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Damage Development on Stone-Armored Breakwaters and Revetments

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PURPOSE: This Coastal and Hydraulics Engineering Technical Note (CHETN) provides a method to calculate damage progression on a rubble-mound breakwater, revetment, or jetty trunk armor layer. The methods apply to uniform-sized armor stone ($0.75W_{50} \leq W_{50} \leq 1.25W_{50}$, W_{50} = median weight of armor stone) as well as riprap ($0.125W_{50} \leq W_{50} \leq 4W_{50}$) exposed to depth-limited wave conditions.

The equations discussed herein are primarily intended to be used as part of a life-cycle analysis, to predict the damage for a series of storms throughout the lifetime of the structure. This life-cycle analysis including damage prediction allows engineers to balance initial cost with expected maintenance costs in order to reduce the overall cost of the structure. The equations are intended to provide a tool for accurate damage estimates in order to reduce the possibility of unexpected maintenance costs.

INTRODUCTION: Rubble-mound breakwater, revetment, and jetty projects require accurate damage prediction as part of life-cycle analyses. However, few studies have been conducted to determine damage progression on stone-armor layers for variable wave conditions over the life of a structure. Previous armor stability lab studies were intended to determine damage for the peak of a design storm. As such, most previous laboratory studies were begun with an undamaged structure and damage measured for a single design wave condition (Hudson 1959; Van der Meer 1988). The empirical equations derived from these studies were valid for determining initial damage but not for damage progression through several storm events. Damage actually occurs as a result of a sequence of storms of varying severity and with varying water levels. This CHETN provides equations that allow the prediction of rubble-mound deterioration with time. These relations are supplemented by predictive equations for the uncertainty or variability of damage for more accurate estimation of reliability or, conversely, probability of failure.

Within this CHETN, damage is defined in terms of the average normalized cross-sectional eroded area of armor on the slope. Damage is defined up to the point that the underlayer is exposed through a hole the size of a nominal armor stone diameter $D_{n50} = (M_{50}/\rho_a)^{1/3}$, where M_{50} is the median mass of armor stone and ρ_a is the armor stone density. The condition where the underlayer is exposed defines failure of the armor layer because rapid destruction of the structure often occurs after this point. The damaged profile is described in terms of the engineering parameters maximum eroded depth, minimum remaining cover depth, and maximum cross-shore length of the eroded region. Relations for these profile descriptors are given in terms of the mean damage. Further, relations describing the alongshore variability of damage and the profile descriptors are provided to support reliability or uncertainty analyses. These relations apply for single storms and for storm sequences given depth-limited normally incident waves. The

relations and supporting studies are described in a series of publications on damage (Melby 1999; Melby and Kobayashi 1998a, 1998b, 1999).

DAMAGE DESCRIPTION: The *Shore protection manual* (1984) provides damage as a function of the marginal wave height exceeding the zero damage wave height. The *Shore protection manual* damage $D\%$ was defined as the normalized eroded volume in the active region, extending from the middle of the breakwater crest down to one wave height below the still-water level. This design information is based on laboratory tests limited to monochromatic nonbreaking waves impinging on long structure slopes. The background reports provide little insight and no data.

Broderick and Ahrens (1982) provided a definition of damage that was not a function of cross-sectional geometry. They defined damage to an armor layer by the normalized eroded cross-sectional area

$$S = \frac{A_e}{D_{n50}^2} \quad (1)$$

where

A_e = measured eroded cross-sectional area (Figures 1 and 2)

D_{n50} = nominal armor stone diameter

This damage formulation was popularized by Van der Meer (1988). Melby and Kobayashi (1998a) utilized S as a general damage description and further defined the eroded profile using several engineering parameters: the maximum eroded depth $E = d_e/D_{n50}$, the minimum remaining cover depth $C = d_c/D_{n50}$, and the maximum cross-shore length of the eroded region $L = l_e/D_{n50}$. These damaged section descriptors are shown in Figure 1. A typical structure cross section is shown in Figure 2. The maximum eroded depth and minimum remaining cover depth may not occur at the same location along a structure because of the variability of the armor layer thickness.

PREDICTIVE RELATIONS: Melby and Kobayashi (1998a) conducted a series of experiments measuring the erosion of a stone-armor layer for varying wave and water-level conditions. The structure profile was measured repeatedly throughout the test at up to 32 sections alongshore. The 32 profiles were used to obtain mean damage and mean damaged profile as well as the alongshore variability of damage and the profile.

The empirical equation proposed by Melby and Kobayashi (1998a) for predicting the temporal progression of mean eroded area as a function of time domain wave statistics is:

$$\bar{S}(t) = \bar{S}(t_n) + 0.025 \frac{(N_s)_n^5}{(T_m)_n^{0.25}} (t^{0.25} - t_n^{0.25}) \quad \text{for } t_n \leq t \leq t_{n+1} \quad (2)$$

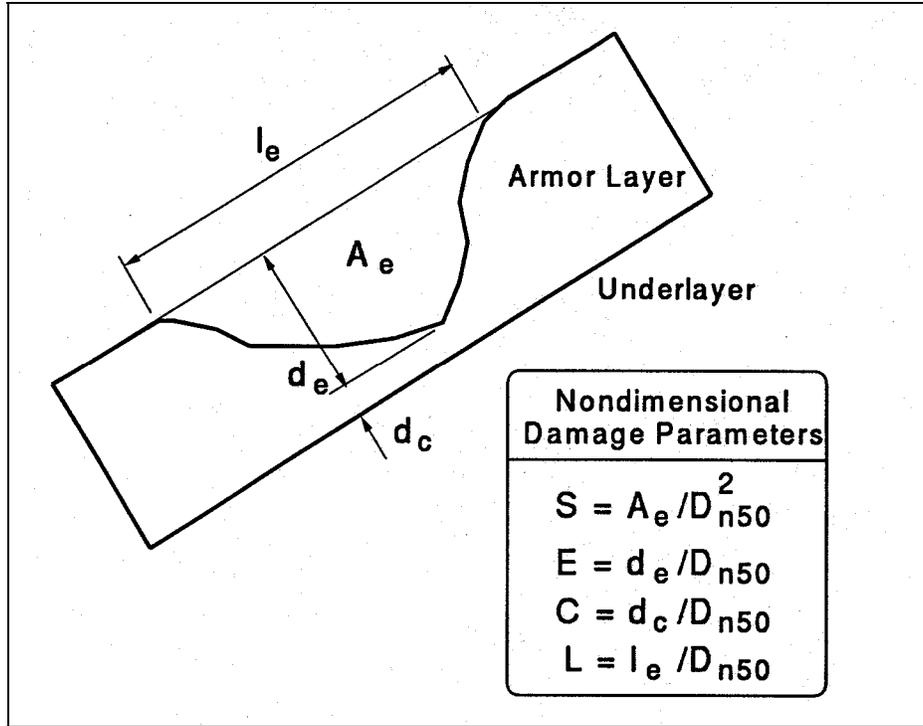


Figure 1. Damaged section parameters

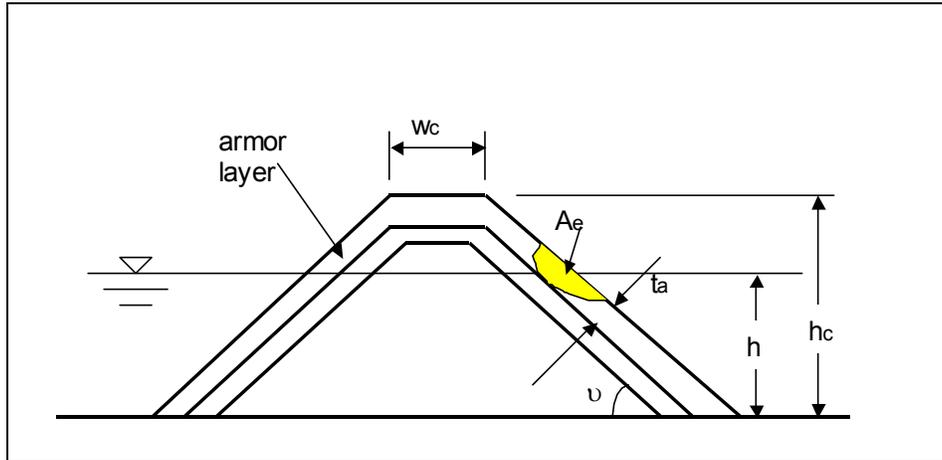


Figure 2. Rubble-mound structure cross section

where $\bar{S}(t)$ and $\bar{S}(t_n)$ are predicted and known mean eroded areas 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 armor stone specific gravity, and T_m is the mean period. The wave parameters are defined $5H_s$ seaward of the structure toe, which is the travel distance of large breaking waves. Equation 2 provides a means to compute damage over a sequence of N events, each of constant wave conditions, where each event is defined over a time period from t_n to t_{n+1} , $1 \leq n \leq N$.

A similar equation relating mean damage to spectral wave characteristics was given by Melby and Kobayashi as:

$$\bar{S}(t) = \bar{S}(t_n) + 0.022 \frac{(N_{mo})_n^5}{(T_p)_n^{0.25}} (t^{0.25} - t_n^{0.25}) \quad \text{for } t_n \leq t \leq t_{n+1} \quad (3)$$

where $N_{mo} = H_{mo} / (\Delta D_{n50})$, $H_{mo} = 4(m_o)^{1/2}$, m_o is the zero moment of the incident wave spectrum, and T_p is the spectral peak period. The empirical coefficients in Equations 2 and 3 will be primarily a function of structure slope, wave period, beach slope, and structure permeability. These equations have been verified for the following range of laboratory conditions:

Structure slope, $\tan \alpha$: 1V:2H

Significant wave height, H_s : 5.05 cm – 15.80 cm

Toe depth, h : 11.9 – 15.8 cm

Mean wave period, T_m : 1.23 sec – 1.80 sec

Iribarren parameter: $\tan \alpha / (H_s / L_{om})^{0.5}$: 2.08 – 4.17

Beach slope: 1V:20H

Structure crest height, h_c : 30.5 cm

Stone density, ρ_a : 2.66 g/cm³

Armor stone gradations: $0.75M_{50} < M_{50} < 1.25M_{50}$, $D_{85}/D_{15} = 1.25$ and $0.125M_{50} < M_{50} < 4M_{50}$, $D_{85}/D_{15} = 1.53$

Filter layer: $(M_{50})_{armor} / (M_{50})_{filter} = 25$, $(D_{n50})_{armor} / (D_{n50})_{filter} = 2.9$

The deepwater wavelength computed from the mean period is $L_{om} = gT_m^2 / 2\pi$, where g is the acceleration of gravity.

Equations 2 and 3 are plotted in Figure 3, where $\bar{S}(t)$ is plotted as a function of number of waves, N_w . Here $N_w = t/T_m$. In Figure 3, the measured mean eroded area plus and minus one standard deviation are plotted. These data are from one long series of six different storms with two water levels. Equations 2 and 3 should be conservative for most applications because they are based on severely breaking waves, a relatively steep beach slope, and a relatively impermeable core. Deviations from the range of tested conditions are likely to produce different empirical coefficients in Equations 2 and 3. Caution should be exercised in applications of these equations outside of the range of conditions tested.

The mean parameters \bar{S} , \bar{E} , \bar{C} , and \bar{L} and the standard deviations σ_S , σ_E , σ_C , and σ_L were used to describe the tendencies, variabilities, and ranges of damage and the damaged profile. All measured values from all measured series were in the following ranges:

$$\text{Damage: } -2.7 < (S - \bar{S}) / \sigma_S < 3 \quad (4)$$

$$\text{Eroded depth: } -2.7 < (E - \bar{E}) / \sigma_E < 2.7 \quad (5)$$

$$\text{Cover depth: } -2.7 < (C - \bar{C}) / \sigma_C < 2.8 \quad (6)$$

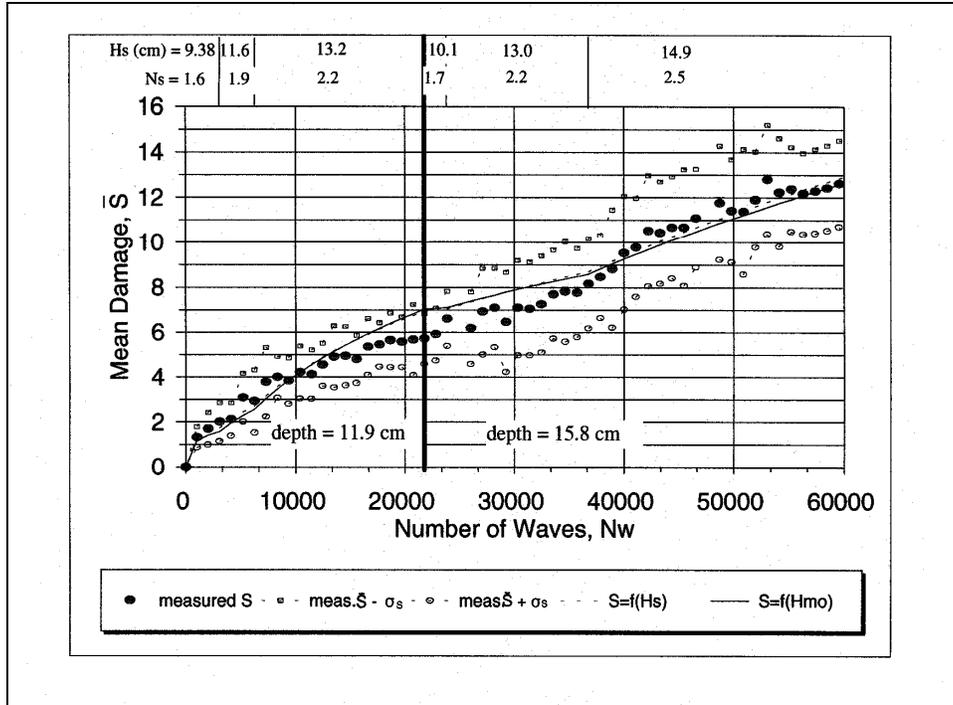


Figure 3. Mean damage as a function of number of waves at mean period

These ranges allow the lower and upper limits of the damaged profile descriptors to be estimated. In order to reduce the number of parameters for design, Melby and Kobayashi (1998a) expressed the key profile parameters as a function of the mean damage as follows:

$$\sigma_S = 0.5 \bar{S}^{0.65} \quad (7)$$

$$\bar{E} = 0.46 \bar{S}^{0.5} \quad (8)$$

$$\bar{C} = C_o - 0.1 \bar{S} \quad (9)$$

$$\bar{L} = 0.44 \bar{S}^{0.5} \quad (10)$$

Using Equations 4 and 7 with $\bar{S} = 13$, corresponding to localized failure in one series, $\sigma_S = 2.65$ and $6 < S < 21$. This illustrates the large alongshore variability of damage at failure for along-shore uniform waves.

EXTENSION OF PREVIOUS MODEL: A modification to Equation 2 was introduced by Melby and Kobayashi (1999) based on the hypothesis of equivalency. The revised equation allows for non-zero initial damage values. The modified equation is:

$$\bar{S}(t) = 0.011N_s^5(N_e + \delta N)^{0.25}$$

$$N_e = \left(\frac{\bar{S}(t_n)}{0.011N_s^5} \right)^4 \quad (11)$$

$$\delta N = (t - t_n) / T_m \quad \text{for } t_n \leq t \leq t_{n+1}$$

This equation is similar to Equation 2, but predicted damage is not dependent on the time that the simulation begins. Therefore, future damage can be predicted for an existing structure whose history is unknown. This equation would be appropriate for a major rehabilitation study.

EXAMPLE: A single storm example is provided to show how parameters are used in the equations.

A traditional rubble-mound breakwater is to be constructed in seawater so that the longitudinal axis of the structure is parallel to the wave crests. The significant wave height used for design is determined to be depth limited from a shoaling/refraction/diffraction study. The design wave height is calculated at a distance of $5H_s$ seaward from the structure toe.

Given design values:

- Significant wave height: $H_s = 2.07$ m (6.8 ft)
- Specific gravity of seawater: $S = 1.0256$
- Specific gravity of armor stone: $S_r = 2.65$
- Specific weight of armor stone: $\gamma_r = 2.66 \times 10^4$ N/m³ (169.6 lb/ft³)
- Density of armor stone: $\rho_r = 2.72$ t/m³ (5.27 slug/ft³)
- Mean wave period: $T_m = 10.8$ sec
- Structure seaward slope: $\tan \alpha = 0.5$
- Median stone size: $W_{50} = 1,311$ kg (2,889 lb)
- Stone gradation: $0.75W_{50} < W_{50} < 1.25W_{50}$
- Storm duration: 4 hr = 14,400 sec (1,333 waves at T_m)
- Armor layer thickness: $2D_{n50}$

Calculations:

$$D_{n50} = \left(\frac{W_{50}}{\gamma_r} \right)^{1/3} = \left(\frac{M_{50}}{\rho_r} \right)^{1/3} = \left(\frac{1.311 \text{ t}}{2.72 \text{ t/m}^3} \right)^{1/3} = 0.784 \text{ m}$$

$$N_s = \frac{H_s}{\Delta D_{n50}} = \frac{2.07 \text{ m}}{(2.65 - 1)0.784 \text{ m}} = 1.60$$

Damage due to a single storm would be computed using Equation 2 as

$$\begin{aligned}\bar{S}(t) &= \bar{S}(t_n) + 0.025 \frac{(N_s)_n^5}{(T_m)_n^{0.25}} (t^{0.25} - t_n^{0.25}) \\ &= 0 + 0.025 \frac{1.60^5}{(10.8\text{s})^{0.25}} (14,400\text{s})^{0.25} = 1.58\end{aligned}$$

This damage level indicates that, for the single 4-hr storm, there would be an eroded area of about 15 to 16 median-sized stones removed from a typical cross section. This is minimal damage and would not affect the integrity of the structure. Assuming uniform wave height alongshore, the alongshore variability, given as the standard deviation, of damage would be given by Equation 7.

$$\sigma_S = 0.5 \bar{S}^{0.65} = 0.5(1.58)^{0.65} = 0.67$$

And the alongshore range of damage would be given by Equation 4 as

$$-2.7(3.0) + 15.8 \leq S \leq 3.0(3.0) + 1.58 \quad S \geq 0$$

$$0 \leq S \leq 3.61$$

So the range of damage along the shore is quite large but still represents only minimal damage to the structure.

The damaged armor layer profile shape can be analyzed in a similar fashion. The initial layer thickness is

$$t_r = 2D_{n50} = 2(0.784 \text{ m}) = 1.57 \text{ m}$$

Then the means of the maximum eroded depth, minimum remaining cover depth, and maximum eroded cross-shore length, respectively, would be given by Equations 8, 9, and 10 as follows.

$$\bar{E} = 0.46 \bar{S}^{0.5} = 0.46(1.58)^{0.5} = 0.58$$

$$\bar{C} = C_o - 0.1 \bar{S} = 1.57 - 0.1(1.58) = 1.41$$

$$\bar{L} = 0.44 \bar{S}^{0.5} = 4.4(1.58)^{0.5} = 5.5$$

The mean dimensional eroded depth is $d_e = \bar{E} D_{n50} = 0.58 (0.784 \text{ m}) = 0.45 \text{ m}$. The mean dimensional minimum remaining cover depth is $d_c = \bar{C} D_{n50} = 1.40 (0.784 \text{ m}) = 1.11 \text{ m}$, so there is significant cover over the underlayer. Note that the initial thickness of the armor layer is not uniform; so the maximum eroded depth may not occur in the same location as the location of

minimum remaining cover. In addition, the sum of \bar{E} and \bar{C} may not be equal to \bar{C}_o for the same reason. The mean of the maximum eroded length or cross-shore eroded hole is $l_e = 4.34$ m. This stone size was very well suited for this application if this was a design level storm.

If a storm of $N_w = 5,000$ waves were to strike the undamaged structure, then $t = 5,000$ (10.8 s) = 15 hr. Then the mean damage would increase to

$$\bar{S}(t) = 0 + 0.025 \frac{1.60^5}{(10.8 \text{ s})^{0.25}} (54,400 \text{ s})^{0.25} = 2.20$$

This damage level is still quite acceptable for most situations.

CONCLUSIONS: This CHETN provides a method for computing damage on a rubble-mound breakwater, revetment, or jetty trunk section stone-armor layer that is exposed to depth-limited breaking waves. The methods are useful in determining the expected life of new or damaged structures and in developing life-cycle analyses. The empirical equations should be conservative for most applications, but care should be exercised when applying outside of the tested conditions, as described in this CHETN.

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