DAMAGE PROGRESSION ON BREAKWATERS

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Abstract
An experiment is described consisting of seven relatively long-duration breakwater damage test series. The test series were conducted in a flume using irregular waves. New damage measurement techniques were developed and damage development data were acquired for breaking wave conditions. Wave height, wave period, water depth, storm duration, storm sequencing, and stone gradation were all varied systematically. The experiment yielded relationships for both temporal and spatial damage development. The relations by Melby and Kobayashi (1998a,b) for predicting temporal variations of mean damage with wave height and period varying with time in steps are shown to describe damage reasonably well (within one standard deviation) for new test series, although damage initiation is consistently underpredicted by more than a standard deviation. The prediction is shown to improve significantly if the initial profile adjustment is accounted for in the test series with relatively small cumulative damage.

1 INTRODUCTION

Contemporary breakwater armor stability design is founded on the well known work of Iribarren and Hudson. Much work has been done to extend these stability models for no-damage design conditions; but little work has been done to quantify damage progression. With only limited knowledge of damage progression, it is difficult to rationally determine life cycle costs or to evaluate and prioritize maintenance requirements for various projects. Further, determining the reliability with adequate accuracy for a particular design is impossible without prediction models for damage progression.

Existing stability formulas are limited to constant wave conditions [e.g. Hudson(1959) and van der Meer (1988)]. They are primarily intended to give a stable armor layer for a design level storm. These existing formulas can be used to design a new armor layer, but are not sufficient to predict life-cycle costs or to determine maintenance requirements for

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damaged rubble mounds. Additionally, existing formulas only predict the average damage, where damage is characterized by eroded area or number of displaced units. Melby and Kobayashi (1998a,b) showed that the damage variability along the structure is significant. Van der Meer (1988) showed that the shape of the eroded profile may be important in assessing the remaining capacity of an armor layer. Mansard et al. (1996) utilized the minimum cover layer thickness to describe failure of an armor layer. Melby and Kobayashi (1998a,b), hereafter referrred to as M&K, showed that this cover layer thickness as well as the depth and extent of erosion can be used to characterize the profile and all are quite variable along the slope. Existing stability formulas give no predictive capabilities for these profile parameters. Thus, existing stability prediction techniques cannot fulfill the need for predicting the future performance of existing structures.

2 PHYSICAL MODEL EXPERIMENT

A small-scale physical model experiment was designed to provide the hasis for an empirical model for spatial and temporal breakwater damage development. The experimental design was focused on quantifying damage for long duration tests composed of sequences of storms. The objectives of the experiment were as follows:

1. Quantify the progression of damage for multiple storm events, with water level, breaking wave height at toe, and storm duration being the primary variables of interest. Wave period and stone gradation were also varied systematically.
2. Quantify the uncertainty or scatter in damage due to natural variahility.
3. Determine whether the ordering of storm events effects the ultimate damage level.
4. Promote laboratory experimental standards for hreakwater damage progression.

The experiment utilized two small-scale ruhble mound breakwater sections in a wave flume. Figure 1 shows the flume profile and Figure 2 shows a typical structure cross section. A total of seven irregular wave test series were conducted as shown in Table 1. M&K describe the first three series. This paper summarizes results from all seven series. Each series was composed of a sequence of storms of varying wave height and water level. Parameters varied systematically from series to series were storm duration, storm ordering, wave height, water depth, wave period, and armor gradation. The structures were profiled using a newly developed automated profiler (Winkelman 1998). The profiles were used to determine the eroded cross sectional area and profile characteristics. The experimental setup, instrumentation, and initial tests are described in detail in M&K and will only be summarized herein.

Table 1
Summary of Test Series

Test Series	Test Type	Armor Type	Water Level Order	Test Duration (hr)
A′	Deterioration to Failure	Uniform	Low - High	28.5
B′	Storm Ordering	Uniform	Low - High	8.5
C′	Storm Ordering	Uniform	High - Low	9.0
D′	Wave Period	Uniform	Low-High	8.5
E′	Wave Period	Uniform	Low-High	8.5
F′	Gradation	Riprap	Low-High	8.5
G′	Gradation	Riprap	Low-High	8.5

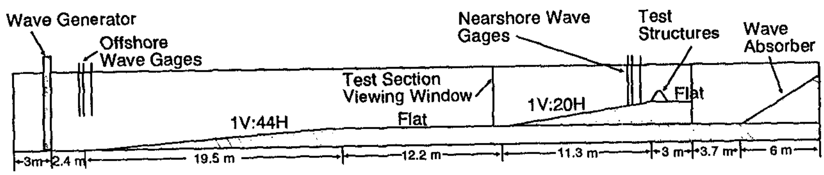

Figure 1. Flume Profile

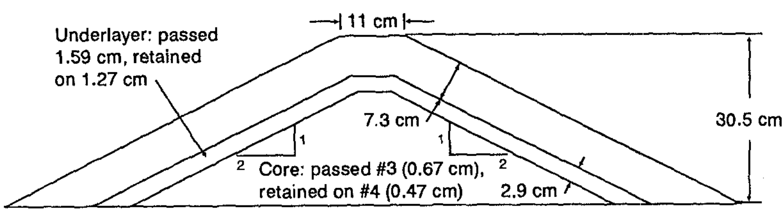

Figure 2. Model structure cross section

The experiment was conducted in a 61 m long by 1.5 m wide by 2 m deep flume, with a beach slope of 1V:20H. Two side-by-side identical conventional rubble mound cross sections were constructed with seaward slopes 1V:2H, crest heights 30.5 cm, and angular armor stone. Irregular waves corresponding to the TMA spectrum were run in bursts of 15 min. The undamaged underlayer and armor layer for the two identical structures were profiled before each series. Then both structures were profiled after each 30 min of irregular waves.

The seven series, summarized in Table 1, were designed to define spatial and temporal damage development under irregular depth limited breaking wave conditions. Series A′, lasting a total of 28.5 hr, was run until failure of the armor layer occurred, where failure was defined as exposure of the underlayer through a hole of diameter of at least D_{n50}. This series was intended to define the long term response of a structure. Series A′ was run once yielding 16 alongshore profiles every 30 min. Series B′, C′, D′, E′, F′, and G′, each lasting approximately 9 hr, were run twice producing 32 alongshore profiles per 30 min. These latter series were not run to failure but were intended to define the damage development for various conditions. Series B′ and C′ were designed to investigate storm sequencing by running low water first then high water in B′, and then reversing the water levels in C′. Series B′, D′, and E′ investigated period effects, each having a different peak period. Series F′ and G′ investigated stone gradation effects. The average damage $\bar{S}$ and the standard deviation of damage, σ_S, were computed using the 16 or 32 profiles after each 30 min of waves.

Two very different armor stone gradations were utilized. The armor stone for Series A′, B′, C′, D′, and E′ was uniformly sized with a median mass M_{50} = 128 g, nominal diameter $D_{n50} = (M_{50}/\rho_a)^{1/3}$ = 3.64 cm, stone density ρ_a = 2.66 g/cm^3, and D_{85}/D_{15} = 1.05, where D_{85} and D_{15} are the nominal diameters corresponding to 85 and 15 percent finer for the stone mass distribution, respectively. The armor stone for Series F′ and G′ was widely graded riprap with a median mass M_{50} = 256 g, nominal diameter $D_{n50} = (M_{50}/\rho_a)^{1/3}$ = 4.58 cm, stone density ρ_a = 2.66 g/cm^3, and D_{85}/D_{15} = 1.53. The riprap followed the widest recommendation of the SPM (1984) of approximately $0.125M_{50} < M < 4M_{50}$. For all series, the underlayer had a gradation of D_{85}/D_{15} = 1.32 and was sized such that $(M_{50})_{\text{armor}} / (M_{50})_{\text{filter}} = 25$ and $(D_{50})_{\text{armor}} / (D_{50})_{\text{filter}} = 2.9$.

Damage can be defined according to Broderick and Ahrens (1982) as

$$S = \frac{A_e}{(M_{50}/\rho_a)^{2/3}} = \frac{A_e}{D_{n50}^2} \qquad (1)$$

where A_e = eroded volume per unit length or cross-sectional eroded area. The eroded area was measured using a profiler composed of eight rods which spanned a width on one structure of 35 cm. The alongshore profiler rod spacing was 5 cm. The width of one structure was 0.76 m so the profiled section did not include the side wall effect. The profiler design was similar to that used by Davies et al. (1994). Each profile rod had a sphere of diameter 3.64 cm at the profiling end that followed the slope as the profiler was moved along the flume. The position of each sphere was determined from digital measurements of the angular rotation of each profile rod and translational position of the profile carriage. This technique provided an accurate and complete profile as the cross-shore spatial sampling interval was less than 1 mm. The eroded area, shown in Figure 3, was defined as the area between the undamaged profile and the damaged profile, but limited to the eroded region. The profile points were averaged over a small cross-shore spatial interval in order to eliminate contributions to the eroded area from minor downslope shifting of the armor layer.

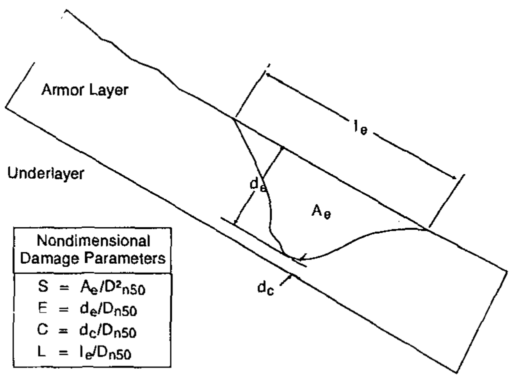

Figure 3. Sketch of breakwater profile with definition of damage parameters

As stated above, the eroded depth d_e, eroded length l_e, and cover depth d_c, shown in Figure 3, were used to define the profile shape. d_e was computed for each profile as the maximum distance between the eroded profile and the undamaged profile, measured normal to the structure slope. Similarly, d_c was computed as the minimum slope-normal difference between the undamaged underlayer slope and the damaged profile. Note that $d_e \neq (t_a - d_c)$ where t_a is the undamaged armor layer thickness, due to irregularities in the original armor layer thickness. The eroded length was defined as $l_e \approx 2A_e/d_e$ corresponding to a roughly triangular shaped region along slope. These three profile parameters were normalized by M&K, in order to generalize the test results, as $E = d_e/D_{n50}$, $C = d_c/D_{n50}$, and $L = l_e/D_{n50}$. The mean values were computed and denoted as $\bar{E}$, $\bar{C}$, and $\bar{L}$ while their standard deviations along the slope were σ_E, σ_C, σ_L. These statistical representations will be used throughout the remainder of this paper.

Incident wave statistics for all series are listed in Table 2. For all tests, the structure toe water depths were limited to h_t = 11.9 and 15.8 cm. The combination of wave periods and water depths produced low depth-to-wavelength ratios which resulted in severely breaking waves at the toe of the structure, which is typical of design conditions on most U.S. coastlines and represents the worst case for stability. In Table 2, T_p = spectral peak period, H_{mo} = spectral significant wave height defined as $H_{mo} = 4m_o^{1/2}$ with m_o = zero moment of the incident wave spectrum, $R = [(m_o)_r/m_o]^{1/2}$ = average reflection coefficient, with $(m_o)_r$ = zero moment of the reflected wave spectrum, T_m = mean wave period, H_s = average height of the highest 1/3 of waves, $H_{1/10}$ = average height of the highest 1/10 of waves, and $H_{2\%}$ = wave height exceeded by 2 percent of the waves in the wave height distribution. Time domain statistics T_m, H_s, $H_{1/10}$, and $H_{2\%}$ were all computed from a zero-upcrossing analysis.

Table 2
Summary of Incident Wave Characteristics

Series	Wave	Duration hr	ht cm	Tp sec	Hmo cm	R	Tm sec	Hs cm	H1/10 cm	H2% cm
A'	1	1.5	11.9	2.48	9.78	0.46	1.76	9.38	11.50	12.80
	2	1.5	11.9	2.48	12.40	0.47	1.69	11.60	13.80	15.37
	3	7.5	11.9	2.48	14.20	0.48	1.74	13.20	15.70	17.30
	4	1.0	15.8	2.59	10.50	0.52	1.73	10.10	12.72	14.30
	5	6.0	15.8	2.59	13.60	0.51	1.67	13.00	15.97	17.80
	6	11.0	15.8	2.59	15.80	0.51	1.66	14.90	18.00	19.30
B'	1	0.5	11.9	2.48	9.78	0.46	1.76	9.38	11.50	12.80
	2	2.0	11.9	2.48	12.40	0.47	1.69	11.60	13.80	15.37
	3	2.0	11.9	2.48	14.20	0.48	1.74	13.20	15.70	17.30
	5	2.0	15.8	2.59	13.60	0.51	1.67	13.00	15.97	17.80
	6	2.0	15.8	2.59	15.80	0.51	1.66	14.90	18.00	19.30
C'	4	1.0	15.8	2.59	10.50	0.52	1.73	10.10	12.72	14.30
	5	2.0	15.8	2.59	13.60	0.51	1.67	13.00	15.97	17.80
	6	2.0	15.8	2.59	15.80	0.51	1.66	14.90	18.00	19.30
	2	2.0	11.9	2.48	12.40	0.47	1.69	11.60	13.80	15.37
	3	2.0	11.9	2.48	14.20	0.48	1.74	13.20	15.70	17.30
D'	7	0.5	11.9	1.97	6.13	0.44	1.64	6.05	7.62	8.36
	8	2.0	11.9	1.97	9.88	0.38	1.54	9.88	12.48	13.59
	9	2.0	11.9	1.97	13.11	0.33	1.44	13.18	16.14	17.11
	10	2.0	15.8	2.02	9.62	0.38	1.61	9.74	12.48	14.00
	11	2.0	15.8	2.02	12.83	0.34	1.55	13.21	16.80	17.87
E'	12	0.5	11.9	1.53	5.05	0.38	1.29	5.05	6.72	7.75
	13	2.0	11.9	1.53	7.13	0.35	1.29	7.26	9.70	11.11
	14	2.0	11.9	1.53	9.93	0.31	1.23	10.19	13.38	14.90
	15	2.0	15.8	1.48	6.60	0.34	1.30	6.58	8.15	8.78
	16	2.0	15.8	1.48	9.41	0.32	1.26	9.53	12.03	13.34
F'	17	0.5	11.9	2.48	7.21	0.49	1.72	6.96	8.68	9.61
	18	2.0	11.9	2.48	11.68	0.42	1.56	11.51	14.39	15.63
	19	2.0	11.9	2.48	15.33	0.37	1.39	14.95	18.09	19.39
	20	2.0	15.8	2.59	6.43	0.47	1.80	6.18	7.78	8.58
	21	2.0	15.8	2.59	8.82	0.44	1.72	8.54	10.95	12.45
G'	22	0.5	11.9	1.97	7.62	0.42	1.50	7.53	9.53	10.62
	23	2.0	11.9	1.97	12.07	0.37	1.36	11.99	14.93	16.22
	24	2.0	11.9	1.97	15.42	0.35	1.30	15.21	17.95	18.99
	25	2.0	15.8	2.02	11.92	0.37	1.44	11.98	15.06	16.67
	26	2.0	15.8	2.02	15.34	0.35	1.34	15.36	18.64	20.03

3 PREDICTIVE EQUATIONS

M&K showed that the normalized damage was in the range $-2.7 \le S^* \le 3.0$, where $S^* = (S - \bar{S})/\sigma_S$. This relation can be used to predict the range of damage on the armor layer. For instance, the damage at failure of Series A′ was $\bar{S} = 13$ and $\sigma_S = 2.65$ yielding a range of $6 \le S \le 21$. This clearly shows that the variability of damage on this short section of structure was significant, especially considering that the waves were uniform alongshore. $E^* = (E - \bar{E})/\sigma_E$ and $C^* = (C - \bar{C})/\sigma_C$ were shown to have similar ranges with S^*, E^* and C^* all in the range from -3 to 3. The standard deviation of damage was shown to be a function of the mean damage, following the relation $\sigma_S = 0.5\bar{S}^{0.65}$. This relation indicates that the variability in damage increases with mean damage. Figure 4 shows σ_S for the four new series (D′, E′, F′, G′) plotted as a function of $\bar{S}$, where Series B′ is included for reference in this and the following figures. This figure indicates that the previously derived relation slightly underpredicts damage variability for the new series. The greater variability in damage for the wider gradation (Series F′ and G′) is expected; but the reason for the greater damage variability for shorter wave periods is not clear.

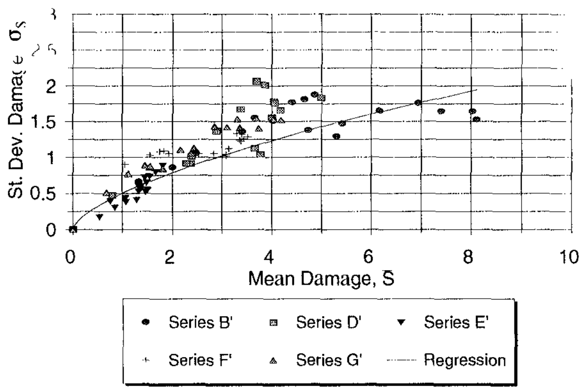

Figure 4. Standard deviation of damage as a function of the mean

M&K also showed that the number of variables could be reduced because the mean and standard deviation of the profile parameters were a function of the mean damage. The relation for the eroded depth was $\bar{E} = 0.44\bar{S}^{0.52}$ indicating that the shape of the eroded area remained geometrically similar during damage progression. Figure 5 shows $\bar{E}$ for the four new series as a function of $\bar{S}$. It is clear that the relation for $\bar{E}$ based on Series A′, B′, and C′ describes the new data well. Similarly M&K showed that the normalized eroded length followed $\bar{L} = 4.6\bar{S}^{0.48}$. This relation along with data from the four new series are shown in Figure 6. Again, the previously derived relation provides an excellent fit to the new data. Finally, the mean cover depth was shown to be described by the relation $(\bar{C}_0 - \bar{C}) = 0.1\bar{S}$, where the subscript 0 indicates the initial value at $\bar{S} = 0$. This relation also provides a good fit to the new data, as shown in Figure 7. In addition, the standard deviations for maximum eroded depth and remaining minimum cover depth were shown to be described by the relations $\sigma_E = [0.26 - 0.00007\,(\bar{S} - 7.8)^4]$ and $\sigma_C = [\sigma_{C_0} + 0.098 - 0.002\,(\bar{S} - 7)^2]$, respectively.

The agreement of these relationships for the four new series is similar to that shown in Figure 4. The above relations in this paragraph were developed using only profile data without regard to incident wave conditions or water levels.

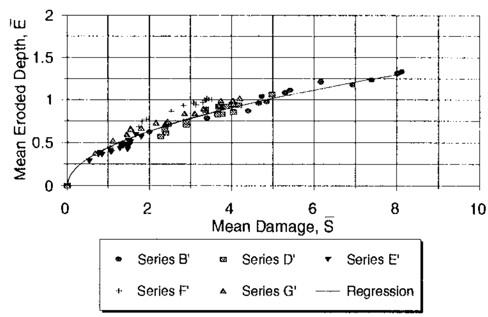

Figure 5. Mean normalized eroded depth as a function of mean damage

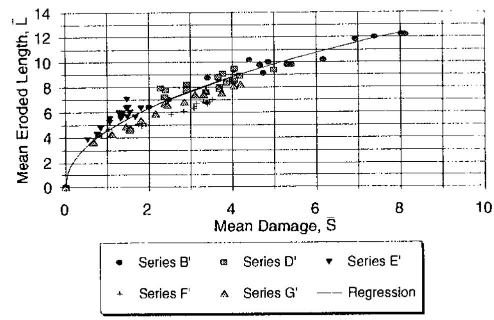

Figure 6. Mean normalized eroded length as a function of mean damage

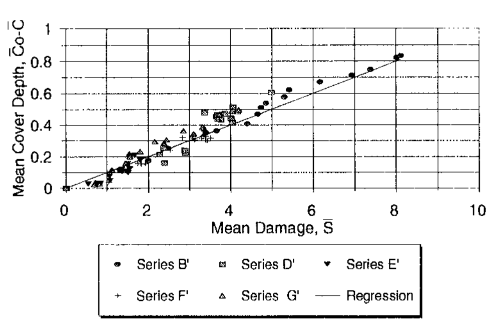

Figure 7. Mean normalized cover depth as a function of mean damage

The relations for the damage variables as a function of mean damage allow prediction of profile shape and alongshore variability of damage. A preliminary empirical equation was also proposed by M&K for predicting the temporal progression of mean eroded area as a function of time domain wave statistics as

$$\bar{S}(t) = \bar{S}(t_n) + a_s \frac{(N_s)_n^5}{(T_m)_n^b} (t^b - t_n^b) \qquad for \quad t_n \le t \le t_{n+1} \tag{2}$$

where $\bar{S}(t)$ and $\bar{S}(t_n)$ are predicted and known mean damages at times t and t_n, respectively, with $t > t_n$. $N_s = H_s / (\Delta D_{n50})$ is the stability number based on the average of the highest one-third wave heights from a zero-upcrossing analysis, $\Delta = S_r - 1$ where S_r is the specific gravity, T_m is the mean period, and a_s, and b are empirical constants. A similar equation relating mean damage to spectral wave characteristics was given as

$$\bar{S}(t) = \bar{S}(t_n) + a_p \frac{(N_{mo})_n^5}{(T_p)_n^b} (t^b - t_n^b) \qquad for \quad t_n \le t \le t_{n+1} \tag{3}$$

where a_p and b are again empirical coefficients and $N_{mo} = H_{mo} / (\Delta D_{n50})$. The empirical coefficients in (2) and (3) will be a function of structure slope, wave period, beach slope, structure permeability, and armor gradation. Figures 8 and 9 show (2) and (3) fitted to the profile data of Series A′, which is characterized as the mean damage from 16 profiles. M&K showed that the generalized formulas (2) and (3) with $a_s = 0.025$, $a_p = 0.022$ and $b = 0.25$, for wave conditions during multiple storm events, predicted the progression of damage quite well for the first three series in Tables 1 and 2. Figures 10 and 11 show (2) plotted along with data from Series B′ and C′. The fit of (3) for Series B′ and C′ with coefficients given above is similar to that shown in Figure 9 for Series A′. It is noted that the final damage was similar for both series consisting of different sequences of storms of similar cumulative wave action.

Figures 12 through 15 show (2) plotted against data from Series D′, E′, F′, and G′. Although (3) is not shown, the fits look very similar to those shown for (2). Series D′ and E′ were similar to Series B′ except that the peak wave period was changed for the three series. Series F′ and G′ were again similar except that the uniform armor was replaced with riprap. Two different peak wave periods were tested in Series F′ and G′. It can be seen that the damage progression equations predict overall damage reasonably well for Series D′ through G′ using $a_s = 0.025$, $a_p = 0.022$, and $b = 0.25$, although there are noted discrepancies. For example, it can be seen that damage initiation is underpredicted in all series. Equations (2) and (3) significantly underpredict damage initiation, if only 1 or 2 stones are displaced at the beginning of each test series. This underprediction appears to be produced by the variability in damage initiation. Thus, a second prediction curve has been added to Figures 12 through 15 which starts at the first measured damage point. As can be seen, the prediction is much better for this advanced damage. Figures 12 through 15 indicate that the empirical coefficients in (2) and (3) may vary somewhat with wave period and stone gradation for the range of experimental conditions described herein.

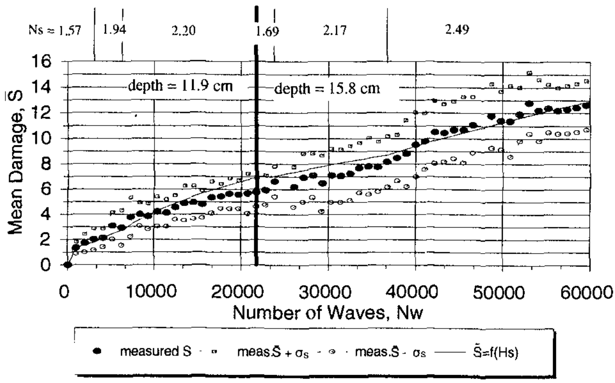

Figure 8. Measured damage and (2) as a function of number of waves for Series A′

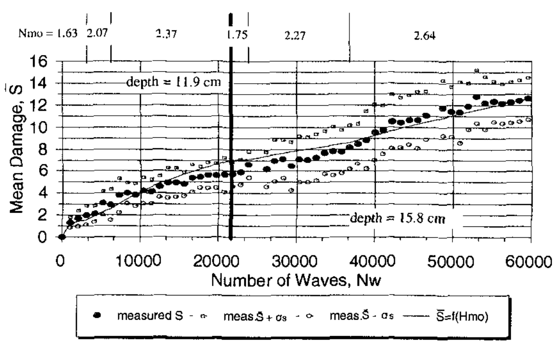

Figure 9. Measured damage and (3) as a function of number of waves for Series A′

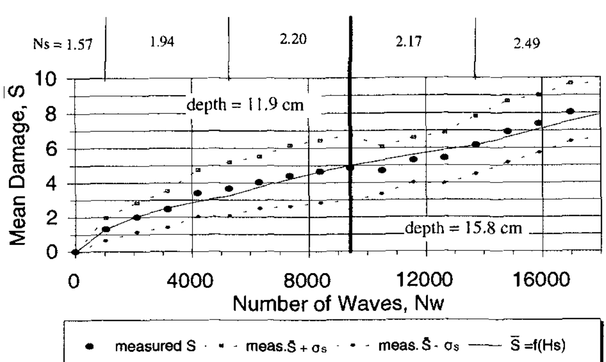

Figure 10. Measured damage and (2) as a function of number of waves for Series B′

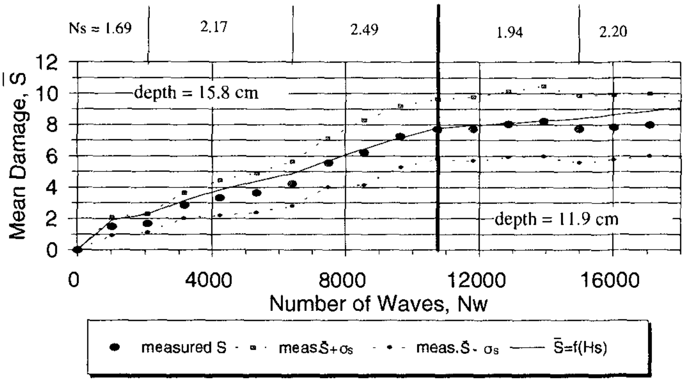

Figure 11. Measured damage and (2) as a function of number of waves for Series C′

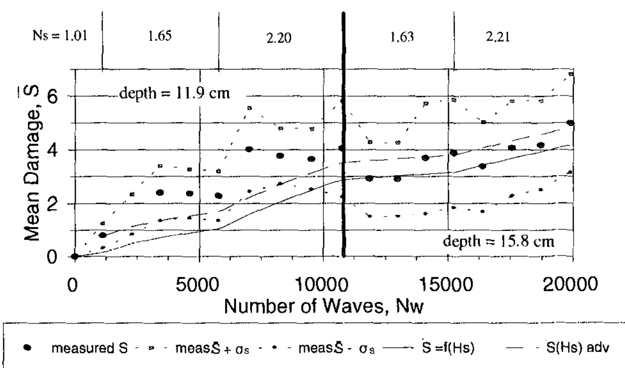

Figure 12. Measured damage and (2) as a function of number of waves for Series D′

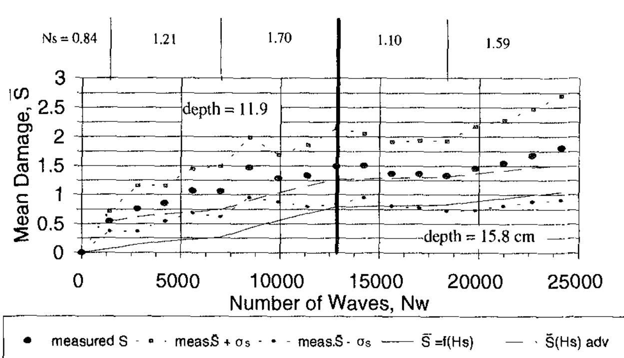

Figure 13. Measured damage and (2) as a function of number of waves for Series E′

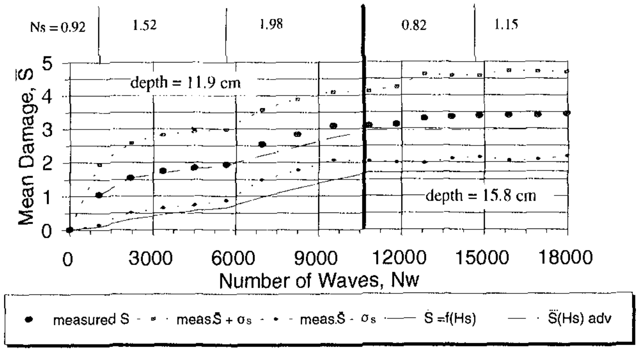

Figure 14. Measured damage and (2) as a function of number of waves for Series F′

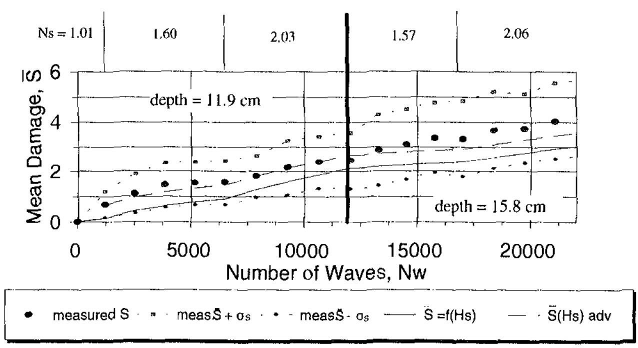

Figure 15. Measured damage and (2) as a function of number of waves for Series G′

4 CONCLUSIONS

An experiment is described consisting of seven relatively long-duration breakwater damage test series. The test series were conducted in a flume using irregular waves. New damage measurement techniques were developed and damage development data were acquired for breaking wave conditions. Wave height, wave period, water depth, storm duration, storm sequencing, and stone gradation were all varied systematically. The experiment yielded relationships for both temporal and spatial damage development.

It is shown that the damaged profile can be described by the eroded area A_e, maximum eroded depth d_e, minimum cover depth d_c, and maximum eroded length l_e. These parameters are normalized as $S = A_e/D^2_{n50}$, $E = d_e/D_{n50}$, $C = d_c/D_{n50}$, and $L = l_e/D_{n50}$ and the mean and standard deviation of each are shown to be a function of the mean damage $\bar{S}$. Relations are given for the standard deviation of S as a function of the mean as well as the mean and standard deviation of E, C, and L. Using these relations, the statistical variability of the profile and S can be quantified, which is a necessary step in a modern minimum cost analysis.

The relations by Melby and Kobayashi (1998a,b) for predicting temporal variations of mean damage with wave height and period varying with time in steps are shown to describe damage reasonably well (within one standard deviation) for four new test series, although damage initiation, where only 1 or 2 stones have moved, is consistently underpredicted by more than a standard deviation. This appears to be due to the variability in damage initiation. It is shown that the prediction is significantly improved if the damage progression is predicted immediately after the initial profile adjustment lasting 30 min. The initial profile adjustment may need to be accounted for in test series with relatively small cumulative damage. These relations are a reasonable first step in developing a damage prediction technique for predicting life-cycle costs and assessing maintenance needs. The empirical parameters in these equations may need to be varied somewhat with wave period and armor gradation. Employment of a critical stability number for measurable damage may be necessary to reduce the number of storms required in a life-cycle analysis.

ACKNOWLEDGEMENTS

This work was funded by the Coastal Structures Evaluation and Design Civil Works Research Program, Headquarters, U.S. Army Corps of Engineers. Permission was granted by the Office, Chief of Engineers, to publish this information. The first author gratefully acknowledges Mr. John Winkelman of the San Francisco, CA Corps District for developing the automated breakwater profiler and assisting with the initial stages of the experiment.

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