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Hydraulic Stability Analysis for Proposed Breakwater for Villa Marina Yacht Harbor, Fajardo

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I. Introduction

This report contains a hydraulic stability analysis for the proposed breakwater for Villa Marina Yacht Harbor. The main purpose of the analysis is to determine the size and weight of armor stone that must be used in order to ensure the structure's hydraulic stability in the worst case scenario: a direct hit by a hurricane.

In Section II, the basic principles behind hydraulic stability and its analysis are discussed. In Sections III and IV, the damage and permeability parameters are defined. In Section V the appropriate stability formula is derived, and in Section VI the values for all the necessary parameters are estimated. In addition, a median armor stone diameter and weight for the outer layer of the structure are calculated. Finally, in Section VII recommendations are made as to the cross section of the structure.

II. Hydraulic Stability

The two principal design criteria that coastal structures must meet are: achieving the desired hydraulic response from the structure, and ensuring the hydraulic stability of the structure.

The hydraulic response of the structure involves the effects of the structure on the existing wave and current fields. These considerations are often addressed by means of refraction/diffraction analyses, bathymetric analyses and current studies, among others. In general, the designer of the structure will attempt to maximize the structure's effectiveness in terms of the structure's intended purpose. At the same time, the designer will attempt to minimize the structure's effects on any other aspect of the oceanographic conditions in the structure's vicinity.

The hydraulic stability of the structure involves ensuring the integrity of the structure itself, in terms of its ability to withstand the loads to which it will be subjected. Wave forces acting on the structure can cause armor movement, or even armor breakage. A movement or displacement of the armor layer is called a hydraulic instability, and is the subject of this report. The breakage of armor units is called structural instability and is not discussed in this report.

There are various ways that a structure can be hydraulically unstable. These include rocking of individual armor units, displacement of units out of the armor layer, the sliding of a blanket of armor units and settlement due to the compaction of the armor layer. The typical armor failure modes are shown in Figure 1.

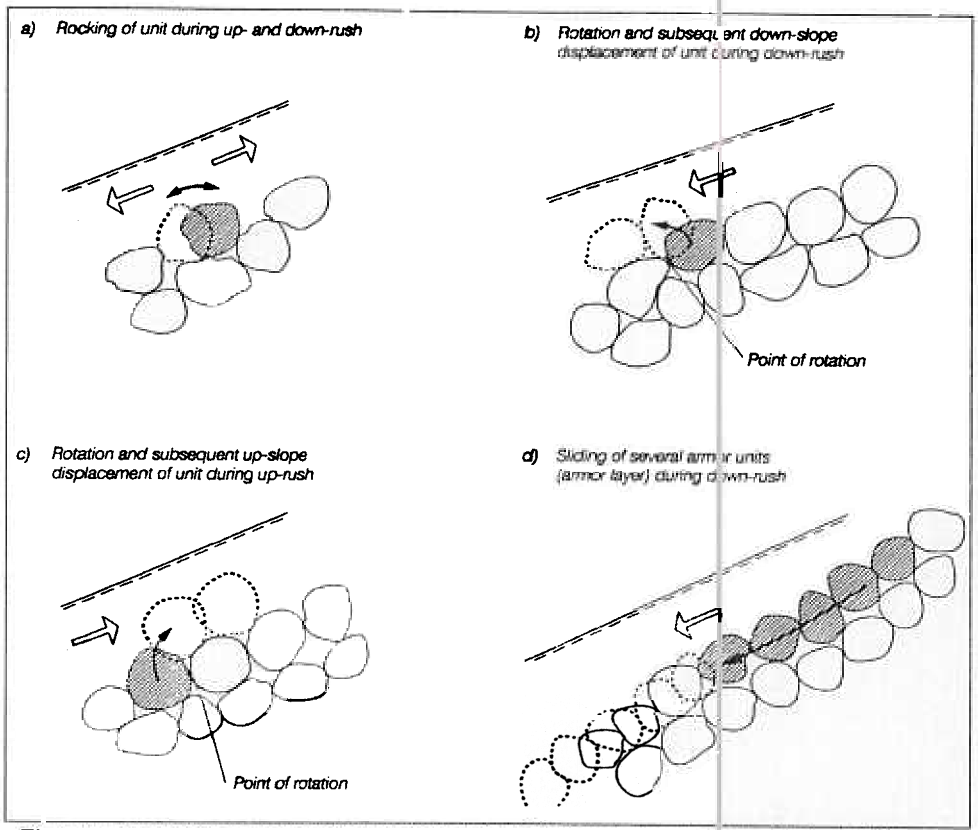

Figure 1. Examples of typical failure modes of armor layers due to hydraulic instability.

From the Coastal Engineering Manual, (Vol. VI, Part V, Page 48):

"The complicated flow of waves impacting armor layers makes it impossible to calculate the flow forces acting on armor units. Moreover, the complex shape of units together with their random placement makes calculation of the reaction forces between adjacent armor units impossible. Consequently, deterministic calculations of the instantaneous armor unit stability conditions cannot be performed."

For this reason, stability formulae are based on hydraulic model tests. These hydraulic model tests allow the parameters of the incident waves to be directly related to the response of armor units in terms of movements, while ignoring the details of the actual forces. In effect, the forces are being treated as a "black box" transfer function.

Some qualitative considerations of the involved forces are however used to develop a stability criterion. The accepted methodology involves expressing the wave-generated flow forces on armor units by a Morison equation between destabilizing lift and drag forces (F_L and F_D), and stabilizing inertial and gravitational forces (F_I and F_G). It is then assumed that at the point of instability the lift and drag forces dominate the inertial forces, and one obtains a stability criterion expressed as the ratio of the destabilizing forces to the stabilizing forces (lift plus drag divided by gravity):

$$\frac{F_D + F_L}{F_G} \approx \frac{\rho_w D_n^2 v^2}{g(\rho_s - \rho_w) D_n^3} = \frac{v^2}{g \Delta D_n}, \qquad (1)$$

where ρ_s and ρ_w are the densities of the armor stone and sea water, respectively, v is a characteristic flow velocity, and $D_n = (\text{armor unit volume})^{1/3}$ is the equivalent cube length.

Using $v \approx (gH)^{1/2}$ for the flow velocity of a breaking wave of height H, one arrives at the following hydraulic stability parameter, N_S:

$$N_s = \frac{H}{} \qquad (2)$$

where $\Delta = \left(\rho_s / \rho_w - 1 \right)$. The hydraulic instability of the structure (or the degree of damage it will suffer under stress) can then be expressed in the general form:

$$N_s = \frac{H}{\Delta D} \le K_1^a K_2^b K_3^c \cdots \qquad (3)$$

where the factors on the right hand side depend on parameters influencing stability other than H, Δ and D_n.

Among the sea state parameters to affect hydraulic stability we find:

- Characteristic wave height
- Characteristic wave length
- Characteristic wave steepness
- Wave asymmetry
- Shape of wave spectrum
- Wave grouping
- Water depth

- Wave incident angle
- Number of waves
- Mass density of water

Among the structural parameters affecting hydraulic stability we find:

- Seaward profile of structure (including slope angle, freeboard, etc.)
- Mass density of armor units
- Grading of rock armor
- Mass and shape of armor units
- Packing density, placement pattern and layer thickness of main armor
- Porosity and permeability of underlayers, filter and core.

The right hand side of Equation 3 has been explored using simple geometrical considerations of the balances of forces acting on armor stone. Some versions of these are:

$$\frac{H}{\Delta N_n} = K \cos \alpha \qquad \text{Svee (1962)}$$

$$\frac{H}{\Delta N_n} = (K \cot \alpha)^{1/3} \qquad \text{Hudson (1958,1959)}$$

The coefficient K includes some level of damage as well as all other influencing parameters not explicitly included in the formulae.

The fact stability formulae do not explicitly contain all the above mentioned parameters, combined with the stochastic nature of wave load and armor response, introduces some uncertainty into all stability formula. This uncertainty is generally treated in the form of a Gaussian distributed stochastic variable with a specified mean and standard deviation.

III. Damage Classification

Damage to armor layers is related to a specific sea state of specified duration. The damage level is characterized by counting the number of displaced units of by measurement of the eroded surface profile of the armor slope.

Broderick (1983) defined a dimensionless damage parameter for rip rap and stone armor given as:

$$S = \frac{A_e}{D_{n50}^2} \qquad (4)$$

which is independent of the length of the slope and takes into account vertical settlements but not settlements and sliding parallel to the slope

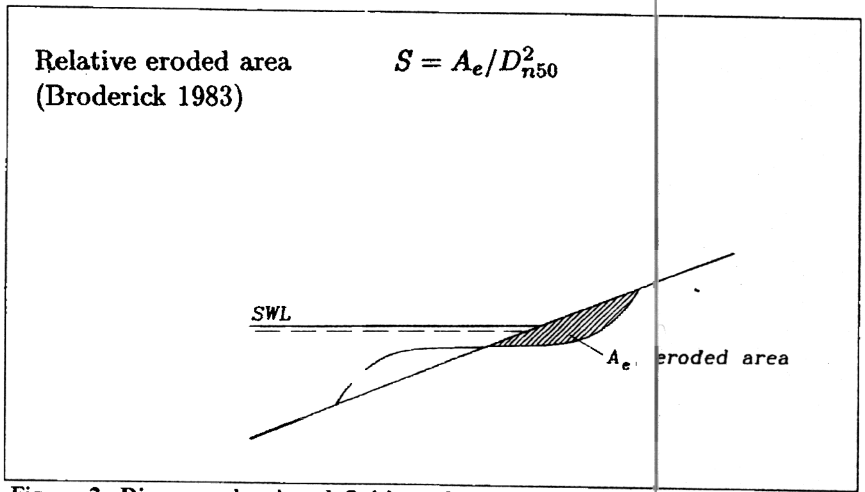

Figure 2. Diagram showing definition of the eroded area used in the damage parameter *S*.

S can be interpreted as the number of squares with side length D_{n50}, which fit into the eroded area, or, as the number of cubes with a side length D_{n50} eroded within a strip of width D_{n50} of the armor layer. Table 1 relates values of S to damage levels:

Unit	Slope	Initial Damage	Intermediate Damage	Failure
Rock	1:1.5	2	3 - 5	8
Rock	1:2	2	4 - 6	8
Rock	1:3	2	6 - 9	12
Rock	1:4 – 1:6	3	8 - 12	17

Table 1. Damage Parameter *S* values for various slope s of the seaward side of the structure.

IV. Permeability

The permeability P of the structure has a damping effect on the wave energy in terms of damage, since some of the incoming wave energy is absorbed into the structure itself. If the structure were completely impermeable, all the wave [illegible] would be concentrated on the structure's outer armor layer, increasing the damage. Figure 3 shows values of the notational permeability for various structure configurations.

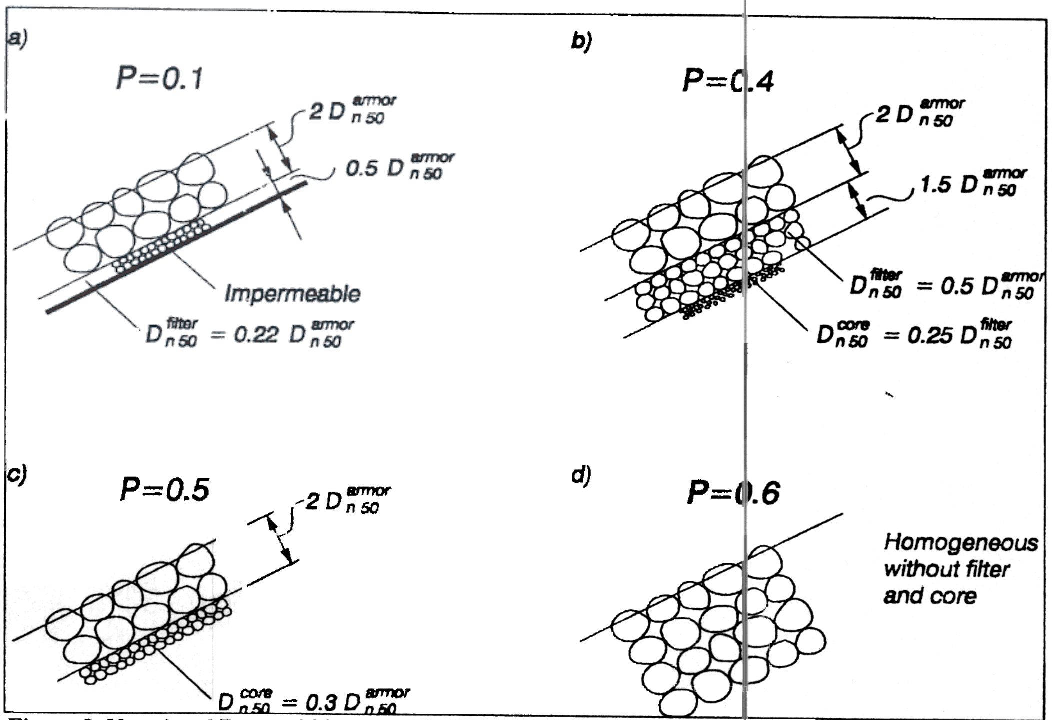

Figure 3. Notational Permeability Coefficients (van der Meer 1988).

V. Stability Formulae

Again from the Coastal Engineering Manual, (Vol. VI, Part V, Page 63):

Formulae for hydraulic stability of armor layers are almost exclusively based on small-scale model tests. Large-scale model tests for verification of small-scale model test results have been performed in few cases. Adjustment of formulae due to prototype experience seems not to be reported in the literature...Generally, small-scale hydraulic tests of armor layer stability are assumed to be conservative, if any bias is present.

Some of the factors by which armor layer stability formulae can be classified are:

- Type of armor unit
- Deep or shallow water wave conditions
- Armor layers crest level relative to wave runup and swl
- Structures with and without superstructure

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Type of armor unit distinguishes between rock armor, for which shape and grading must be defined, and uni-size concrete armor units.

Deep-water conditions correspond to Rayleigh distributed wave heights at the structure, i.e., depth-limited wave breaking does not take place. Shallow-water conditions correspond to non-Rayleigh distributed wave heights at the structure, i.e., depth limitations cause wave breaking in front of, or in the worst case, directly upon the structure.

It is common to distinguish between different types of overtopping because of its effect on hydraulic stability. When the crest is lower than the runup level, wave energy can pass over the structure. Thus, the size of the front slope armor can be reduced while the size of the crest and rear slope armor must be increased compared to non-overtopped structures.

For two-layer armored, non-overtopped slopes, the following relationship has been found (van der Meer 1988):

$$\frac{H_s}{\Delta D_{n50}} = S^{0.2} P^{-0.13} N_z^{-0.1} (\cot\alpha)^{0.5} \xi_m^P \qquad (5)$$

with

$$\xi_m = s_m^{-0.5} \tan\alpha , \qquad (6)$$

where:

H_s is the significant wave height in front of the breakwater,
D_{n50} is the equivalent cube length of median rock,
ρ_s is the mass density of rocks,
ρ_w is the mass density of water,
S is the relative eroded area (see discussion above
P is the notational permeability (see discussion above),
N_z is the number of waves attacking the structure in a given event, $(N_z = t/T)$,
t is the duration of the impacting event,
α is the structure's slope angle,
s_m is the wave steepness, given by $s_m = H_s / L_{om}$,
L_{om} is the deep-water wavelength corresponding to the mean period,

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Δ is defined as $\left(\rho_s / \rho_w \right) - 1$.

The relation between the period and the wavelength for water waves, known as the dispersion relation, is given by:

$$\omega^2 = gk \tanh kh$$

where:

h is the water depth,
$\omega = 2\pi / T$ is the wave angular frequency,
$k = 2\pi / L$ is the wave number, and
$g = 9.8 \, m/s^2$ is the acceleration due to gravity.

In the deep-water approximation, $L >> h$, Equation 7 becomes:

$$L_{om} = 1.56 T_{om}^2$$

where T_{om} is the mean deep-water wave period.

Combining Equations 6 and 8, we get:

$$\xi_m = (1.56)^{1/2} H^{-1/2} T \tan \alpha$$

Solving Equation 5 for D_{n50} yields:

$$D_{n50} = H_s S^{-0.2} P^{0.13} N_z^{0.1} (\cot \alpha)^{-0.5} \xi_m^{-P} \Delta^{-1}$$

Combining Equations 9 and 10, we arrive at:

$$D_{n50} = H_s^{1.0+0.5P} T^{-P} S^{-0.2} N_z^{0.1} \Delta^{-1} P^{0.13} (1.56)^{-0.5P} (\tan \alpha)^{0.5-P}$$

and using $(N_z = t / T)$ we find:

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$$D_{n50} = H_s^{1.0+0.5P} T^{-(.1+P)} t^{0.1} S^{-0.2} \Delta^{-1} P^{0.13} (1.56)^{-0.5P} (\tan\alpha)^{0.5-P}$$

Equation 12 gives an expression for the median stone size in terms of the significant wave height H_s, wave period T, event duration t, relative eroded area S, notional permeability P, and slope angle α. The values used for each one of these parameters will be discussed next.

VI. Parameter Values

We will now discuss the values chosen for the parameters in Equation 12.

The slope angle α is assumed to be 33°, which corresponds to a vertical to horizontal ratio (V:H) of 1:1.5. The structure should not be constructed any steeper than this.

The value used for the permeability is $P = 0.5$. This corresponds to a two-layer structure, with the underlayer median diameter roughly one third that of the outer layer (See Figure 3).

The damage level chosen was $S = 4$, corresponding to intermediate damage (See Table 1.)

The time duration of the event chosen was $t = 8$, corresponding to a storm event (such as a hurricane) lasting for a period of eight (8) hours.

The wave period chosen to represent hurricane-induced waves was $T = 13$, corresponding to thirteen (13) second waves. (See *Refraction/Diffraction Analysis for Villa Marina, Fajardo (2003), Alfredo Torruella, Ph.D., Caribbean Oceanography Group* for details).

Finally, the wave height H is assumed to be depth limited. This is a reasonable assumption, since we are simulating damage done by a large hurricane, which can generate wave heights of up to 15 meters or more. This wave height clearly exceeds the theoretical breaking wave height (H_b) limit, given by:

$$H_b = \gamma_b d_b,$$

where the breaker depth index $0.78 \le \gamma_b \le 1.56$ (depending on the slope) Weggel (1972), and d_b is the depth at the breakwater. Using four (4.0) meters as the depth at the breakwater plus assuming a storm surge of two (2.0) meters (so that $d_b = 6.0$

meters), and using $\gamma_b = .78$, obtained by assuming a small slope in Weggel's (1972) expression for the breaker depth index, we find: $H_b = 4.68$ meters.

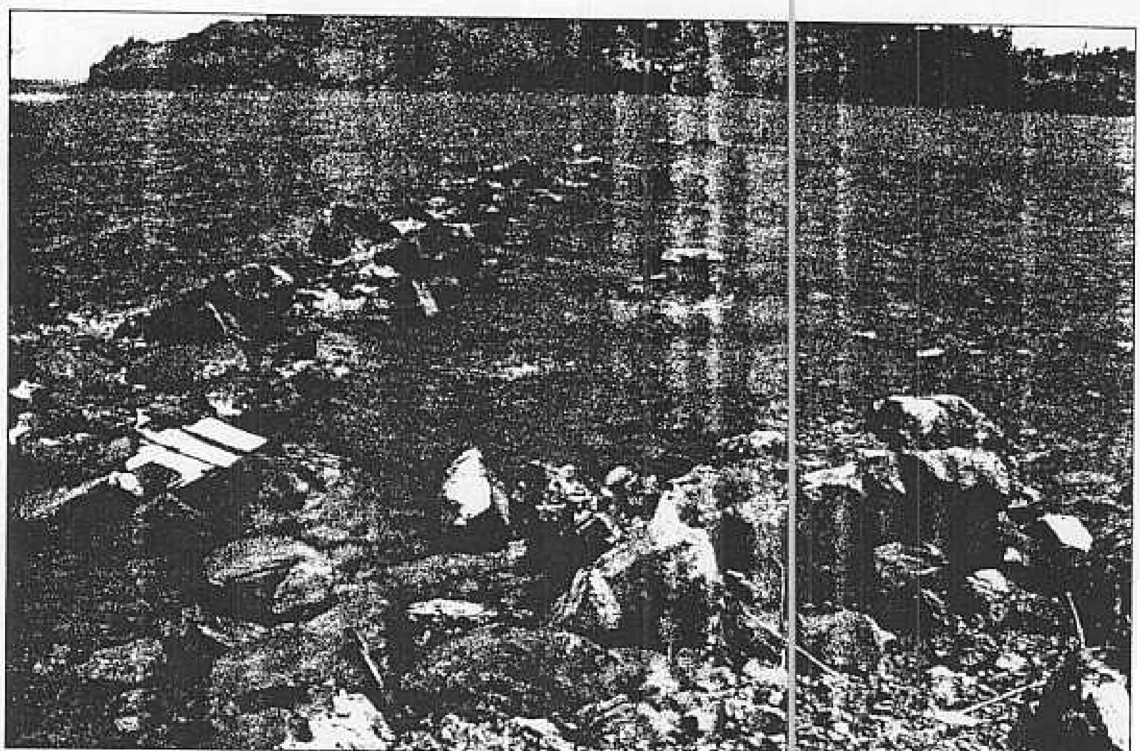

Figure 4. View of destroyed tip of Puerto Chico Breakwater after Hurricane Georges in 1998.

Another method of estimating breaking wave height during a hurricane is to hindcast based on observed damage. During hurricane Georges in 1998, the tip of the breakwater at Puerto Chico was destroyed. (See Figure 4). Armor stones of between 1.5 and 2.2 metric tones were displaced to the point where the damage level could be classified as $S = 8$ (See Figure 5). Assuming the same values discussed above for the rest of the parameters, one arrives at a wave height estimate of $4.0 \le H_b \le 4.5$ meters.

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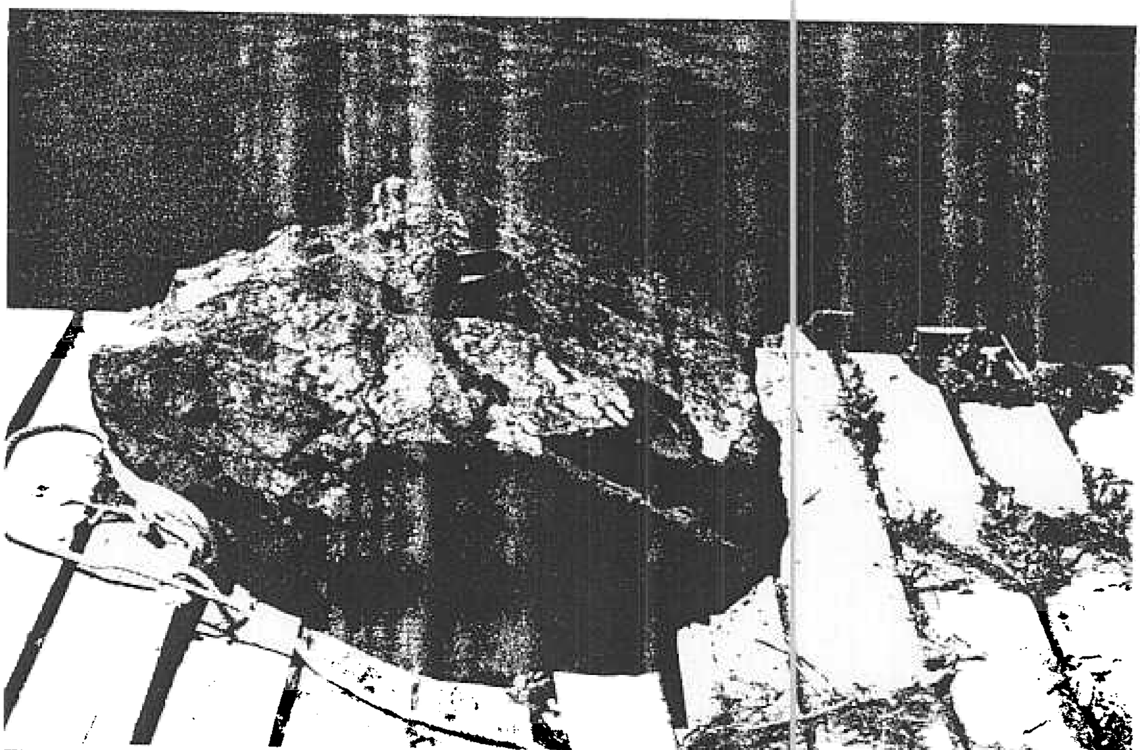

Figure 5. Example of displaced stone from Puerto Chico breakwater after Hurricane Georges in 1988.

Given the extreme damage evident in figures 4 and 5, we have chosen to take the most conservative approach and use $H_b = 4.68$ meters as our depth-limited design wave height. This corresponds to using $H_s = 3.27$ meters in Equations 12, which is derived for deep-water wave heights. Figure 6 is a plot of equation 12, with the parameter values discussed above.

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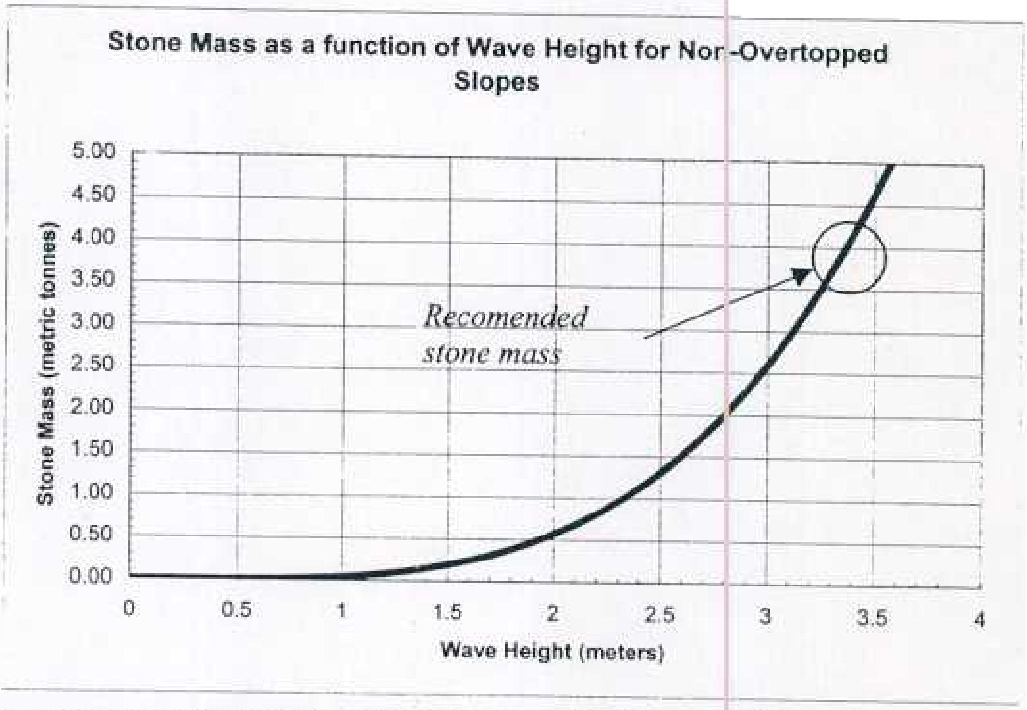
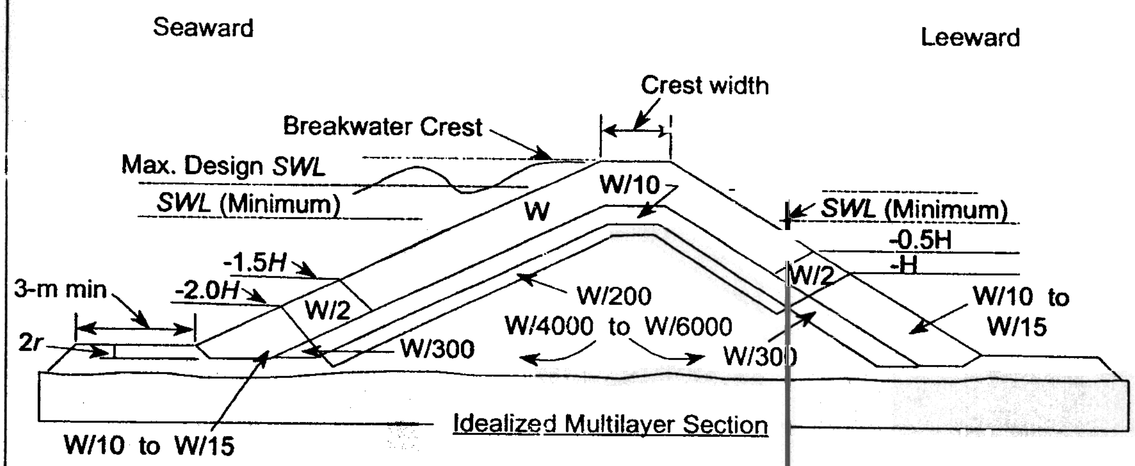


Figure 6. Example of stone mass as a function of wave height, assuming $T = 13$s, $t = 8$ hours, $S = 4$, $P = 0.5$, and $\alpha = .59$ radians (slope = 1:1.5).

Figure 6 is a plot of equation 12, with the parameter values discussed above. ***Our calculations indicate that the median stone diameter for the outer layer armor stone should be between 1.12 and 1.16 meters, corresponding to weights of between 3.7 and 4.15 metric tones.***

VII. Structure Cross-Section

A rubble-mound structure is normally composed of a bedding layer and a core of quarry-run stone covered by one or more layers of larger stone and an exterior layer or layers of large quarry stone or concrete units. A typical rubble mound cross section for



Rock Size	Layer	Rock Size Gradation (%)	Legend
W	Primary cover layer	125 to 75	H = Wave Height
W/2 and W/15	Secondary cover layer	125 to 75	W = Weight of individual armor unit
W/10 and W/300	First underlayer	130 to 70	r = Average layer thickness
W/200	Second underlayer	150 to 50	
W/4000-W/6000	Core and bedding layer	170 to 30	

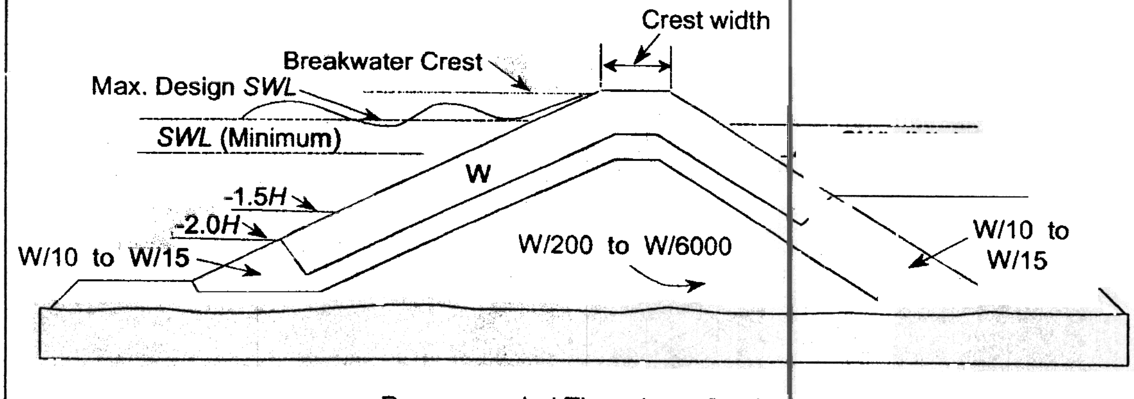
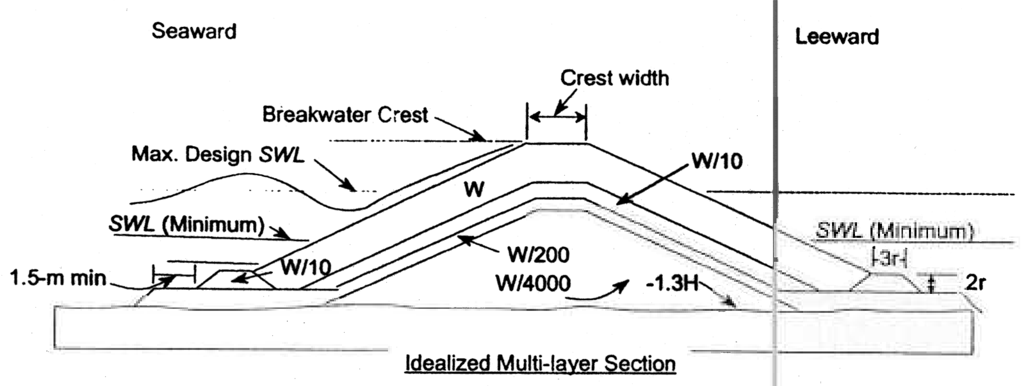


Figure 7. Rubble mound section for seaward wave exposure with zero-to-moderate overtopping conditions.

breakwaters exposed to waves on one side (seaward) and intended to allow minimal wave transmission to the other (leeward) side is shown in Figure 7. Breakwaters of this type are usually designed with crests elevated to allow overtopping only in very severe storms with long return periods.

Figure 8 shows features common to designs where the breakwater may be exposed to substantial wave action from both sides, such as artificial reefs, and the outer portion of jetties, where overtopping is more frequent.



Rock Size	Layer	Rock Size Gradation (%)	Legend
W	Primary cover layer	125 to 75	H = Wave Height
W/10	Toe berm and first underlayer	130 to 70	W = Weight of individual armor unit
W/200	Second underlayer	150 to 50	r = Average layer thickness
W/4000	Core and bedding layer	170 to 30	

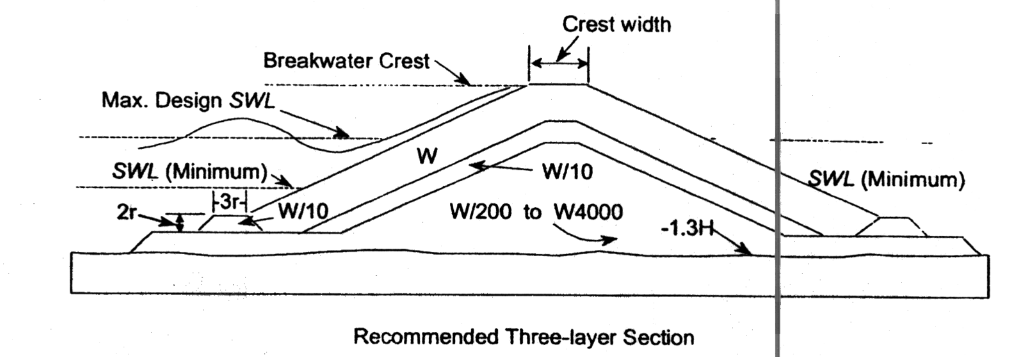


Figure 8. Rubble-mound section for wave exposure on both sides (*e.g.*, artificial reef)

More complex idealized cross-sections are shown in Figures 7 and 8, along with recommended cross-sections. The idealized cross sections provide more complete use of the range of materials typically available from a quarry, but it are more difficult to construct. The recommended cross-sections take into account some of the practical problems involved in constructing submerged portions of the structure. Figures 7 and 8 also include tables giving the average layer rock size in terms of the stable primary armor unit weight W, along with the gradation of stone used in each layer. To prevent smaller rocks from being pulled thorough an overlayer by [illegible] action, the following criteria for filter design may be used:

$$D_{15}(\text{cover}) \leq 5D_{85}(\text{under}) \qquad (14)$$

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where $D_{85}(\text{under})$ is the diameter exceeded by the coarsest 15 percent of the underlayer, and $D_{15}(\text{cover})$ is the diameter exceeded by the coarsest 85 percent of the layer immediately above the underlayer.

The outer layer of the structure should be constructed to a thickness of at least two median stone diameters, and the seaward facing slope angle of should be no greater than 33° (1:1.5 V:H).

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EXHIBIT H

GOVERNMENT OF PUERTO RICO
OFFICE OF THE GOVERNOR
PLANNING BOARD

MINILLAS GOVERNMENTAL CENTER, NORTH BLDG.
DE DIEGO AVE, STOP 22
P.O. BOX 41119, SAN JUAN, PUERTO RICO 00940-1119

September 29, 2000

Mr. Richard Vito
Sea Lovers Marina
P.O. Box 1064
Fajardo, Puerto Rico 00738

Federal Consistency Determination
CZ-2000-0329-057, Sol.-conj.0039
Extension of piers, Sardinera Ward
Fajardo, Puerto Rico

Dear Mr. Vito:

This letter is in response to your application for Certification of Consistency with the Puerto Rico Coastal Management Program (PRCMP) submitted for the expansion of the existing marina by additional 45 slips for a total of 185 at Sardinera Ward, Fajardo, Puerto Rico. The existing dock A would be extended an additional 210 feet with a "T" end pier of 105'x 12' and the placement of three tie-up pilings in order to accomodate larger boats. The dock B would be extended an addional 150 feet with a "T" end pier with piling similar to dock A. The width of the main piers is 12 feet. The purpose of the project is to provide additional slips for recreational boat users.

The consistency certification was submitted for an U.S. Army Corps of Engineers permit for the above mentioned project. The review period of this certification began on March 29, 2000. The certification was sent for comments to the Department of Natural and Environmental Resources, the Environmental Quality Board and the Council for Underwater Archaeology. Public notifications were also issued.

No commensts were received from the Department of Natural and Environmental Resources. The Fish and Wildlife Service (FWS) removed their objections after this agency discussed with the applicant their concerns. By letter dated on August 23, 2000, the applicant agreed to follow the recommendations suggested by the FWS in order to protect the manatee. The applicant agreed to participate in a manatee awareness program which includes posting signs (informational and speed limits) in the marina, providing educational material to the marina users and to limit the additional boat slips for the use of boats greater than 25 feet in lenght.

Pursuant to 15 CFR 930.64, the Federal Consistency Procedures with the PRCMP and based upon the project information submitted, the Puerto Rico Planing Board determines that the proposed project is consistent with the Puerto Rico Coastal Management Program. Notwithstanding, a concession from the Departament of Natural and Environmental Resources to use the maritime terrestrial zone, territorial waters and submerged lands should be required prior to the beginning of the works.

Sincerely,

José R. Caballero Mercado

José R. Caballero-Mercado
Chairman

Enclosure: Site plan

c: OCRM, Maryland
Mr. Edwin Muñiz, CoE
Mrs. Blanch González, DNER
PRCMO, DNER, San Juan
Mrs. Wanda García, EQB
FWS, Boquerón

BAM/ET/cgr