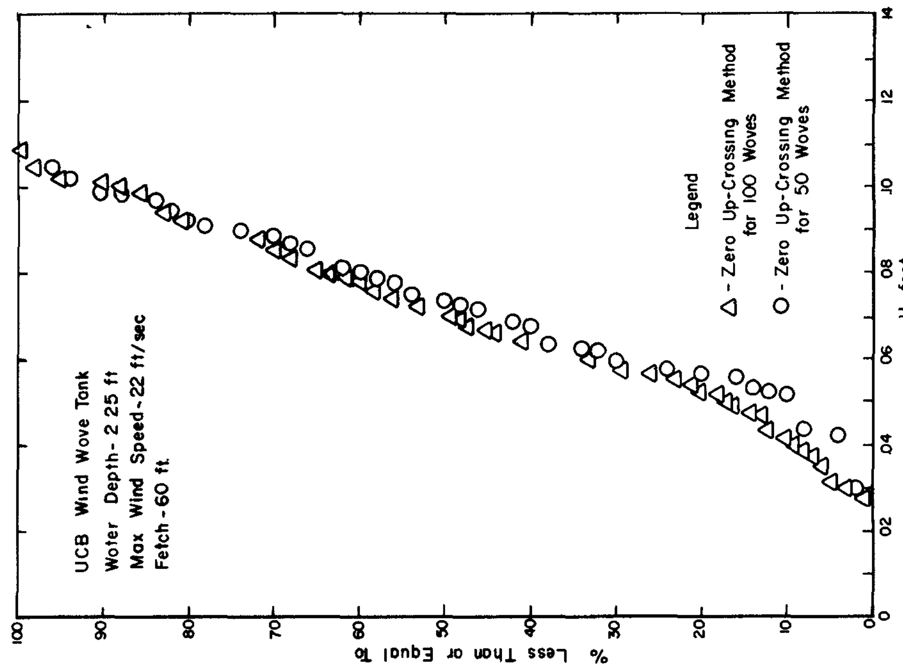

FIGURE 5b - COMPARISON OF CUMULATIVE FREQUENCY DISTRIBUTION CURVES OF WAVE HEIGHT FOR 50 AND 100 CONSECUTIVE WAVES

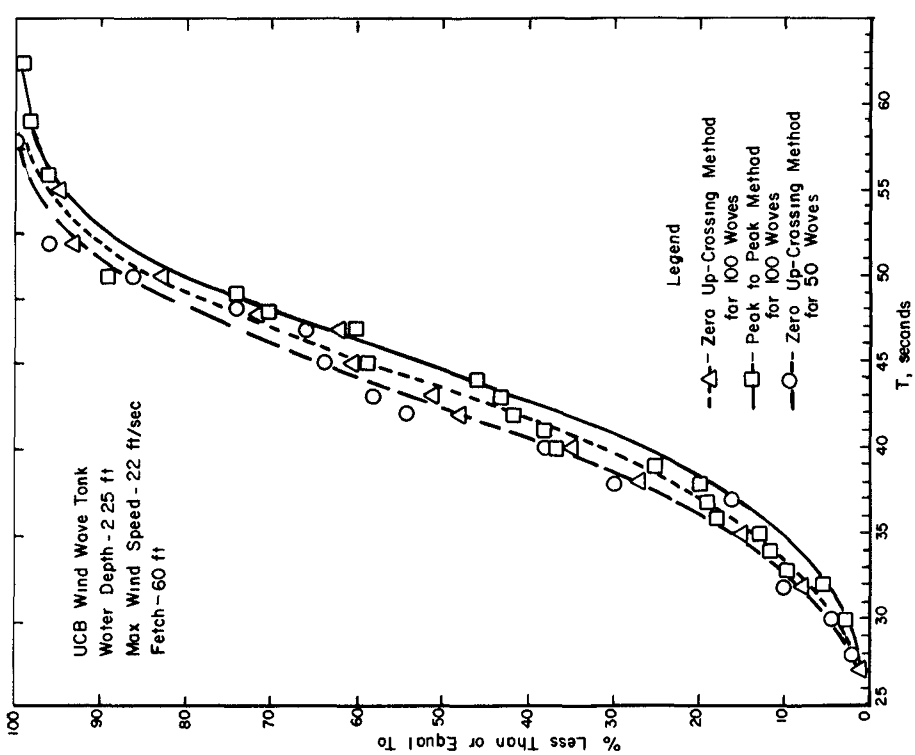

FIGURE 5a - COMPARISON OF CUMULATIVE FREQUENCY DISTRIBUTION CURVES OF WAVE PERIOD FOR 50 AND 100 CONSECUTIVE WAVES.

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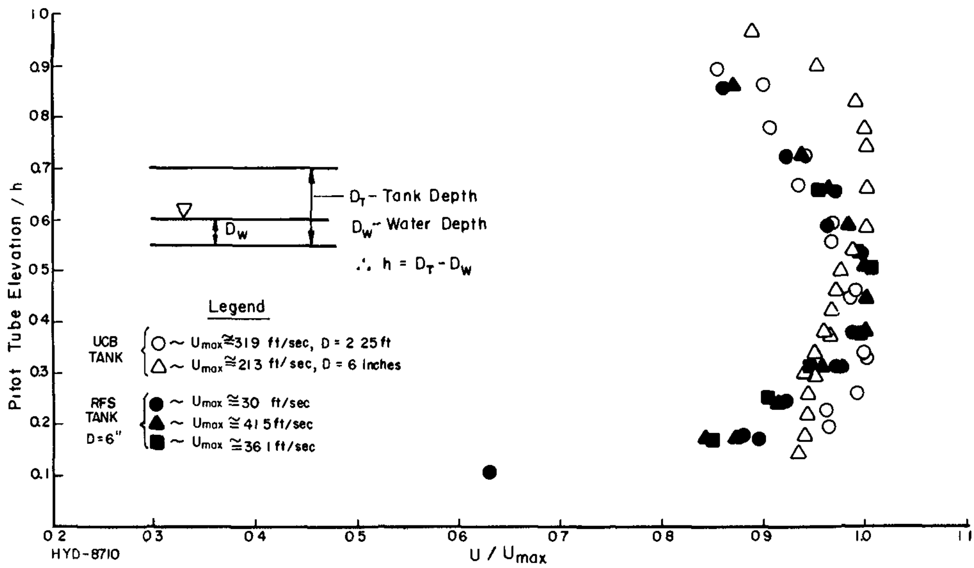

Fig. 6 - Wind speed profiles

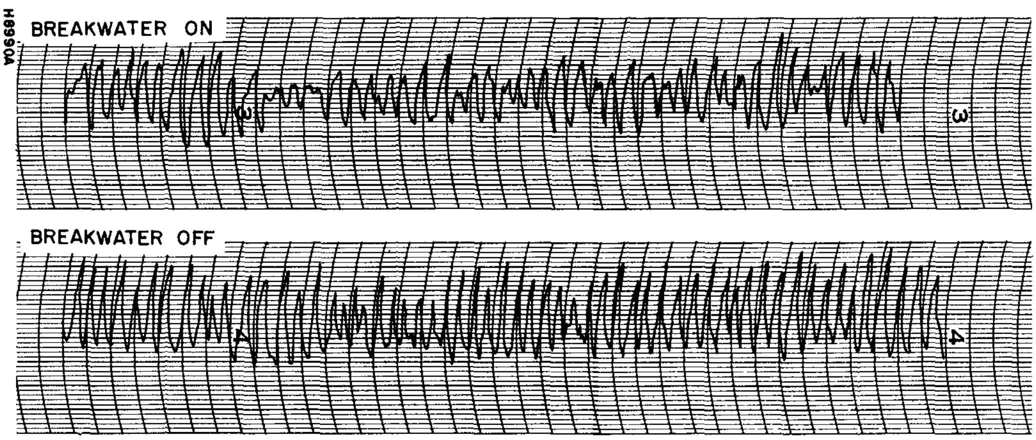

Fig. 7 - Sample wave record

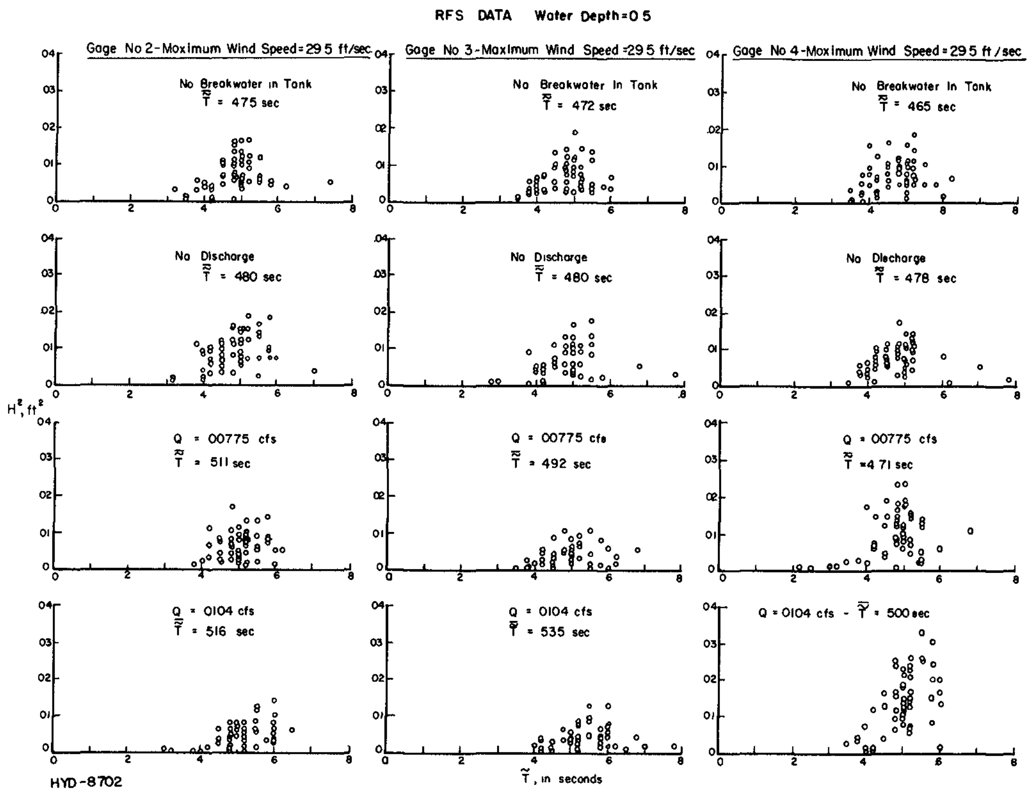

Fig. 8 - Joint frequency distribution

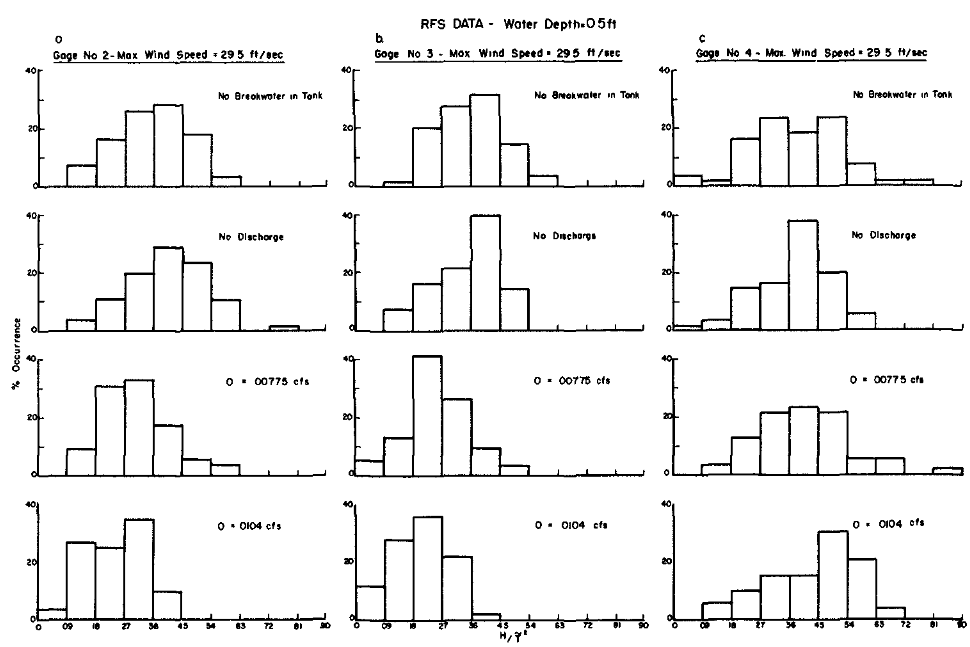

Fig. 9 - Frequency distribution

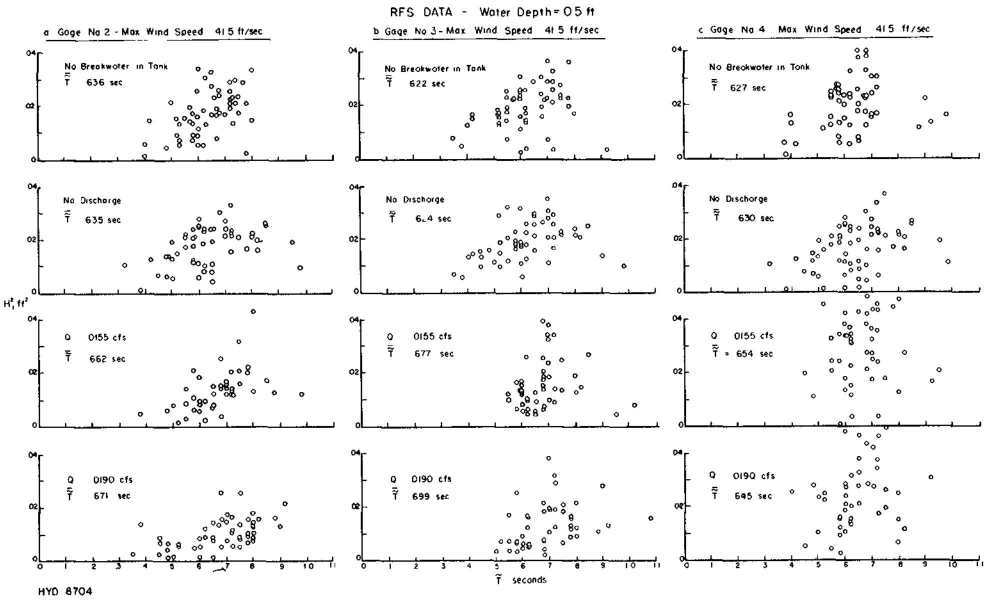

Fig. 10 - Joint frequency distribution

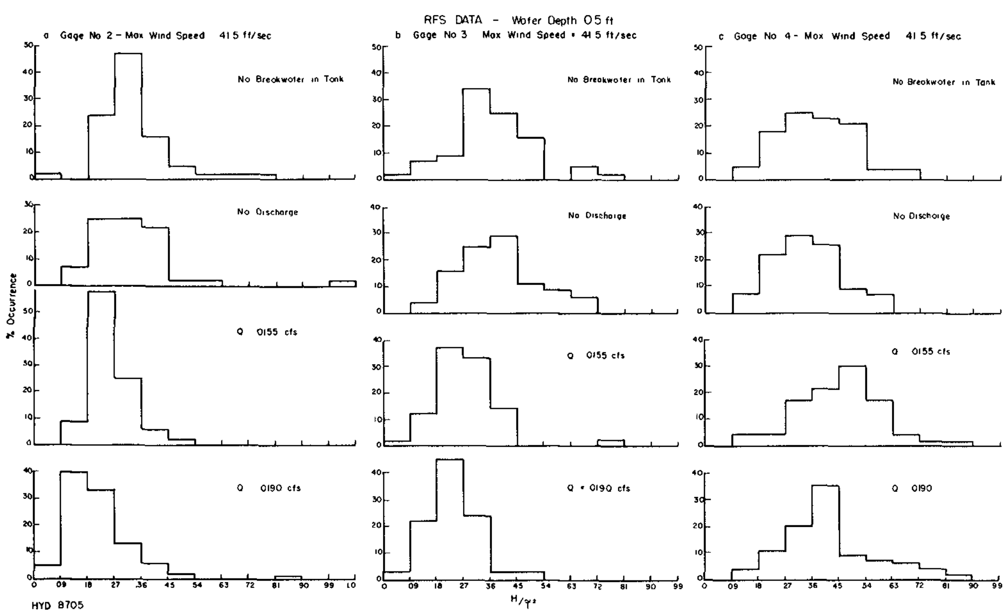

Fig. 11 - Joint frequency distribution, $H/\tilde{T}^2$

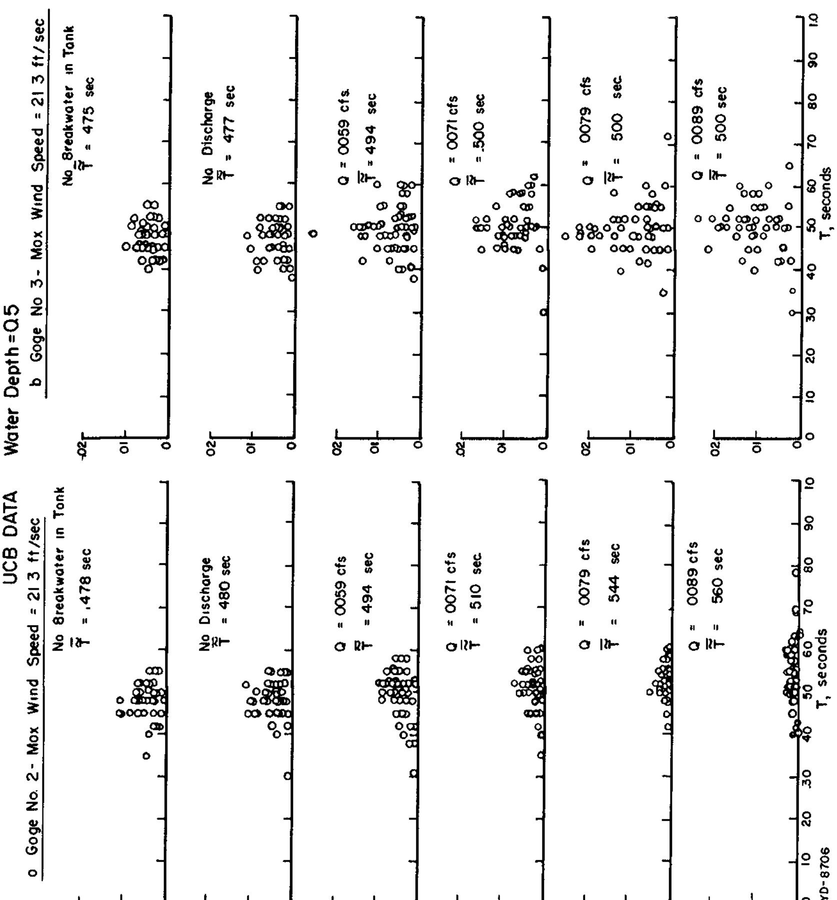

Fig. 12 - Joint frequency distribution

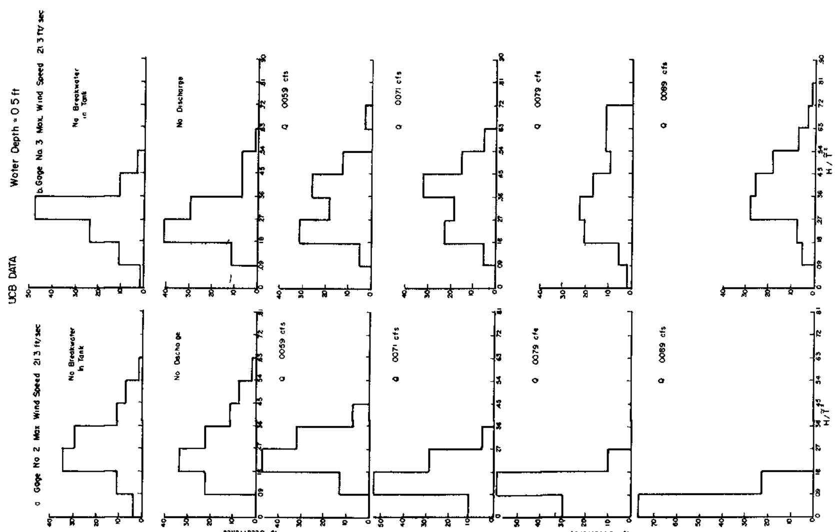

Fig. 13 - Joint frequency distribution, $H/\tilde{T}^2$

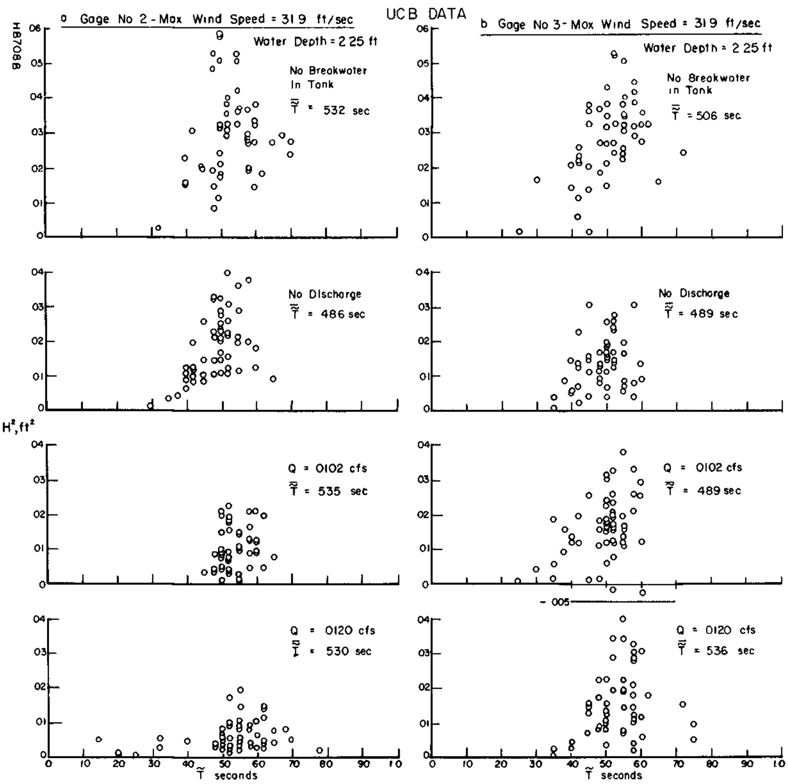

Fig. 14 - Joint frequency distribution

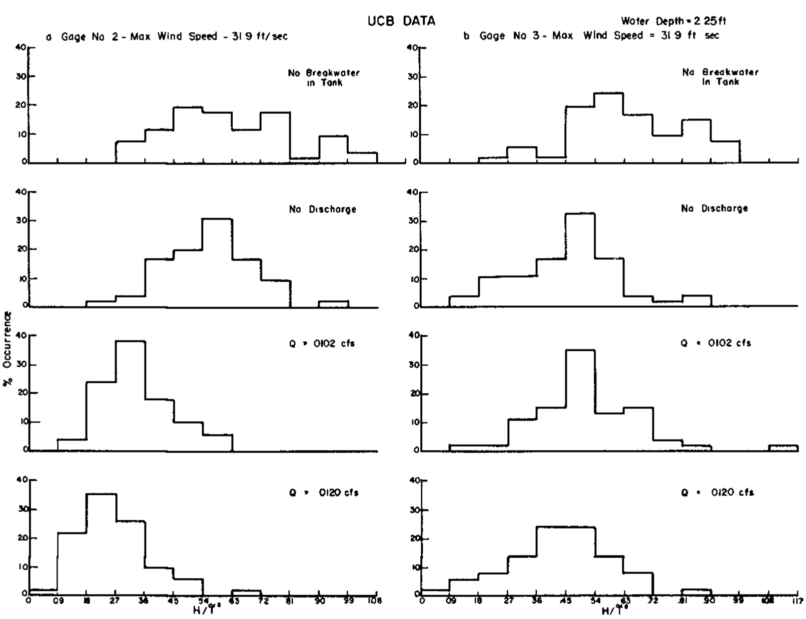

Fig. 15 - Frequency distribution, $H/\tilde{T}^2$

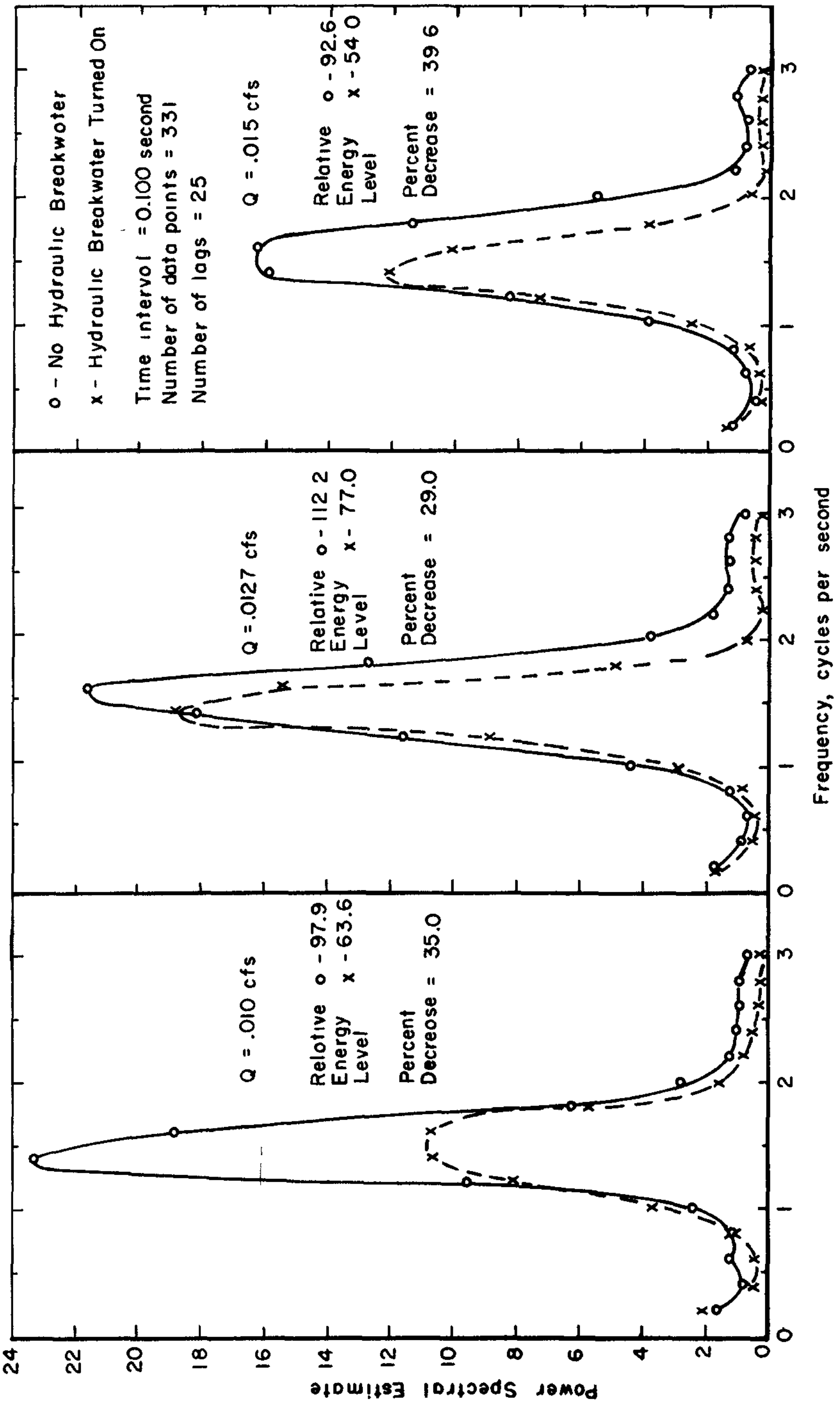

FIGURE 16 - SPECTRAL ANALYSIS OF WAVE RECORDS BEFORE AND AFTER HYDRAULIC BREAKWATER TURNED ON

RICHMOND FIELD STATION DATA -- ADDITIONAL RUNS
WIND SPEED 41 FT/SEC, FETCH - 41.5 FT

Several of the records were analyzed to obtain power spectra, using the IBM 7090 computer at the Computer Center, University of California, Berkeley, Calif., using the share sub-routine #574 "CS TUKS". The results are shown in Fig. 16.

EXPERIMENTAL RESULTS AND CONCLUSIONS

The joint distribution plots of H^2 and $\tilde{T}$ reveal two facts: (1) the highest waves occur at a period approximately equal to the average period, $\tilde{\bar{T}}$, both behind and ahead of the breakwater, and (2) the average period $\tilde{\bar{T}}$ increases consistently with increasing breakwater discharge for waves in the lee of the breakwater. Since the sample time interval for a given set of runs was constant, fact number two implies that the shorter period waves were eliminated from the given portion of the record. From fact number one it is clear that the steepest waves are those which in general have periods such that

$$\tilde{T} \leq \tilde{\bar{T}}$$

hence it is concluded that the steepest wave components are filtered out of the spectrum by the current and those longer than the average pass through the current. The frequency distribution plots of percent occurrence of $H/\tilde{T}^2$ also leads to this conclusion. That is, the range of $H/\tilde{T}^2$ narrows and $H/\tilde{T}^2$ decreases behind the breakwater for increasing breakwater discharge. In viewing these data it should be kept in mind that the wind was still blowing over the water surface so that some new relatively high frequency waves were formed in the lee of the breakwater by this wind. In front of the breakwater just the opposite occurs, the range widens and the average value increases for an increasing breakwater discharge. This filtering action was recently verified in experiments carried out for a two-frequency system of waves that were combined linearly with the higher frequency component (twice the frequency of the lower frequency component) completely filtered out and the lower frequency component getting through undistorted (Williams, 1961). However, this test was a simplification of the actual case, as the steep wind waves have higher harmonics which, in a spectral analysis, would show up as high frequency components as the spectral analyses presumes linear superposition. Thus, some of the higher frequency components shown in Fig. 16 are in reality higher harmonics of lower frequency components.

The fact that a surface current will filter out the shortest (which are th steepest in a wind-wave system, being at a limit of stability) waves while permitting the longer waves to pass may well explain the discrepancy in the reported success of breakwaters utilizing the action of such a current. That is, if the reported degree of effectiveness of the breakwater was based on visual observation (as it was in several instances, see Laurie, 1955) then the observer would likely be misled by the filtering action of the current, since long waves are not as easily detected by visual observation as the shorter and steeper waves. Hence, in spite of the apparent attenuation of the sea surface, considerable energy may still be transmitted by the longer waves, and it is this energy which may be damped out only at the expense of a disproportionate increase in the energy input to the breakwater. Thus, the difference between the power requirements as based on controlled laboratory experiments and those as observed in prototype action in the fields, as well as the disagreement between the different prototype operations themselves, may be due to inadequate field measurements. It is interesting to note that in the two prototype tests involving the visual observation (Laurie, 1955; Kurihara, 1958) the efficienc of the breakwater was reported as higher than predicted or observed elsewhere (Evans, 1955a, b). In one of the cases (Evans, 1952(b); Laurie, 1955) later measurements using a wave recorder rather than visual observations indicated little effect due to a pneumatic breakwater.

A reason which might account for an actual increase in effectiveness of the prototype breakwater has been suggested by R. C. H. Russell (personal communication, 1962). Because actual spectra of wind waves are two dimensional, some component waves are advancing at angles to the direction of mean wave advance, so that the component wave speed heading into the current is lower for these component waves which would permit a lower current to attenuate them. It is difficult to access this effect as the current would tend to cause the wave components to refract at the same time.

ACKNOWLEDGMENTS

The authors wish to thank Mr. Walter Louscutoff for his help in taking and reducing the data. The work was performed under Contract Nonr-222(46) with the Office of Naval Research.

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