PN-ACR-179

Example Problems

# DESIGN OF COASTAL MARITIME STRUCTURES

for

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by

**David R. Basco, Ph.D., P.E.**
Professor of Civil Engineering
Director, the Coastal Engineering Centre
Old Dominion University
Norfolk, VA 23529
(basco@cee.odu.edu)

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## Introduction

"Only the application makes the rod into a lever" is the famous remark of the philosopher Ludwig Wittgenstein (1964). All engineers remember their university days (and nights) doing homework problems that turned the lectures (rod) into useful information and tools (lever) by the *application* of the materials presented. Those text books with many example problems (and answers to the homework problems) always rate as the best.

The new Coastal Engineering Manual (CEM) is divided into six Parts. The first four cover mainly the science surrounding the subject while the remaining Parts V and VI summarize our latest engineering knowledge for studies, designs and constructions. Chapter 7 of Part VI has been set aside for the Example Problems. This chapter is currently in preparation with some of the materials included in these notes, namely wave runup, wave overtopping, and armor-layer stability.

The single, most important improvement has been the use of irregular, water wave spectrums in the analytical treatment, physical, (laboratory) experiments, and numerical model simulations to study wave runup, overtopping and armor-layer stability. Coastal engineers must adopt this new technology quickly to prepare more cost effective and safe designs in the future.

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# Wave Runup

EXAMPLE PROBLEM VI-7-1

FIND:

The surf-similarity parameter* for use in wave runup and wave overtopping calculations for long-crested, irregular waves on impermeable and permeable slopes. [* Also called the Iribarren Number]

GIVEN:

An impermeable structure has a smooth slope of 1 on 2.5 and is subjected to a design, significant wave, $H_s = 2.0m$ (6.6 ft) measured at a gage located in a depth, $d = 4.5m$ (14.8 ft). Design wave peak period, $T_p = 8$ sec. Design depth at structure toe at high water is $d_{toe} = 3.0m$ (9.8 ft). (Assume no change in the refraction coefficient between the structure and the wave gage)

SOLUTION:

The surf-similarity parameter for irregular waves depends on the wave steepness and structure slope. Two definitions are given as equation (VI-5-2) depending on the peak wave period, $T_p$ or the mean wave period, $T_m$, but both use the significant wave height at the toe of the structure.

1. Linear, regular wave shoaling

(a) Deep water

First calculate the deepwater, unrefracted wave height, $H_o'$ from where measured back out to deep water. Using the depth where waves measured

$$\frac{d}{L_o} = \frac{2\pi d}{gT^2} = \frac{2\pi(4.5)}{(9.81)(8^2)} = 0.0451$$

(1) From the SPM (1984), Table C-1, Appendix C for $d/L_o = 0.0451$

$$\frac{H}{H_o'} = 1.041, \text{ the shoaling coefficient, } K_s$$

Therefore, $$H_o' = \frac{2.0}{1.041} = 1.92\,m\ (6.3\,ft)$$

(2) Or, using ACES (Leenknecht et al., 1992), Snell's Law, Crest Angle = 0.0°

$$H_o' = 1.92\,m\ (6.3\,ft)$$

(Note: ACES indicates wave breaks in $d_b = 3.16m$, $H_b = 2.61m$, but setting $d = 3.17m$ gives $H = 2.14\,m$. ACES does not permit finding H in $d = 3.0m$ water depth

(3) Or, using Alternate Wave Tables (e.g. Nielsen, 1984)

$$\frac{C_g}{C_o} = \sqrt{k_o d}\left[1 - \frac{1}{2}k_o d + \frac{7}{72}(k_o d)^2\right]$$

$$k_o \equiv \frac{2\pi}{L_o} = \frac{4\pi^2}{gT^2}$$

$$K_s \equiv \sqrt{0.5\ C_o/C_g}$$

EXAMPLE PROBLEM VI-7-1 (continued)

gives $K_{od} = 0.28296$

$$\frac{C_g}{C_o} = 0.46082$$

$$K_s = 1.0416$$

and $H_o' = 1.920$ m (6.3 ft)

(b) Toe of structure

Next, shoal the deep water wave to a depth, $d = 3.0$ m at the toe of the structure

(1) From the SPM (1984), Table C-1, Appendix for

$$\frac{d_{toe}}{L_o} = \frac{2\pi d_{toe}}{gT^2} = \frac{2\pi(3.0)}{(9.81)(8^2)} = 0.030023$$

$$K_s = \frac{H_{toe}}{H_o'} = 1.125$$

$$H_{toe} = 2.16\text{m } (7.0\text{ ft})$$

(2) From ACES, $H = 2.14$m at $d = 3.17$m, cannot find $H_{toe}$ because gives $H_b = 2.61$ m when $d_b = 3.16$m and ERROR message when $d = 3.01$

(3) From Alternate Wave Table

$$K_{od} = 0.18864$$

$$\frac{C_g}{C_o} = 0.39486$$

$$K_s = 1.1253 \text{ m}$$

$$H_{toe} = 2.161 \text{ m} \cdot (7.0 \text{ ft})$$

2. Wave steepness, $S_{op}$

$$(S_{op})_{toe} \equiv \frac{(H_s)_{toe}}{L_o} = \frac{2\pi H_{toe}}{gT_p^2} = \frac{2\pi(2.16)}{9.81(8)^2}$$

$$S_{op} = 0.02162$$

3. Surf-similarity parameter, $\xi_{op}$

Finally, the surf-similarity parameter, $\xi_{op}$ as defined (VI-5-2)

$$\xi_{op} \equiv \frac{\tan\alpha}{\sqrt{S_{op}}} = \frac{1/2.5}{\sqrt{0.02162}} = \frac{0.4}{0.1469}$$

Therefore,

$$\underline{\xi_{op} = 2.72}$$

Note that the subscript notation means using the deepwater wave length, $L_o$ and the peak wave period, $T_p$ to calculate $\xi_{op}$

EXAMPLE PROBLEM VI-7-1 (continued)

4. Surf-similarity parameter, $\xi_{om}$

The mean wave period, $T_m$ requires knowledge of variations in the width of the wave spectrum. From Part VI-5-2 (p. VI-5-18) for the theoretical spectrums

JONSWAP $T_m/T_p = 0.79$ to $0.87$.

PIERSON-MOSKOWITZ $T_m/T_p = 0.71$ to $0.82$

Therefore, assuming $T_m/T_p = 0.76$, $T_m = 6.1$ sec

$$S_{om} = \frac{H_{toe}}{\frac{g}{2\pi}T^2} = \frac{2.16\,(2\pi)}{9.81\,(6.1^2)} = 0.03718$$

Therefore

$$\xi_{om} \equiv \frac{\tan\alpha}{\sqrt{S_{om}}} = \frac{1/2.5}{\sqrt{0.03718}} = \frac{0.4}{0.1928}$$

$$\underline{\xi_{om} = 2.07}$$

Both $\xi_{op}$ and $\xi_{om}$ are employed in wave runup and overtopping formulations.

DISCUSSION

In general (for either $\xi_{op}$ or $\xi_{om}$)

$$\xi_o \equiv \frac{\tan\alpha}{\sqrt{S_o}}$$

$$S_o \equiv \frac{H_s}{L_o} = \frac{2\pi\, H_s}{g\,T^2}$$

$$\therefore \quad \xi_o \equiv \frac{\tan\alpha\; T}{\left(\frac{2\pi}{g}\right)^{1/2}\left(H_s\right)^{1/2}}$$

or

$$\xi_o \equiv \frac{T}{(K)\cot\alpha\,(H_s)^{1/2}}$$

(1) As $T$ increases, $\xi_o$ increases

(2) As $\cot\alpha$ increases (flatter), $\xi_o$ decreases

(3) As $H_s$ increases, $\xi_o$ decreases, nonlinearly

DRBasco
8/14/00

EXAMPLE PROBLEM VI-7-2

FIND: (a) The height above the still water level (SWL) to which a revetment must be built to prevent wave overtopping by the design wave.
(b) The reduction in required structure height if uniform-sized riprap is placed on the slope

GIVEN: An impermeable structure has a smooth slope of 1 on 2.5 and is subjected to a design significant wave, $H_s = 2.0$ m (6.6 ft) measured at a gage located in a depth, $d = 4.5$ m (14.8 ft). Design wave peak period, $T_p = 8$ sec. Design depth at structure toe at high water is $d_{toe} = 3.0$ m (9.8 ft).

SOLUTION: From Example Problem VI-7-1, linear wave theory gives the wave height due to wave shoaling as

$$(H_s)_{toe} = 2.16 \text{ m } (7.0 \text{ ft})$$

and the surf-similarity parameter as

$$\xi_{op} = 2.72$$

To prevent wave overtopping, the wave runup value at the 2 percent probability of exceedance level is calculated. Figure VI-5-3 displays the considerable scatter in the data for smooth slopes, irregular, long-crested, head-on waves and Table VI-5-2 gives the coefficients for use in Equation VI-5-3 when

$$\xi_{op} > 2.5$$

namely

$$\frac{R_{u2\%}}{(H_s)_{toe}} = -0.2\,\xi_{op} + 4.5$$

$$= -0.2(2.72) + 4.5 = -0.545 + 4.5 = 3.955$$

and

$$R_{u2\%} = 3.955(2.16)$$

(a) Smooth slopes

(1) To prevent overtopping; $R_{u2\%} = 8.55$ m

(Note that $\gamma_r = \gamma_b = \gamma_h = \gamma_\beta = 1.0$ are taken in Eqn VI-5-3 for smooth, no berm, Rayleigh Distribution, and zero incidence angle conditions, respectively.)

Another set of runup data for smooth slopes is presented in Figure VI-5-5 and Equation VI-5-6. (from de Waal and van der Meer, 1992). When

$$\xi_{op} > 2.0$$

namely

$$\frac{R_{u2\%}}{(H_s)_{toe}} = 3.0$$

hence

(2) To prevent overtopping, $R_{u2\%} = 6.48$ m

EXAMPLE PROBLEM VI-7-2 (continued)

(b) Rough slopes

The surface roughness reduction factor, $\gamma_r$ for Equation VI-5-3 is given in Table VI-5-3 and lies in the range, $\gamma_r = 0.5 - 0.6$ for one or more layers of rock.

$$\text{Use } \gamma_r = 0.55$$

(1) Eqn VI-5-3 $\quad \dfrac{R_{u2\%}}{(H_s)_{toe}} = 3.955\,\gamma_r = 3.955(0.55) = 2.175\,m$

hence $\quad \underline{R_{u2\%} = 4.70\,m}$

(2) Eqn VI-5-6 $\quad \dfrac{R_{u2\%}}{(H_s)_{toe}} = 3.0\,\gamma_r = 3.0(0.55) = 1.65$

and $\quad \underline{R_{u2\%} = 3.56\,m}$

(3) The Delft Hydraulics test program (Table VI-5-4) also provided data for <u>impermeable</u> rock slopes. Here, the mean wave period, surf-similarity parameter, $\xi_{om}$ is employed to develop design Equation (VI-5-12) with coefficients in Table VI-5-5. for a wide range of exceedance probabilities. (See Example Problem VI-7-1 for $\xi_{om}$ value for this example, $\xi_{om} = 2.07$)

When $\xi_{om} \geq 1.5$

$$\frac{R_{u2\%}}{(H_s)_{toe}} = 1.17\left(\xi_{om}\right)^{0.46}$$

$$= 1.17(2.07)^{0.46} = 1.635$$

Therefore $\quad R_{u2\%} = 1.635(2.16) = \underline{3.53\,m}$

using coefficients for B and C at the 2% exceedance probability level. This result is very close to that in (2) above taking $\gamma_r = 0.55$.

(4) Partial safety factors, $\gamma_H$ and $\gamma_z$ and $\hat{R}_{u2\%}$. The Delft Hydraulics data set has been analyzed for partial safety factors as discussed in Part VI-6 and Table VI-6-17.

Assume the annual failure probability, $P_f = 0.10$ (90% reliability). For relatively low uncertainty in knowledge of the wave height ($\sigma'_{FHS} = 0.05$) gives

$$\gamma_H \cdot \gamma_z = 1.2(1.06) = 1.272$$

and a probabilistic estimate, $\hat{R}_{u2\%} = 1.272(3.53) = 4.49\,m$. If the uncertainity is higher regards wave height ($\sigma'_{FHS} = 0.2$) then

$$\gamma_H \cdot \gamma_z = 1.3(1.08) = 1.404$$

EXAMPLE PROBLEM VI-7-2 (continued)

and a probabilistic estimate, $\hat{R}_{u2\%} = 1.404\,(3.53) = 4.96$ m

The range of $\hat{R}_{u2\%} = 4.5 - 5.0$ m brackets the estimate of $R_{u2\%} = 4.7$ m as found from Eqn VI-5-3. This higher estimate of the $R_{u2\%}$ value could also be explained as being reliable at the 90% annual level.

DISCUSSION

As seen in Figure VI-5-3, at $\xi_{op} = 2.5$, $R_{u2\%}/H_s$ reaches a maximum value. Solving for the variables involved in $\xi_{op}$ gives approximately

Metric System

$$\frac{\tan\alpha\, T_p}{(H_{toe})^{1/2}} \doteq 2.0$$

English System

$$\frac{\tan\alpha\, T_p}{(H_{toe})^{1/2}} = 1.10$$

For the above example, keeping $\tan\alpha = 0.4$, $H_{toe} = 2.16$ m gives

$(T_p) = 7.35$ sec for maximum runup

For the above example, keeping $T_p = 8.0$ sec, $H_{toe} = 2.16$ m gives

$\tan\alpha = 0.36$ $\cot\alpha = 2.7$ for maximum runup.

Using a steeper or flatter slope will reduce the wave runup, all else being equal

D. Basco
8/14/00



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EXAMPLE PROBLEM VI-7-3

FIND: The height above the SWL to which a rock-armored structure (permeable) should be built to prevent wave overtopping by the design wave.

GIVEN: The same data for Example Problem VI-7-2 as summarized below, but now for a permeable breakwater (jetty) structure

$$\text{slope} = 1:2.5$$
$$H_s = 2.0\ m$$
$$\text{measured } d = 4.5\ m$$
$$T_p = 8\ sec$$
$$d_{toe} = 3.0\ m$$

SOLUTION: Core permeability may significantly influence wave runup. Notational permeability coefficients are defined in Figure VI-5-11. The previous Example Problems VI-7-2 and VI-7-3 were for $P=0.1$ defined as impermeable. Test results shown in Figures VI-5-12 are with $P=0.1$ and $P=0.5$ and clearly reveal the runup reduction when $\xi_{om} > 3$ for permeable structures. Equation VI-5-13 has been developed as the central fit to the permeable data with coefficients again found in Table VI-5-5.

For $R_{u2\%}$ $B=1.17$, $C=0.46$ and $D=1.97$ gives

$$(D/B)^{1/C} = \left(\frac{1.97}{1.17}\right)^{\frac{1}{0.46}} = (1.68)^{2.17} = 3.10$$

Use $1.5 < \xi_{om} < (D/B)^{1/C}$

$$\frac{R_{u2\%}}{(H_s)_{toe}} = B(\xi_{om})^C$$
$$= 1.17(2.07)^{0.46} = 1.635\ m$$

Therefore $R_{u2\%} = 3.53\ m$

and the same result as for $P=0.1$, impermeable slopes.

By definition,

$$\xi_{om} \equiv \frac{\tan\alpha}{\sqrt{S_{om}}} \geq 3.10$$

at the point where the permeable core reduces wave runup. Longer period waves will increase $\xi_{om}$ but the runup remains constant because of the structure permeability. At this limit,

$$\frac{R_{u2\%}}{(H_s)_{toe}} = D = 1.97 \text{ (see Table VI-5-5)}$$

Therefore $(R_{u2\%})_{max} = 4.25\ m$ for $T_m = 9.1\ sec$ ($T_p \doteq 12\ s$)

EXAMPLE PROBLEM VI-7-4

FIND: The height above the stillwater level to which a revetment must be built to prevent wave overtopping by the design wave (same as Ex. Prob. VI-7-2) but for following conditions.
(a) statistical distributions of wave runup.
(b) influence of shallow water on wave runup
(c) influence of wave angle and directional spreading on wave runup

GIVEN: Same conditions as Example Problem VI-7-2 for smooth slope

SOLUTION: Equation VI-5-3 holds in general for any $R_{ui\%}$ defined as the runup level exceeded by $i$ % of the incident waves. Coefficients A and C depend on both $\xi_{op}$ and $i$ for Rayleigh distributed wave heights.

(a) statistical distributions

(1) significant runup. Figure VI-5-4 displays the data scatter and Table VI-5-2 provides coefficients to calculate the significant wave runup, $R_{us}$. Again from Example Problem VI-7-1

$$(H_s)_{toe} = 2.16\,m$$

and

$$\xi_{op} = 2.72$$

so that when $2 < \xi_{op} < 9$

$$\frac{R_{us}}{(H_s)_{toe}} = -0.25\,\xi_{op} + 3.0$$

$$= -0.25(2.72) + 3.0 = -0.68 + 3.0 = 2.32$$

therefore, $R_{us} = 5.01\ m$

The SPM (1984) calculated a runup value of 5.6m for Example Problem No. 4 for the same data taking the design wave as the significant wave height. The Rayleigh distribution for wave heights gives the following relationships for extreme events

$$\frac{H_{0.1}}{H_{0.135}} = \frac{H_{0.1}}{H_s} = 1.073$$

$$\frac{H_{0.02}}{H_s} = 1.398$$

$$\frac{H_{0.01}}{H_s} = 1.517$$

If the wave runup also followed a Rayleigh distribution, then

EXAMPLE VI-7-4 (continued)

$$\frac{R_{u0.2}}{R_{us}} = 1.398$$

gives $R_{u0.2} = 1.398\,(5.01) = 7.0\,m$

This result is much lower than $R_{u2\%} = 8.55\,m$ calculated in EXAMPLE VI-7-2 for the smooth slope. In general, values for $R_{us}$ and $R_{u2\%}$ calculated from EQN VI-5-3 and coefficients in Table VI-5-2 do not follow a Rayleigh Distribution for wave runup, [illegible]

(2) statistical distributions of runup on permeable slopes.

For the following restrictions*: (1) Rayleigh distributed wave heights
(2) Permeable, rock armored slopes
(3) Slope, $\cot\alpha > 2$

Equation VI-5-15 says

$$R_{up\%} = B(-\ln p)^{1/C}$$

where $R_{up\%}$ = runup level exceeded by $p$% of the runup

and B, C are calculated from Eqn VI-5-16, 17, respectively.

From Example Problems VI-7-1 and VI-7-2

$$\xi_{om} = 2.07 \qquad (S_{om} = 0.03718)$$
$$P = 0.5$$
$$\tan\alpha = 0.4$$

gives

$$\xi_{omc} = \left(5.77\,P^{0.3}\sqrt{\tan\alpha}\right)^{[1/(P+0.75)]}$$
$$= \left(5.77\,(0.5)^{0.3}\sqrt{0.4}\right)^{[1/(1.25)]}$$
$$= (2.964)^{0.8}$$
$$\xi_{omc} = 2.385$$

Therefore, $\xi_{om} < \xi_{omc}$, plunging waves

so that

$$C = 3.0(\xi_{om})^{-3/4}$$
$$= 3.0\,(2.07)^{-3/4} = 1.738 \qquad \left(\frac{1}{C} = 0.5754\right)$$

(Note: when $C = 2$, Eqn VI-5-15 becomes the Rayleigh distribution)

The scale parameter becomes

* See DISCUSSION

EXAMPLE VI-7-4 (continued)

$$B = H_s\left[0.4(S_{om})^{-1/4}(\cot\alpha)^{-0.2}\right]$$

or

$$= 2.16\left[0.4(0.03718)^{-1/4}(2.5)^{-0.2}\right]$$

$$= 2.16\left[0.4(2.2773)(0.83255)\right]$$

$$= 2.16\left[0.7584\right]$$

$$= 1.638$$

Now check previous results

$$R_{u2\%} = 1.638\left[-\ln(0.02)\right]^{1/1.738}$$

$$R_{u2\%} = 1.638\left[3.912\right]^{0.5754}$$

$$= 1.638\left[2.192\right]$$

$$R_{u2\%} = 3.59\ m$$

OKAY see (EX PROB VI-7-3) $R_{u2\%} = 3.53m$

and

$$R_{us} = 1.638\left[-\ln(0.135)\right]^{0.5754}$$

$$R_{us} = 1.638\left[2.002\right]^{0.5754}$$

$$= 1.638\left[1.491\right]$$

$$R_{us} = 2.44\ m$$

Now

$$\frac{R_{u2\%}}{R_{us}} = \frac{3.59}{2.44} = 1.47$$

and is not the same as the Rayleigh distribution where $H_{2\%} = 1.398\ H_s$

At the 1% level

$$R_{u1\%} = 1.638\left[-\ln(0.01)\right]^{0.5754}$$

$$= 1.638\left[4.605\right]^{0.5754}$$

$$= 1.638\left[2.4079\right] = 3.94m$$



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EXAMPLE PROBLEM VI-7-4 (continued)

For design, wave runup values calculated at the 2% exceedance probability level are considered the upper limit "... to prevent wave overtopping".

(b) influence of shallow water on wave runup

Assuming the breaker index for shallow water wave breaking as

$$\left(\frac{H}{d}\right)_b = 0.78$$

then $H_b = 0.78d = 0.78(3.0) = 2.34$ m.

Therefore, since $H_s = 2.16$ m $< H_b$, no wave breaking occurs

Hence taking $\gamma_h = 1.0$ is justified.

Note that if the design water depth at the structure toe dropped to 2.8 m, then breaking begins. Equation VI-5-10 can only be applied if knowledge of $H_{2\%}$ and $H_s$ are known from field data or numerical model results

(c) influence of wave angle and directional spreading

As seen in Equation VI-5-11, the above results hold for wave angles, $\beta$ less than 10° from normal incidence of long-crested swell-type, wave spectrums. Angles of incidence larger than 10° will reduce the wave runup ($\gamma_\beta < 1$) as expected.

Short-crested, wide spectrums give similar results, i.e. when the nominal wave angle, $\beta$ is less than 10° ($\gamma_\beta = 1.0$).

DISCUSSION:

For other conditions, the statistical distribution of the wave runup has not been analyzed.

DRBasco
8/15/00



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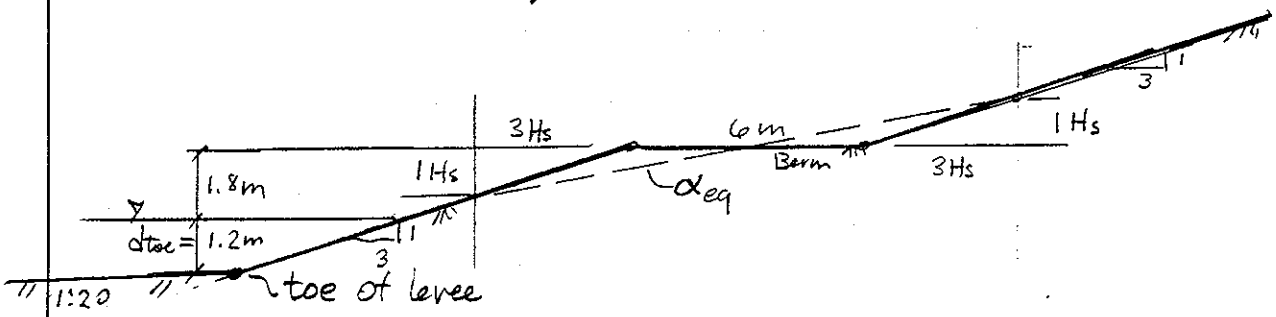
EXAMPLE PROBLEM VI-7-5

FIND: Determine the wave runup at the 2% exceedance probability level for a composite slope shown below

GIVEN: A smooth-faced levee of composite slope shown with water depth, $d_{toe} = 1.2\,m$ is subjected to a significant wave height in deepwater $H_o' = 1.5\,m$ and $T_p = 8$ sec. The offshore slop is 1:20




SOLUTION: (1) Wave height, $(H_s)_{toe}$

(a) SPM (1984)

$$L_o = \frac{g}{2\pi} T^2 = 1.56(8)^2 = 99.92\,m$$

$$\frac{H_o'}{L_o} = \frac{1.5}{99.92} = 0.015$$

From Fig. 7-2 SPM (1984) for $m = 1:20$

$$\frac{d_b}{H_b} = 0.95$$

$d_b = 2.1\,m$ and occurs 18 m in front of toe

(b) ACES 1.07

GODA method not applicable for $d < 3.048\,m$

Linear theory / Snell's Law

Wave broken, $H_b = 2.41\,m$, $d_b = 2.33\,m$

gives $\frac{d_b}{H_b} = \frac{2.33}{2.41} = 0.97$ (checks okay)

(c) Assume wave energy decay continues from $d_b = 2.1 - 2.3\,m$ to toe of levee,

hence, $(H)_{toe} = \frac{d_b}{0.95} = \frac{1.2}{0.95} = 1.25\,m$

Use $(H_s)_{toe} = 1.25\,m$ at toe of levee

EXAMPLE PROBLEM VI-7-5 (continued)

(2) Berm influence factor, $\gamma_b$

(a) Breaking wave surf similarity parameter based on an equivalent slope, $\xi_{eq}$. (see sketch)

equivalent slope, $\alpha_{eq} = \tan^{-1} \dfrac{1.25}{3+3(1.25)} = \tan^{-1} 0.185 = 10.48°$

average slope $\alpha = \tan^{-1} 0.333 = 18.43°$

$$S_{op} \equiv \frac{(H_s)_{toe}}{L_{op}} = \frac{2\pi (H_s)_{toe}}{g T_p^2} = \frac{2\pi(1.25)}{9.81(8)^2} = 0.01251$$

Therefore $$\xi_{eq} = \frac{0.185}{\sqrt{0.01251}} = 1.65$$

$$\xi_{op} = \frac{0.333}{\sqrt{0.01251}} = 2.977$$

From Eqn VI-5-8

$$\gamma_b = \frac{\xi_{eq}}{\xi_{op}} = \frac{1.65}{2.977} = 0.55$$

Since $0.6 < \gamma_b \leq 1.0$ Use $\gamma_b = 0.6$

For $\xi_{eq} \leq 2$ from Eqn VI-5-7

$$\frac{R_{u2\%}}{(H_s)_{toe}} = 1.5\, \xi_{op}\, \gamma_r\, \gamma_b\, \gamma_h\, \gamma_\beta$$

Take, $\gamma_r = 1.0$ (smooth)

$\gamma_\beta = 1.0$ ($\theta = 0°$)

$\gamma_h = 0.9$ (since wave breaking begins 13 m from toe, assume some reduction in $\gamma_h$)

$\gamma_b = 0.6$ berm influence

gives $$\frac{R_{u2\%}}{(H_s)_{toe}} = 1.5(2.977)(1.0)(0.6)(0.9)(1.0)$$

$$= 2.411$$

$$\underline{R_{u2\%} = 3.0\ m}$$

(Note: by composite method, SPM (1984) gives $R_{u5} = 1.8$ m. Assuming Rayleigh distribution (approximation), $R_{u2} = 1.4\, R_{u5} = 2.5$ m)



D. Basco 8/4/90

(2) Surf Similarity Parameter (Irribarren Number). Wave runup and rundown on a structure depend on the type of wave breaking. Breaker types can be identified by the so-called surf-similarity parameter, $\xi$ (Battjes 1974b). The parameter $\xi$ is also referred to as the breaker parameter or Irribarren number. The surf-similarity parameter was originally defined for *regular* waves as

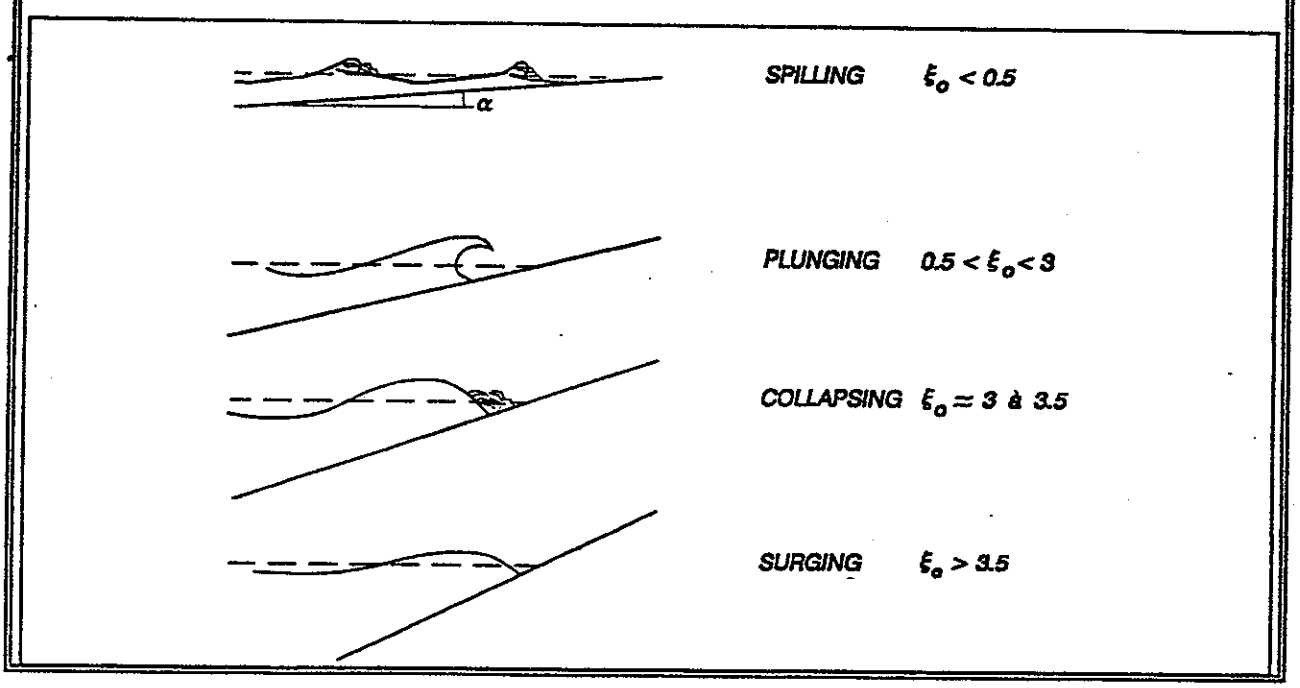
$$\xi_o = \frac{\tan \alpha}{\sqrt{s_o}} \qquad \text{(VI-5-1)}$$

where

- $\alpha$ - slope angle
- $s_o$ - deep water wave steepness [$= H_o/L_o$]
- $H_o$ - deep water wave height
- $L_o$ - deep water wave length [$= gT^2/2\pi$]
- $T$ - wave period
- $g$ - acceleration due to gravity

The wave height $H_b$ at the breaking point is sometimes substituted for $H_o$ in which case the parameter is denoted by $\xi_b$. Breaker types and related ranges of $\xi_o$-values are given for impermeable slopes in Table VI-5-1. The boundaries of transition from one type of breaker to another are approximate.

**TABLE VI-5-1**
**Types of Wave Breaking on Impermeable Slopes and Related $\xi_o$-Values.**



For *irregular* waves the surf--similarity parameter is defined as

$$\xi_{om} = \frac{\tan \alpha}{\sqrt{s_{om}}} \qquad or \qquad \xi_{op} = \frac{\tan \alpha}{\sqrt{s_{op}}} \tag{VI-5-2}$$

where

$$s_{om} = \frac{H_s}{L_{om}} = \frac{2\pi}{g}\frac{H_s}{T_m^2}$$

$$s_{op} = \frac{H_s}{L_{op}} = \frac{2\pi}{g}\frac{H_s}{T_p^2}$$

and

$H_s$ - significant wave height of incident waves at the toe of the structure
$T_m$ - mean wave period
$T_p$ - wave period corresponding to the peak of the wave spectrum

Note that $s_{om}$ and $s_{op}$ are fictitious wave steepnesses because they are ratios between a statistical wave height at the structure and representative deep water wave lengths.

The relative runup $R_u/H$ is a function of $\xi$, the wave angle of incidence, and the slope geometry (profile, surface roughness, porosity). Differences in runup characteristics make it convenient to distinguish between impermeable and permeable slopes. Impermeable slopes belong to dikes, revetments, and breakwaters with either impermeable surfaces (e.g. asphalt, concrete) or rough surfaces (e.g., rubble stones, concrete ribs) on fine core materials. Permeable slopes belong typically to rubble-mound structures with secondary armor layers, filter layers, and quarryrun core.

(3) Wave Runup and Rundown on Impermeable Slopes. Runup on impermeable slopes can be formulated in a general expression for irregular waves having the form (Battjes 1974)

$$\frac{R_{ui\%}}{H_s} = (A\xi + C)\gamma_r\ \gamma_b\ \gamma_k\ \gamma_\beta \tag{VI-5-3}$$

where

$R_{ui\%}$ - runup level exceeded by $i\%$ of the incident waves

$\xi$ - surf-similarity parameter, $\xi_{om}$ or $\xi_{op}$

$A$, $C$ - coefficients dependent on $\xi$ and $i$ but related to the reference case of a smooth, straight

impermeable slope, long-crested head-on waves and Rayleigh-distributed wave heights

$\gamma_r$ - reduction factor for influence of surface roughness ($\gamma_r$ = 1 for smooth slopes)

$\gamma_b$ - reduction factor for influence of a berm ($\gamma_b$ = 1 for non-bermed profiles)

$\gamma_h$ - reduction factor for influence of shallow water conditions where the wave height distribution deviates from the Rayleigh distribution ($\gamma_h$ = 1 for Rayleigh distributed waves)

$\gamma_\beta$ - factor for influence of angle of incidence $\beta$ of the waves ($\gamma_\beta$ = 1 for head-on long-crested waves, i.e., $\beta$ = 0°). The influence of directional spreading in short-crested waves is included in $\gamma_\beta$ as well.

(a) Smooth Slope, Irregular Long-Crested Head-On Waves. Van Oorschot and d'Angremond (1968) tested slopes of 1:4 and 1:6 for $\xi_{op}$ < 1.2. Ahrens (1981a) investigated slopes between 1:1 and 1:4 for $\xi_{op}$ > 1.2. Figure VI-5-3 shows the range of test results and the fit of Eqn. VI-5-3 for $R_{u2\%}$. Considerable scatter is observed, most probably due to the fact that the runs for $\xi_{op}$ > 1.2 contained only 100-200 waves. The coefficient of variation, $\sigma_{Ru} / \overline{R}_u$, seems to be approximately 0.15.

The *significant runup* level $R_{us} = R_{u33\%}$ depicted in Figure VI-5-4 does not contain data for $\xi_{op}$ < 1.2. The coefficient of variation appears to be approximately 0.1.

The coefficients $A$ and $C$ together with estimates of the coefficient of variation for $R_u$ are given in Table VI-5-2. It should be noted that data given in Allsop, et al. (1985) showed runup levels considerably smaller than given here.

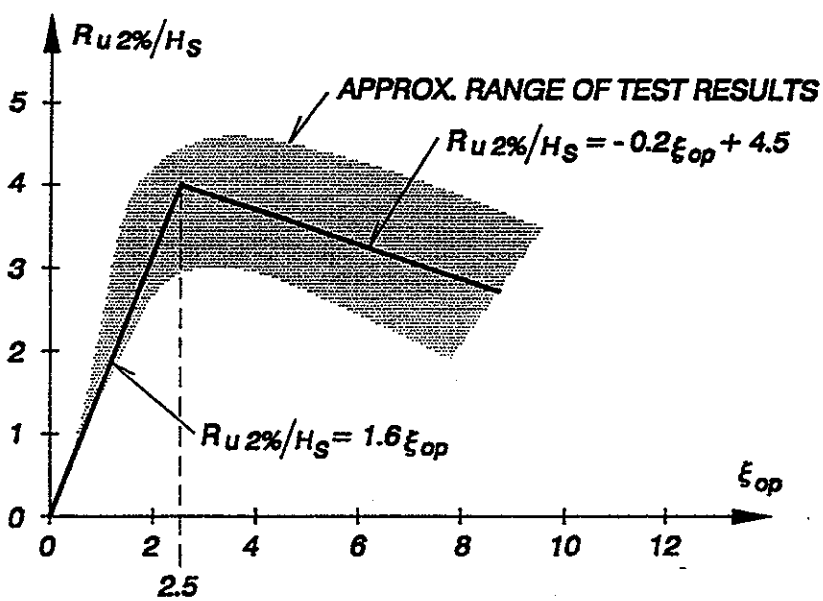



Figure VI-5-3. $R_{u2\%}$ for head-on waves on smooth slopes. Data by Ahrens (1981a) and Van Oorschot and d'Angremond (1968).

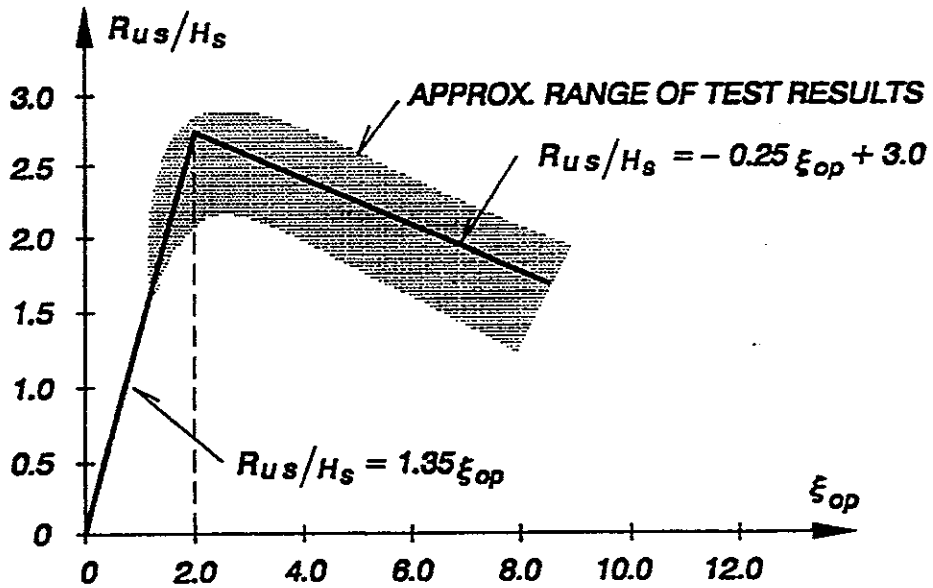



Figure VI-5-4. $R_{us}$ for head-on waves on smooth slopes. Data by Ahrens (1981a).

**TABLE VI-5-2**
**Coefficients in Eqn. VI-5-3 for Runup of Long-Crested Irregular Waves on Smooth Impermeable Slopes**

| $\xi$ | $R_u$ | $\xi$-Limits | $A$ | $C$ | $\sigma_{Ru} / R_u$ |
|---|---|---|---|---|---|
| $\xi_{op}$ | $R_{u2\%}$ | $\xi_p \le 2.5$ | 1.6 | 0 | $\approx 0.15$ |
| | | $2.5 < \xi_p < 9$ | -0.2 | 4.5 | |
| | $R_{us}$ | $\xi_p \le 2.0$ | 1.35 | 0 | $\approx 0.10$ |
| | | $2.0 < \xi_p < 9$ | -0.25 | 3.0 | |

Generally less experimental data are available for rundown. Rundown corresponding to $R_{d2\%}$ from long-crested irregular waves on a smooth impermeable slope can be estimated from

$$\frac{R_{d2\%}}{H_s} = \begin{cases} 0.33\ \xi_{op} & for \quad 0 < \xi_{op} \le 4 \\ 1.5 & for \quad \xi_{op} > 4 \end{cases} \qquad \text{(VI-5-4)}$$

In the Dutch publication by Rijkswaterstaat *"Slope revetments of placed blocks, 1990"* the following expression was given for rundown on a smooth revetment of placed concrete block

$$\frac{R_{d2\%}}{H_s} = 0.5\ \xi_{op} - 0.2 \qquad \text{(VI-5-5)}$$

Another set of runup data for long-crested head-on waves on smooth slopes was presented by de Waal and van der Meer (1992). The data cover small scale tests for slopes 1:3, 1:4, 1:5, 1:6 and large scale tests for slopes 1:3, 1:6, 1:8. The surf-similarity parameter range for the small scale tests is $0.6 < \xi_{op} < 3.4$, and for the large scale tests $0.6 < \xi_{op} < 2.5$. The data are shown in Figure VI-5-5 and were used by de Waal and van der Meer (1992) and van der Meer and Jansson (1995) as the reference data for the evaluation of the $\gamma$-factors defined by Eqn. VI-5-3.

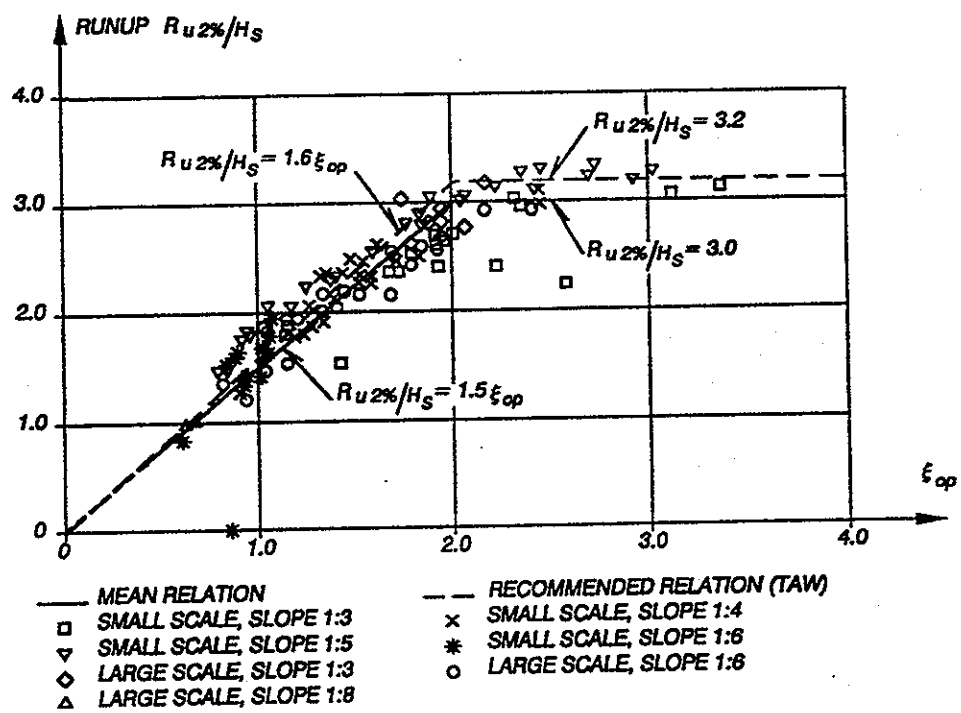



Figure VI-5-5. $R_{u2\%}$ for Long-Crested Head-On Waves on Smooth Slopes. From de Waal and van der Meer (1992).

The mean relationship, taken as the reference case for Eqn. VI-5-3, is shown with the solid line and is represented by the expression

$$\frac{R_{u2\%}}{H_s} = \begin{cases} 1.5\,\xi_{op} & \text{for} \quad 0.5 < \xi_{op} \le 2 \\ 3.0 & \text{for} \quad 2 < \xi_{op} < 3\text{-}4 \end{cases} \qquad \text{(VI-5-6)}$$

The dotted line includes a small safety factor, and this relationship is recommended for design by the Technical Advisory Committee on Water Defence in Holland.

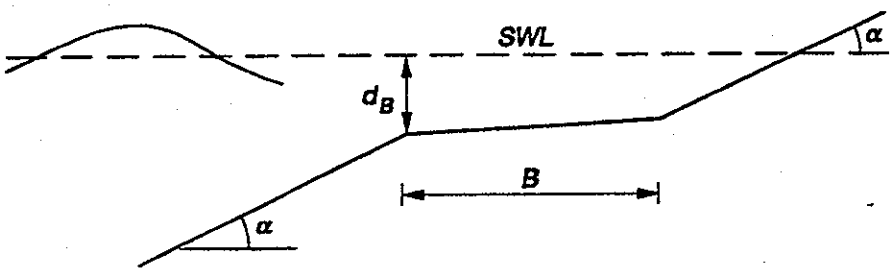
Based on a somewhat reduced data set compared to Figure VI-5-5, the uncertainty on Eqn. VI-5-6 is described by de Waal and van der Meer (1992) by assuming the factor 1.5 as a stochastic variable with a normal distribution and a coefficient of variation of 0.085.

(i) Influence of Surface Roughness on Runup. The original values for $\gamma_r$ given in Dutch publications and in the old Shore Protection Manual have been updated based on experiments including large scale tests with random waves. These factors are given in Table VI-5-3. The new $\gamma_r$ values taken from de Waal and van der Meer (1992) are valid for $1 < \xi_{op} < 3\text{-}4$. For larger $\xi_{op}$-values the $\gamma_r$ factors will slowly increase to 1.

**TABLE VI-5-3**
**Surface Roughness Reduction Factor $\gamma_r$ in Eqn. VI-5-3, Valid for $1 < \xi_{op} < 3\text{-}4$.**

| Type of Slope Surface | | | $\gamma_r$ |
|---|---|---|---|
| Smooth, concrete, asphalt | | | 1.0 |
| Smooth block revetment | | | 1.0 |
| Grass (3 cm length) | | | 0.90 - 1.0 |
| 1 layer of rock, diameter $D$, ($H_s/D$ = 1.5 - 3.0) | | | 0.55 - 0.6 |
| 2 or more layers of rock, ($H_s/D$ = 1.5 - 6.0) | | | 0.50 - 0.55 |
| Roughness elements on smooth surface (length parallel to water line = $\ell$, width = $b$, height = $h$) | | | |
| Quadratic blocks, $\ell = b$ | | | |
| $h/b$ | $b/H_s$ | area coverage | |
| 0.88 | 0.12 - 0.19 | 1/9 | 0.70 - 0.75 |
| 0.88 | 0.12 - 0.24 | 1/25 | 0.75 - 0.85 |
| 0.44 | 0.12 - 0.24 | 1/25 | 0.85 - 0.95 |
| 0.88 | 0.12 - 0.18 | 1/25 (above SWL) | 0.85 - 0.95 |
| 0.18 | 0.55 - 1.10 | 1/4 | 0.75 - 0.85 |
| Ribs | | | |
| 1.00 | 0.12 - 0.19 | 1/7.5 | 0.60 - 0.70 |

(ii) Influence of a Berm on Runup. A test program at Delft Hydraulics was designed to clarify the influence of a horizontal or almost horizontal berm on wave runup. Figure VI-5-6 shows the range of tested profiles and sea states.




| | |
|---|---|
| tan α | 1:3 , 1:4 |
| $s_{op}$ | 0.01 , 0.02 , 0.03 , 0.04 , 0.05 |
| $H_S$ (m) | 0.10 - 0.20 |
| B (m) | 0.40 , 0.60 , 1.00 |
| $d_B$ (m) | -0.08 , 0.00 , 0.08 , 0.16 |

Figure VI-5-6. Parameters in berm test program at Delft Hydraulics.

According to de Waal and van der Meer (1992) the effect of a berm can be taken into account by the following formulation of the reference Eqn. VI-5-6

$$\frac{R_{u2\%}}{H_s} = \begin{cases} 1.5\ \xi_{op}\gamma_r\gamma_b\gamma_h\gamma_\beta = 1.5\ \xi_{eq}\gamma_r\gamma_h\gamma_\beta & \text{for } 0.5 < \xi_{eq} \le 2 \\ 3.0\ \gamma_r\gamma_h\gamma_\beta & \text{for } \xi_{eq} > 2 \end{cases} \qquad \text{(VI-5-7)}$$

where $\xi_{eq}$ is the breaking wave surf similarity parameter based on an equivalent slope (see Figure VI-5-7). The berm influence factor $\gamma_b$ is defined as

$$\gamma_b = \frac{\xi_{eq}}{\xi_{op}} = 1 - r_B(1 - r_{dB}) \quad , \quad 0.6 \le \gamma_b \le 1.0 \qquad \text{(VI-5-8)}$$

where

$$r_B = 1 - \frac{\tan \alpha_{eq}}{\tan \alpha}$$

$$r_{dB} = 0.5 \left( \frac{d_B}{H_s} \right)^2 \quad , \quad 0 \le r_{dB} \le 1 \qquad \text{(VI-5-9)}$$

and the *equivalent* slope angle $\alpha_{eq}$ and the *average* slope angle $\alpha$ are defined in Figure VI-5-7.

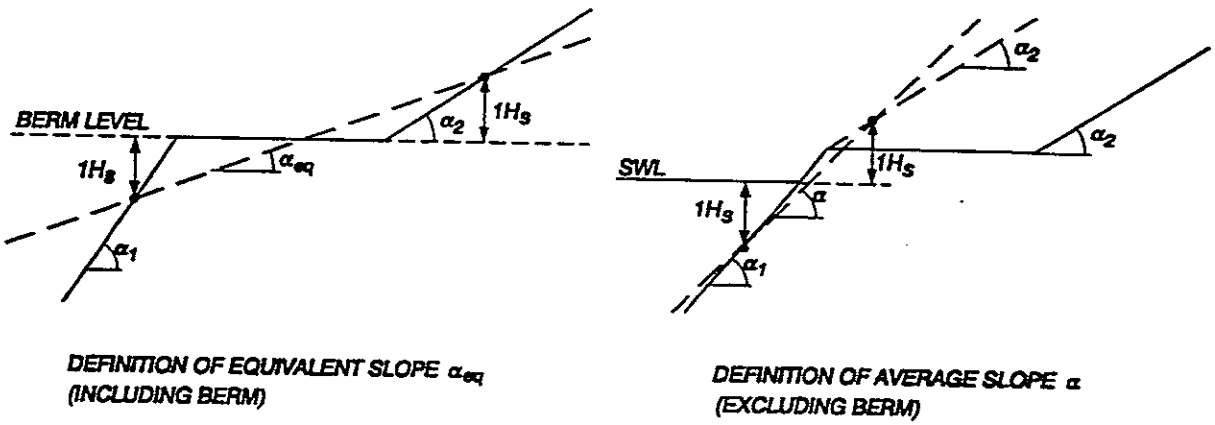



Figure VI-5-7. Definition of $\alpha_{eq}$ and $\alpha$ in Eqn. VI-5-9.

The influence of the berm can be neglected when the berm horizontal surface is positioned more than $H_s \sqrt{2}$ below SWL. If the berm horizontal surface lies higher than $d_B = H_s \sqrt{2}$ above SWL, then the runup can be set to $R_{u2\%} = d_B$ if $B/H_s \ge 2$. The berm is most effective when lying at SWL, i.e., $d_B = 0$. An optimum berm width $B$, which corresponds to $\gamma_b = 0.6$, can be determined from the formulae given by Eqns. VI-5-8 and VI-5-9.

The use of $\xi_{eq}$ in Eqn. VI-5-7 is evaluated in Figure VI-5-8 on the basis of the test program given in Figure VI-5-6, which implies $\gamma_r = \gamma_h = \gamma_\beta = 1$.

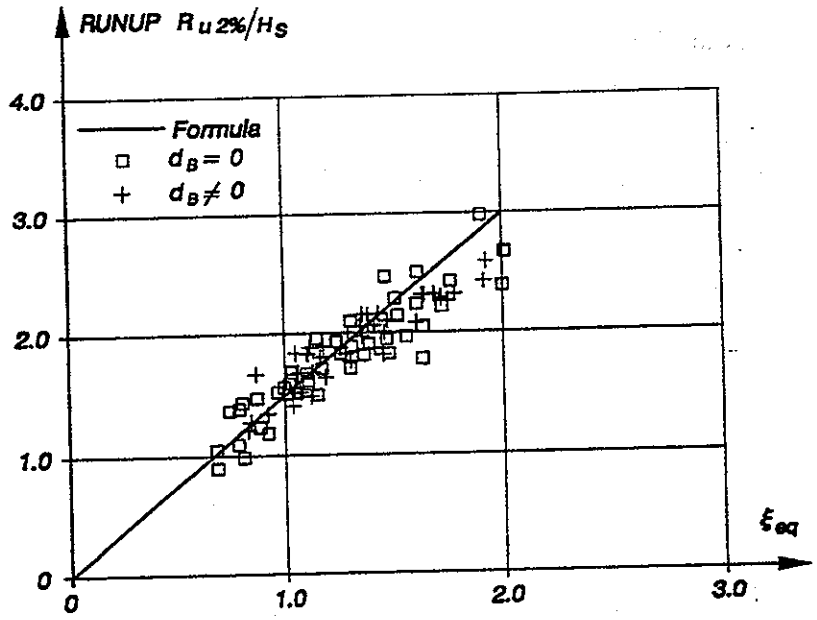


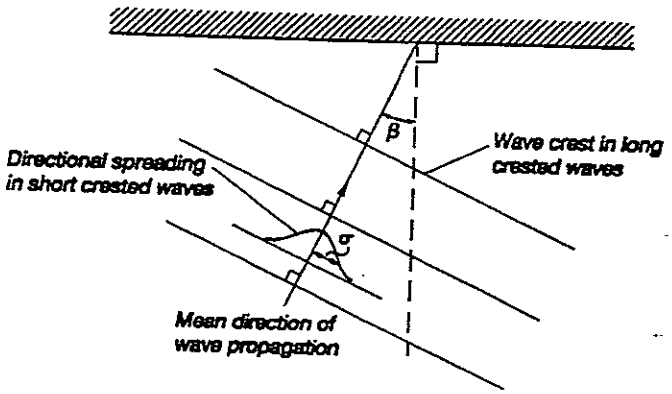

Figure VI-5-8. Evaluation of the use of $\xi_{eq}$ to account for the influence of a berm.

(iii) Influence of Shallow Water on Runup. Wave heights in Eqn. VI-5-7 are characterized by $H_s$ which provides a unique definition for deep water conditions where wave heights are Rayleigh distributed. In shallow water where some waves break before they reach the structure, the wave heights will no longer be Rayleigh distributed. According to de Waal and van der Meer, the influence factor can be estimated as

$$\gamma_h = \frac{H_{2\%}}{1.4\ H_s} \qquad \text{(VI-5-10)}$$

where the representative wave heights are specified for the water depth at the toe of the structure ($H_{2\%}/H_s$ = 1.4 for Rayleigh distributed wave heights).

(iv) Influence of Angle of Wave Attack on Runup. Both the angle of incidence and the directional spreading of the waves influence the runup. A test program for runup on smooth slopes at Delft Hydraulics, as specified in Figure VI-5-9, revealed the variations in the influence factor $\gamma_\beta$ as given by Eqn. VI-5-11 and depicted in Figure VI-5-10.




| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| tan α | 1/2.5 | | 1/4 | | 1/4 with berm | | | | |
| $s_{op}$ | 0.01 | | 0.02 | | 0.03 | | 0.04 | | 0.05 |
| $H_s$ (m) | 0.06 | | 0.12 | | | | | | |
| β° | 0 | 10 | 20 | 30 | 40 | 50 | 60 | 70 | 80 |
| σ° | 0 | 12 | 25 | 32 | 45 | | | | |

Figure VI-5-9. Test program for wave runup on smooth slopes. Delft Hydraulics, de Waal and van der Meer (1992).

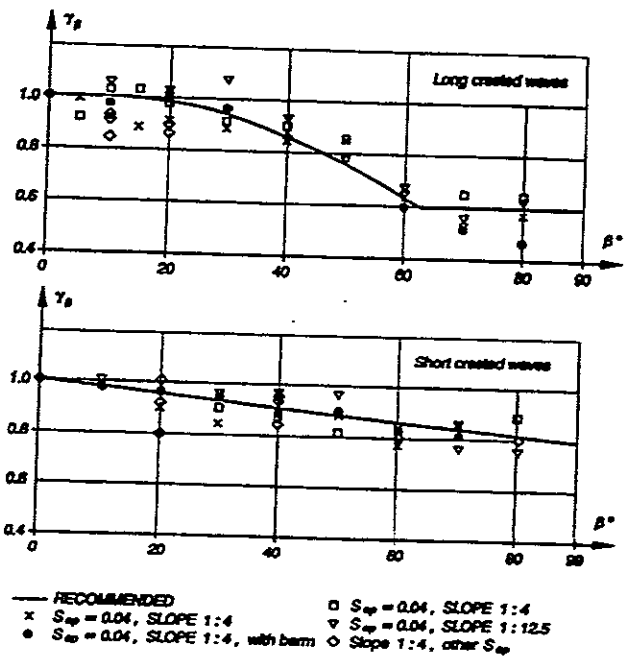



Figure VI-5-10. Influence of angle of incidence $\beta$ and directional spreading on runup on smooth slopes. Delft Hydraulics, de Waal and van der Meer (1992).

Note that $\gamma_\beta$-values larger than 1 were obtained for long-crested waves in the range $10° \le \beta \le 30°$, and that values very close to 1 were obtained for short-crested waves for $\beta$ up to 50°.

Based on the results, the following formulas for mean values of $\gamma_\beta$ were given

$$\text{Long-crested waves (mainly swell)} \quad \gamma_\beta = \begin{cases} 1.0 & \text{for } 0° \le \beta \le 10° \\ \cos(\beta - 10°) & \text{for } 10° < \beta \le 63° \\ 0.6 & \text{for } \beta > 63° \end{cases}$$

$$\text{Short-crested waves} \quad \gamma_\beta = 1 - 0.0022\,\beta \qquad \text{(VI-5-11)}$$

(b) Rock Armored Slopes, Irregular Long-Crested Head-On Waves. Runup on *rock armored impermeable and permeable slopes* was studied by Delft Hydraulics in the test program given in Table VI-5-4.

**TABLE VI-5-4**
**Test Program, van der Meer (1988).**

| Slope Angle cot $\alpha$ | Grading $D_{85}/D_{15}$ | Spectral Shape | Core Permeability | Relative Mass Density | Number of Tests | Range $H_s/\Delta D_{n50}$ | Range $s_{om}$ |
|---|---|---|---|---|---|---|---|
| 2 | 2.25 | PM | none | 1.63 | 19 | 0.8-1.6 | 0.005-0.016 |
| 3 | 2.25 | PM | none | 1.63 | 20 | 1.2-2.3 | 0.006-0.024 |
| 4 | 2.25 | PM | none | 1.63 | 21 | 1.2-3.3 | 0.005-0.059 |
| 6 | 2.25 | PM | none | 1.63 | 26 | 1.2-4.4 | 0.004-0.063 |
| 3* | 1.25 | PM | none | 1.62 | 21 | 1.4-2.9 | 0.006-0.038 |
| 4 | 1.25 | PM | none | 1.62 | 20 | 1.2-3.4 | 0.005-0-059 |
| 3 | 2.25 | narrow | none | 1.63 | 19 | 1.0-2.8 | 0.004-0.054 |
| 3 | 2.25 | wide | none | 1.63 | 20 | 1.0-2.4 | 0.004-0.043 |
| 3* | 1.25 | PM | permeable | 1.62 | 19 | 1.6-3.2 | 0.008-0.060 |
| 2 | 1.25 | PM | permeable | 1.62 | 20 | 1.5-2.8 | 0.007-0.056 |
| 1.5 | 1.25 | PM | permeable | 1.62 | 21 | 1.5-2.6 | 0.008-0.050 |
| 2 | 1.25 | PM | homogeneous | 1.62 | 16 | 1.8-3.2 | 0.008-0.059 |
| 2 | 1.25 | PM | permeable | 0.95 | 10 | 1.7-2.7 | 0.016-0.037 |
| 2 | 1.25 | PM | permeable | 2.05 | 10 | 1.6-2.5 | 0.014-0.032 |
| 2** | 1.25 | PM | permeable | 1.62 | 16 | 1.6-2.5 | 0.014-0.031 |
| 2*** | 1.25 | PM | permeable | 1.62 | 31 | 1.4-5.9 | 0.010-0.046 |

PM Pierson Moskowitz spectrum
* Some tests repeated in Delta Flume
** Foreshore 1:30
*** Low-crested structure with foreshore 1:30

The core permeability in Table VI-5-4 refers to the structures shown in details *a*, *c* and *d* of Figure VI-5-11, taken from van der Meer (1988). The figure provides definition of a notational permeability parameter *P* which is used in various formulae by van der Meer to take into account the effect of permeability on response to wave action. The value $P = 0.4$ in Figure VI-5-11, detail *b*, is not identified by tests, but instead is an estimated value.

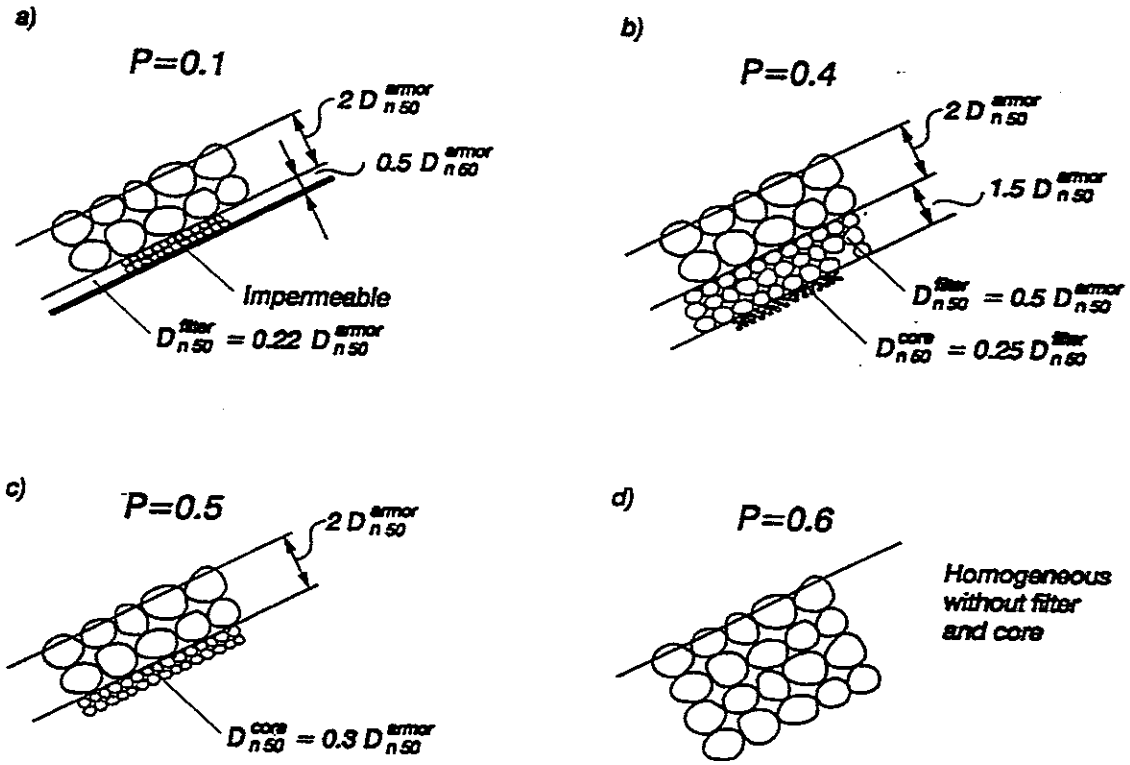



Figure VI-5-11. Notational permeability coefficients. van der Meer (1988).

The runup results from the test program described in Table VI-5-4 are presented in Figure VI-5-12.

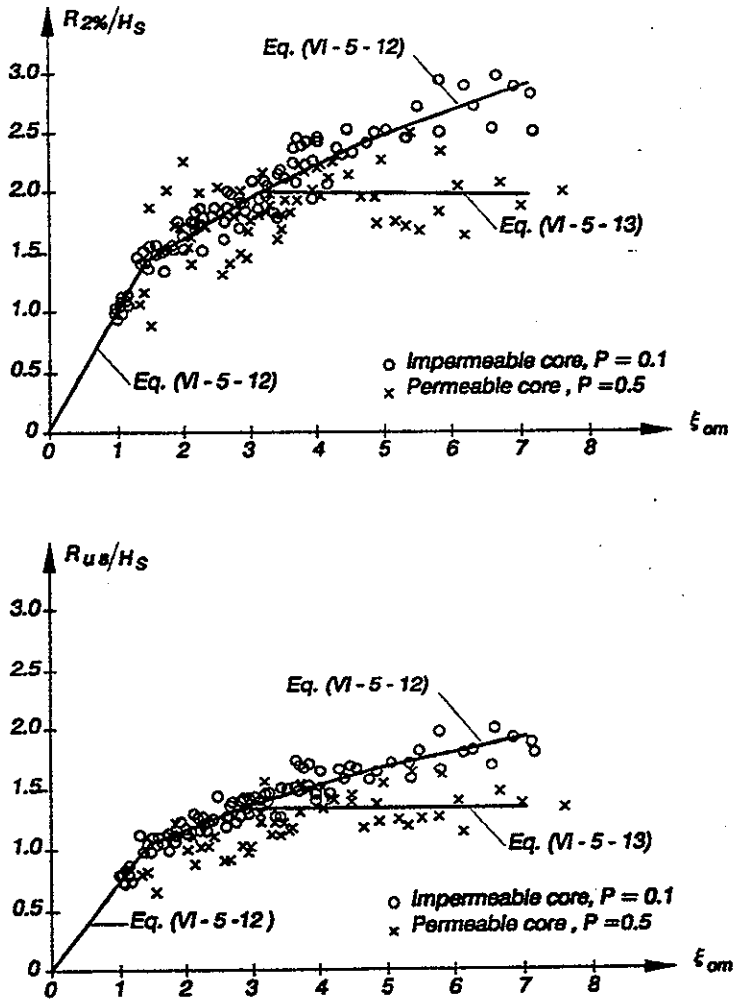



Figure VI-5-12. 2% and significant runup of irregular head-on waves on impermeable and permeable rock slopes. Delft Hydraulics (1989).

Note that $\xi_{om}$ = tan $\alpha$ / $(2\pi H_s/gT_{om}^2)^{1/2}$, where $T_{om}$ is the mean wave period, is used instead of $\xi_{op}$. By using $T_{om}$ instead of $T_{op}$ variations in the width of the wave spectrum are taken into account. The ratio $T_{om}/T_{op} = \xi_{om}/\xi_{op}$ = 0.79 - 0.87 for JONSWAP spectra and 0.71 - 0.82 for Pierson-Moskowitz spectra.

The central fit to the data for *impermeable rock slopes* was given by Delft Hydraulics (1989) as

$$\frac{R_{ui\%}}{H_s} = \begin{cases} A\,\xi_{om} & \text{for} \quad 1.0 < \xi_{om} \le 1.5 \\ B\,(\xi_{om})^C & \text{for} \quad \xi_{om} > 1.5 \end{cases} \qquad \text{(VI-5-12)}$$

The coefficients $A$, $B$ and $C$ are given in Table VI-5-5. For impermeable slopes the coefficient of variation for $A$, $B$ and $C$ is 7%. Data presented by Ahrens and Heinbaugh (1988a) for maximum runup on impermeable riprap slopes are in agreement with the data represented by Eqn. VI-5-12.

**TABLE VI-5-5**
**Coefficients in Eqns. VI-5-12 and VI-5-13 for Runup of Irregular Head-On Waves on Impermeable and Permeable Rock Armored Slopes.**

| % * | A | B | C | D ** |
|---|---|---|---|---|
| 0.1 | 1.12 | 1.34 | 0.55 | 2.58 |
| 2.0 | 0.96 | 1.17 | 0.46 | 1.97 |
| 5 | 0.86 | 1.05 | 0.44 | 1.68 |
| 10 | 0.77 | 0.94 | 0.42 | 1.45 |
| 33 (significant) | 0.72 | 0.88 | 0.41 | 1.35 |
| 50 (mean) | 0.47 | 0.60 | 0.34 | 0.82 |

* Exceedence level related to number of waves
** Only relevant for permeable slopes

Equation VI-5-12 is valid for relatively deep water in front of a structure where the wave height distribution is close to the Rayleigh distribution. Wave breaking on a foreshore results in a truncation in the runup distribution which mainly results in lower runup heights for small exceedence probability levels. However, sometimes higher runup may occur according to observations in the Delft Hydraulics tests and recent tests conducted at Texas A&M University.

(4) Wave Runup and Rundown on Permeable Slopes. With respect to runup, permeable structures are defined as structures with core material of such permeability that wave induced porous flow and fluctuations of the internal phreatic line do vary with the frequencies of the waves. The storage capacity of the structure pores results in maximum runup that is smaller than for an equivalent structure with an impermeable core.

(a) Rock Armored Slopes, Irregular Long-Crested Head-On Waves. Rock armored permeable slopes with notational permeability $P = 0.5$, as shown in detail *c* of Figure VI-5-11, were tested in irregular head-on waves by Delft Hydraulics in the program specified in Table VI-5-4. The results are shown in Figure VI-5-12, and the corresponding equation for the central fit to the data is given by

$$R_{ui\%} / H_s \begin{cases} = A\,\xi_{om} & \text{for} \quad 1.0 < \xi_{om} \le 1.5 \\ = B\,(\xi_{om})^C & \text{for} \quad 1.5 < \xi_{om} \le (D/B)^{1/C} \\ = D & \text{for} \quad (D/B)^{1/C} \le \xi_{om} < 7.5 \end{cases} \qquad \text{(VI-5-13)}$$

The coefficients $A$, $B$, $C$ and $D$ are listed in Table VI-5-5. For permeable structures the coefficient of variation for $A$, $B$, $C$ and $D$ is 12%. Tests with homogeneous rock structures with notational permeability $P = 0.6$, as shown in detail $d$ of Figure VI-5-11, showed results almost similar to the test results corresponding to $P = 0.5$ as shown in Figure VI-5-12.

Equation VI-5-13 is valid for relatively deep water conditions with wave height distributions close to a Rayleigh distribution. Wave breaking due to depth limitations in front of the structure cause truncation of the runup distribution and thereby lower runup heights for small exceedence probability levels. However, higher runup might also occur according to observations in the Delft Hydraulics tests, van der Meer and Stam (1992). The influence on runup for the shallow water conditions included in the test program given in Table VI-5-4 were investigated for the rock armored permeable slope. However, no systematic deviations from Eqn. VI-5-13 were observed.

(b) Statistical Distribution of Runup. The runup of waves with approximately Rayleigh distributed wave heights on rock armored permeable slopes with $\tan \alpha \geq 2$ were characterized by van der Meer and Stam (1992) with a best-fit two-parameter Weibull distribution as follows:

$$Prob\ (R_u > R_{up\%}) = \exp\left[-\left(\frac{R_{up\%}}{B}\right)^C\right] \qquad \text{(VI-5-14)}$$

or

$$R_{up\%} = B\ (-\ln p)^{1/C} \qquad \text{(VI-5-15)}$$

where

$R_{up\%}$ = Runup level exceeded by $p$% of the runup

$$B = H_s[0.4\ (s_{om})^{-1/4}\ (\cot \alpha)^{-0.2}] \qquad \text{(VI-5-16)}$$

$$C = \begin{cases} 3.0(\xi_{om})^{-3/4} & \text{for} \quad \xi_{om} \leq \xi_{omc} \quad \textit{(plunging waves)} \\ 0.52\ P^{-0.3}(\xi_{om})^P\sqrt{\cot \alpha} & \text{for} \quad \xi_{om} > \xi_{omc} \quad \textit{(surging waves)} \end{cases} \qquad \text{(VI-5-17)}$$

$$\xi_{omc} = \left(5.77\ P^{0.3}\sqrt{\tan \alpha}\right)^{[1/(P+0.75)]} \qquad \text{(VI-5-18)}$$

$$s_{om} = \frac{2\pi H_s}{gT_{om}^2}.$$

$P$ = notational permeability, see Figure VI-5-11.

It follows from Eqn. VI-5-15 that the scale parameter $B$ is equal to $Ru_{37\%}$ (ln $p$ = -1 for $p$ = 0.37). If the shape parameter $C$ is equal to 2, then Eqn. VI-5-14 becomes a Rayleigh distribution. The uncertainty on $B$ corresponds to a coefficient of variation of 6% for $P < 0.4$ and 9% for $P \geq 0.4$.

*Rundown* on rock slopes in the Delft Hydraulics test program listed in Table VI-5-4 gave the following relationship which includes the effect of structure permeability $P$ (see Figure VI-5-11).

$$\frac{R_{d2\%}}{H_s} = 2.1\sqrt{\tan\alpha} - 1.2\,P^{0.15} + 1.5\,e^{-(60\,s_{om})} \qquad \text{VI-5-19)}$$

**Table VI-6-18**
Partial safety factors for run-up, rock armored slopes. De Waal and van der Meer (1992). Design without model tests.

Design equation

For $\zeta_m = (\cot\alpha)^{-1}(s_{om})^{-0.5} \leq 1.5 \quad : \quad R_u/H_s = a\,\zeta_m$

$$G = \frac{1}{\gamma_Z}\hat{R}_u\,\hat{a}^{-1}\,(cot\,\hat{\alpha})(\hat{s}_{om})^{0.5} - \gamma_H\hat{H}_s^T \qquad \text{(VI-6-57)}$$

| | $\sigma'_{FH_S} = 0.05$ | | $\sigma'_{FH_S} = 0.2$ | |
|---|---|---|---|---|
| $P_f$ | $\gamma_H$ | $\gamma_Z$ | $\gamma_H$ | $\gamma_Z$ |
| 0.01 | 1.7 | 1.04 | 2.0 | 1.00 |
| 0.05 | 1.4 | 1.06 | 1.6 | 1.02 |
| 0.10 | 1.3 | 1.04 | 1.4 | 1.06 |
| 0.20 | 1.2 | 1.02 | 1.3 | 1.00 |
| 0.40 | 1.0 | 1.08 | 1.1 | 1.00 |

For $\zeta_m = (\cot\alpha)^{-1}(s_{om})^{-0.5} > 1.5 \quad : \quad R_u/H_s = b\,(\zeta_m)^c$

$$G = \frac{1}{\gamma_Z}\hat{R}_u\,\hat{b}^{-1}[\cot\hat{\alpha}\,(\hat{s}_{om})^{0.5}]^c - \gamma_H\hat{H}_s^T \qquad \text{(VI-6-58)}$$

| | $\sigma'_{FH_S} = 0.05$ | | $\sigma'_{FH_S} = 0.2$ | |
|---|---|---|---|---|
| $P_f$ | $\gamma_H$ | $\gamma_Z$ | $\gamma_H$ | $\gamma_Z$ |
| 0.01 | 1.5 | 1.08 | 1.8 | 1.02 |
| 0.05 | 1.3 | 1.06 | 1.4 | 1.10 |
| 0.10 | 1.2 | 1.06 | 1.3 | 1.08 |
| 0.20 | 1.1 | 1.08 | 1.2 | 1.06 |
| 0.40 | 1.0 | 1.06 | 1.0 | 1.10 |

For permeable structures, $P > 0.4$, the upper limit of $R_u$ is given by $R_u/H_s = d$.

| | |
|---|---|
| $\alpha$ | Slope angle |
| $s_{om}$ | Wave steepness, $H_s/L_{om}$ |
| $L_{om}$ | Deep water wave length corresponding to mean wave period |
| $R_u$ | Wave run-up |
| $H_s^T$ | Significant wave height with return period $T$ |
| $P$ | Notational permeability, cf. Figure VI-5-11 |

Values of $a$, $b$, $c$ & $d$ coefficients.

| exceedence Prob. (%) | a | b | c | d |
|---|---|---|---|---|
| 0.1 | 1.12 | 1.34 | 0.55 | 2.58 |
| 2 | 0.96 | 1.17 | 0.46 | 1.97 |
| Significant | 0.72 | 0.88 | 0.41 | 1.35 |

22-141 50 SHEETS
22-142 100 SHEETS
22-144 200 SHEETS
AMPAD

# WAVE OVERTOPPING

# EXAMPLE PROBLEM VI-7-6

FIND: Estimate the average overtopping discharge rate for the given wave, water level, and structure geometry.

GIVEN: An impermeable structure with a smooth slope of 1 on 2.5 is subjected to waves having a deepwater significant height $H_o' = 1.5$ m and a period $T = 8$ sec. Water depth at the structure toe is $d_{toe} = 3.0$ m. The crest elevation, $R_c$ is 1.5 m above the design, SWL. Onshore winds of 35 knots are assumed.

SOLUTION: Table VI-5-7 lists two models applicable for this example. We shall determine the average overtopping discharge rate, $q$ ($m^3$/sec per meter) from two formulas, namely Owen (1980, 1982) and van der Meer and Janssen (19 Both require knowledge of the wave height, $H_s$ at the toe of the structure.

Assume wave direction is shore normal to the structure

(1) Wave height, $H_s$ at structure toe

(a) Linear wave theory

$$L_o = \frac{g}{2\pi} T^2 = 1.56(8)^2 = 99.92 \text{ m}$$

$$\frac{d}{L_o} = \frac{3.0}{99.92} = 0.0300$$

$$\frac{H}{H_o'} = 1.125 \quad \text{(Table C-1, SPM, 1984)}$$

Assume $H = H_s = 1.125(1.5) = 1.69$ m (non-breaking)

(b) Irregular waves (GODA method, see ACES 1.07)

$$H_s = 1.6 \text{ m} \quad \text{(checks okay)}$$

(c) Use $(H_s)_{toe} = 1.69$ m (conservative)

(2) Table VI-5-8, Owen (1980, 1982)

Using Eqn VI-5-22

$$\frac{q}{g H_s T_m} = a \exp\left(-b \frac{R_c}{H_s} \sqrt{\frac{S_{om}}{2\pi}} \frac{1}{\gamma_r}\right)$$

requires knowledge of $T_m$. As discussed on p. VI-5-18, the relation between $T_m$ and $T_p$ can be estimated from

EXAMPLE PROBLEM VI-7-6 (continued)

JONSWAP spectra, $T_m/T_p = 0.79 - 0.87$
or Pierson-Moskowitz spectra, $T_m/T_p = 0.71 - 0.82$
in deep water
here, take $T_m = 0.8\,T_p$ so that $T_m = 6.4$ sec.

Therefore $$L_{om} = \frac{g}{2\pi} T_m^2 = 1.56(6.4)^2 = 63.9 \text{ m}$$

and $$S_{om} = \frac{H_s}{L_{om}} = \frac{1.69}{63.9} = 0.02645$$

Now from coefficients table for smooth slopes

| Slope | a | b | |
|---|---|---|---|
| 1.2 | 0.0130 | 22 | |
| 1.25 | 0.0145 | 27 | ← by linear interpolation |
| 1.3 | 0.0160 | 32 | |

Therefore, with $\gamma_r = 1.0$

$$\frac{q}{(9.81)(1.69)(6.4)} = 0.0145 \exp\left(-27 \frac{1.5}{1.69}\sqrt{\frac{0.02645}{2\pi}}\,\frac{1}{1.0}\right)$$

$$= 0.0145 \exp(-1.5548)$$

$$= 0.0145(0.21122) = 0.003063$$

or $$q = (9.81)(1.69)(6.4)(0.003056)$$

$$= (106.10)(0.003056)$$

$$\underline{q = 0.324 \text{ m}^3/\text{sec per meter width}}$$

(3) Table VI-5-11, van der Meer and Janssen (1995)

The data employed to develop Equation VI-5-24 is shown in Figure VI-5-15 (top) when $\xi_{op} < 2$. This is a very comprehensive data set showing the 95 % confidence bands in the data.

Now, using $T_p = 8$ sec gives

$$S_{op} = \frac{H_s}{L_{op}} = \frac{1.69}{99.92} = 0.01691$$

so that

$$\xi_{op} = \frac{0.4}{\sqrt{0.0169}} = \frac{0.4}{0.12995} = 3.07$$

Therefore $\xi_{op} > 2$
so that Equation VI-5-25 (see Figure VI-5-15 (bottom)) governs.

EXAMPLE PROBLEM VI-7-6 (continued)

Using Eqn VI-5-25

$$\frac{q}{\sqrt{9.81\cdot(1.69)^3}} = 0.2\exp\left(-2.6\,\frac{1.5}{1.69}\cdot\frac{1}{(1.0)(1.0)(1.0)(1.0)}\right)$$

with all the reduction factors as unity.

or

$$\frac{q}{6.881} = 0.2\exp(-2.30769)$$

$$q = 6.881\,(0.019898)$$

$$\underline{q = 0.136\ m^3/sec\ per\ meter\ width}$$

This result is considerably lower than that found from Table VI-5-8 by Owen (1980, 1982). However, a check of this result by examining the data scatter in Figure VI-5-15 provides some insight.

The horizontal axis is

$$\frac{R_c}{H_s}\cdot\frac{1}{\gamma_r\gamma_b\gamma_h\gamma_\beta} = \frac{1.5}{1.69}\,\frac{1}{1.0} = 0.89$$

and the range covered on the vertical axis is

$$\frac{q}{\sqrt{g H_s^3}} = \begin{cases} 5*10^{-2} \\ 1*10^{-2} \end{cases} \quad 0.019898 \leftrightarrow 2*10^{-2}\ (\text{mean})$$

Therefore

$$q = \begin{cases} 0.344 \\ 0.136 \leftarrow \text{mean} \\ 0.069 \end{cases}\ m^3/sec\ per\ meter$$

The range of q at the 95% confidence level is about 0.07 to 0.34 $m^3/$. The result from Table VI-5-8 with $q = 0.32\ m^3/sec$ now seems reasonable.

This example problem is identical to Ex. Prob 8 in the SPM (1984) where $\bar{Q} = 0.3\ m^3/sec$ per meter was found. This result includes a factor for wind that is not included above.

Because of the large range of possibility variability in the time-average, overtopping discharge rate, taking for design

$$q = 0.3\ m^3/sec\ per\ meter\ width$$

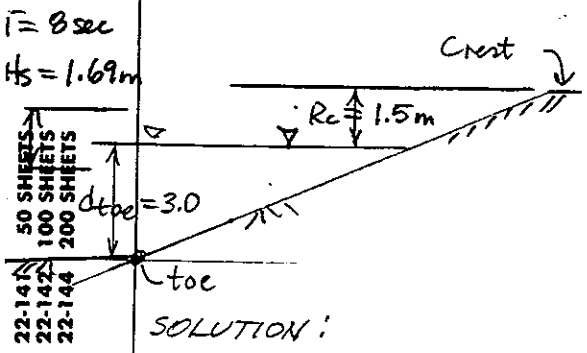
This level is dangerous for vehicles, pedestrians and the safety of structures as illustrated in Table VI-5-6 so that raising the crest elevation should be considered

# EXAMPLE PROBLEM VI-7-7

FIND: a. Estimate the overtopping volumes of individual waves and distributions for the given wave, water level and structure geometry.

b. What effect does the structure permeability have on the results?

GIVEN: The identical conditions of Example Problem VI-7-6 (see sketch)




impermeable, smooth

$\cot\alpha = 2.5$

$H_o' = 1.5$ m

$T_p = 8$ sec

$d_{toe} = 3.0$ m

$R_c = 1.5$ m above design SWL

$\theta = 0°$ (shore normal waves)

SOLUTION:

From Example Problem VI-7-6

$$q = 0.3 \, m^3/sec \text{ per meter}$$

is the average overtopping discharge rate for waves with $(H_s)_{toe} = 1.69$ m and $T_m = 6.4$ sec

Equations VI-5-30 (or VI-5-31) with coefficient B (Eqn VI-5-32) depend on $P_{ow}$, probability of overtopping per incoming wave.

(1) Rayleigh distribution for runup on smooth, impermeable slopes

Assuming the runup levels follow a Rayleigh distribution, Equation (VI-5-33) gives for the probability of overtopping per incoming wave,

$$P_{ow} = \exp\left[-\left(\frac{1.5}{C(1.69)}\right)^2\right]$$

and Equation (VI-5-34) gives:

$$C = 0.21 \, \xi_{eq} \, \gamma_r \, \gamma_h \, \gamma_B$$

From Example Problem VI-7-6, taking

$$\xi_{eq} = \xi_{op} = 3.07 \qquad (\text{i.e. } \gamma_b = 1, \text{ no berm})$$

and all other reduction factors of unity gives

$$C = 2.437$$

and

$$P_{ow} = \exp\left[-\left(\frac{1.5}{2.437(1.69)}\right)^2\right]$$

$$P_{ow} = 0.88 = \frac{\text{number of overtopping waves}}{\text{number of incoming waves}}$$

This large percentage is due to the relatively low crest elevation of the structure.

EXAMPLE PROBLEM VI-7-7 (continued)

(2) Other distributions

As shown in Example Problem VI-7-4, the relation between

$$\frac{R_{u2\%}}{R_{us}} = \frac{8.55}{5.01} = 1.71$$

for smooth, impermeable slopes is much different than the Rayleigh distribution for wave heights where $H_{0.02}/H_s = 1.398$. Until further research is conducted, however, it must be assumed that wave runup on smooth, impermeable slopes can be approximated by the Rayleigh distribution.

(a) Overtopping volumes of individual waves, $V$

From Eqn. VI-5-32

$$B = 0.84 \frac{6.4\,(0.3)}{0.88}$$

$$B = 1.833$$

as the scale factor for the one-parameter, Weibull distribution given by Eqn VI-5-31, so that

$$V = 1.833\left(-\ln[p_>]\right)^{4/3}$$

where $p_>$ is the probability of an <u>individual</u> wave overtopping volume (per unit width) exceeding the specified overtopping volume, $V$ (per unit width)

for some representative probabilities of exceedance for individual waves,

| $p_>$ | $V$ $m^3/m$ | Ranks |
|---|---|---|
| 0.5 | 1.12 | |
| 0.135 | 4.62 | |
| 0.10 | 5.57 | |
| 0.05 | 7.92 | |
| 0.02 | 11.29 | |
| 0.01 | 14.04 | |
| 0.001 | 24.11 | |

The maximum overtopping volume per unit width, $V_{max}$ produced by one wave can be estimated from Eqn VI-5-35

$$V_{max} = 1.833\,(\ln N_{ow})^{4/3}$$

which depends on storm duration, $t$

Assuming $T_m = 6.4$ sec over the storm duration, t and $P_{ow} = 0.88$

| t hours | $N_w$ | $N_{ow}'$ | $V_{max}$ $m^3/m$ | Remarks |
|---|---|---|---|---|
| 1 | 563 | 495 | 20.9 | |
| 2 | 1125 | 990 | 24.7 | Similar to $p_s = 0.001$ |
| 5 | 2813 | 2475 | 28.4 | |
| 10 | 5630 | 4954 | 31.8 | |
| 15 | 8438 | 7425 | 33.3 | |
| 20 | 11250 | 9900 | 35.3 | |
| 24 | 13500 | 11880 | 36.3 | Gives $p_s \doteq 0.0001$ |

Storm surge hydrographs with varying design water levels and accompanying wave condition variability during the storm will modify these results, considerably.

(b) Effect of structure permeability

No data exists for permeable, straight and bermed slopes as summarized in Table VI-5-7. to estimate average wave overtopping discharge rates. However, as shown in Example VI-7-2 for a rough, <u>impermeable</u> slope

(Impermeable) $R_{u2\%} = 4.70\,m$ (Eqn VI-5-3) $\gamma = 0.55$

$= 3.56\,m$ (Eqn VI-5-6) $\gamma = 0.55$

$= 3.53\,m$ (Eqn VI-5-12) (Table VI-5-5) $P = 0.1$

using various models. And, as shown in Example VI-7-3 for rock-armored, <u>permeable</u> slopes ($P = 0.5$)

(Permeable) $R_{u2\%} = 3.53\,m$ (Eqn VI-5-13) (Table VI-5-5)

The runup elevation at the 2% exceedance level for this example is about the same for permeable and impermeable slopes.

Statistical distributions for wave runup on rock-armored, permeable slopes are discussed in VI-5-2-b.(4)(b) and best-fit, by a two-parameter Weibull distribution (Eqn VI-5-14). Structure permeability absorbs the higher frequency runup components to modify the distribution from that given by the Rayleigh distribution.

Research is needed for the probability distribution of wave overtopping per incoming waves on <u>permeable slopes</u>



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(2) Average Overtopping Discharge Formulas. Formulae for overtopping are empirical because they are fitted to hydraulic model test results for specific breakwater geometries. In general the average overtopping discharge per unit length of structure, $q$, is a function of the standard parameters:

$$q = \textit{function}\ (H_s,\ T_{op},\ \sigma,\ \beta,\ R_c,\ h_s,\ g,\ \textit{structure geometry})$$

where

- $H_s$ - significant wave height
- $T_{op}$ - wave period associated with the spectral peak in deep water (alternately $T_{om}$)
- $\sigma$ - spreading of short crested waves
- $\beta$ - angle of incidence for the waves
- $R_c$ - freeboard
- $h_s$ - water depth in front of structure
- $g$ - gravitational acceleration

Two types of mathematical formulatons (models) for dimensionless overtopping dominate the literature, i.e.,

$$Q = a\ e^{-(bR)} \qquad \text{(VI-5-20)}$$

and

$$Q = a\ R^{-b} \qquad \text{(VI-5-21)}$$

where $Q$ is a dimensionless average discharge per meter and $R$ is a dimensionless freeboard. Table VI-5-7 gives an overview of the models used in recent overtopping formulae along with the associated definitions for dimensionless discharge and freeboard.

The fitted coefficients $a$ and $b$ in Eqns. VI-5-20 and VI-5-21 are specific to the front geometry of the structure and must be given in tables. So far no general model for the influence of front geometry exists except for rubble mound slopes with a seawall (Pedersen 1996), in which case the front geometry (described by the front berm width $B$, berm crest height $A_c$, and slope angle $\alpha$), as well as $R_c$, enters into $R$.

Some formulae take into account the reduction in overtopping due to slope surface roughness, berm, shallow water, angle of wave incidence and shortcrestedness, and specific front geometries by dividing $R$ by the respective reduction coefficients: $\gamma_r$ (Table VI-5-3), $\gamma_b$ (Eqn. VI-5-8), $\gamma_k$ (Eqn. VI-5-10), $\gamma_\beta$ (Eqns. VI-5-11, VI-5-26, VI-5-29), and $\gamma_s$ (Table VI-5-13).

Goda (1985) presented diagrams for wave overtopping of vertical revetments and block mound seawalls on bottom slopes of 1:10 and 1:30. The diagrams are based on model tests with irregular long-crested head-on waves and express average discharge per meter width as a function of wave height, wave steepness, freeboard, and water depth.

(a) Sloping Structures. Tables VI-5-8 to VI-5-12 pertain to sloping-front structures.

**TABLE VI-5-8**
**Overtopping Formula by Owen (1980, 1982).**

Straight and bermed impermeable slopes, Figures VI-5-14 *a* and *b*.
Irregular, head–on waves.

$$\frac{q}{g\,H_s\,T_{om}} = \mathrm{a}\ exp\left(-\mathrm{b}\ \frac{R_c}{H_s}\sqrt{\frac{s_{om}}{2\pi}}\ \frac{1}{\gamma_r}\right) \qquad \text{(VI–5–22)}$$

Coefficients in Eqn. VI-5-22
Straight smooth slopes.
Non-depth limited waves.

| Slope | a | b |
|---|---|---|
| 1 : 1 | 0.008 | 20 |
| 1 : 1.5 | 0.010 | 20 |
| 1 : 2 | 0.013 | 22 |
| 1 : 3 | 0.016 | 32 |
| 1 : 4 | 0.019 | 47 |

Surface roughness reduction factor $\gamma_r$.
Updated $\gamma_r$-values are given in Table VI-5-3.

| | |
|---|---|
| Smooth impermeable (including smooth concrete and asphalt) | 1.0 |
| One layer of stone rubble on impermeable base | 0.8 |
| Gravel, gabion mattresses | 0.7 |
| Rock rip-rap with thickness greater than $2\,D_{n50}$ | 0.5 - 0.6 |

Coefficients in Eqn. VI-5-22
Bermed smooth slopes.
Non-depth limited waves.

| Slope | $h_B$ (m) | $B$ (m) | a·10⁴ | b |
|---|---|---|---|---|
| 1 : 1 | −4.0 | 10 | 64 | 20 |
| 1 : 2 | | | 91 | 22 |
| 1 : 4 | | | 145 | 41 |
| 1 : 1 | −2.0 | 5 | 34 | 17 |
| 1 : 2 | | | 98 | 24 |
| 1 : 4 | | | 159 | 47 |
| 1 : 1 | −2.0 | 10 | 48 | 19 |
| 1 : 2 | | | 68 | 24 |
| 1 : 4 | | | 86 | 46 |
| 1 : 1 | −2.0 | 20 | 8.8 | 15 |
| 1 : 2 | | | 20 | 25 |
| 1 : 4 | | | 85 | 50 |
| 1 : 1 | −2.0 | 40 | 3.8 | 23 |
| 1 : 2 | | | 5.0 | 26 |
| 1 : 4 | | | 47 | 51 |
| 1 : 1 | −1.0 | 5 | 155 | 33 |
| 1 : 2 | | | 190 | 37 |
| 1 : 4 | | | 500 | 70 |
| 1 : 1 | −1.0 | 10 | 93 | 39 |
| 1 : 2 | | | 340 | 53 |
| 1 : 4 | | | 300 | 80 |
| 1 : 1 | −1.0 | 20 | 75 | 46 |
| 1 : 2 | | | 34 | 50 |
| 1 : 4 | | | 39 | 62 |
| 1 : 1 | −1.0 | 40 | 12 | 49 |
| 1 : 2 | | | 24 | 56 |
| 1 : 4 | | | 1.5 | 63 |
| 1 : 1 | 0 | 10 | 97 | 42 |
| 1 : 2 | | | 290 | 57 |
| 1 : 4 | | | 300 | 80 |

**TABLE VI-5-11**
**Overtopping Formula by van der Meer and Janssen (1995)**

Straight and bermed impermeable slopes including influence of surface roughness, shallow foreshore, oblique, and short crested waves, Figures VI-5-14a and VI-5-14b.

$\underline{\xi_{op} < 2}$

$$\frac{q}{\sqrt{g\,H_s^3}}\sqrt{\frac{s_{op}}{\tan\alpha}} = 0.06\,exp\left(-5.2\,\frac{R_c}{H_s}\,\frac{\sqrt{s_{op}}}{\tan\alpha}\,\frac{1}{\gamma_r\,\gamma_b\,\gamma_h\,\gamma_\beta}\right) \qquad \text{(VI-5-24)}$$

application range: $0.3 < \frac{R_c}{H_s}\,\frac{\sqrt{s_{op}}}{\tan\alpha}\,\frac{1}{\gamma_r\,\gamma_b\,\gamma_h\,\gamma_\beta} < 2$

Uncertainty: Standard deviation of factor 5.2 is $\sigma = 0.55$ (See Figure VI-5-15).

$\underline{\xi_{op} > 2}$

$$\frac{q}{\sqrt{g\,H_s^3}} = 0.2\,exp\left(-2.6\,\frac{R_c}{H_s}\,\frac{1}{\gamma_r\,\gamma_b\,\gamma_h\,\gamma_\beta}\right) \qquad \text{(VI-5-25)}$$

Uncertainty: Standard deviation of factor 2.6 is $\sigma = 0.35$ (See Figure VI-5-15).

The reduction factors references are

$\gamma_r$ Table VI-5-3

$\gamma_b$ Eqn. VI-5-8

$\gamma_h$ Eqn. VI-5-10

$$\left.\begin{array}{l} \text{Short crested waves} \\ \gamma_\beta = 1 - 0.0033\,\beta \\ \text{Long crested waves (swell)} \\ \gamma_\beta = \begin{cases} 1.0 & \text{for } 0^\circ \le \beta \le 10^\circ \\ \cos^2(\beta - 10^\circ) & \text{for } 10^\circ < \beta \le 50^\circ \\ 0.6 & \text{for } \beta > 50^\circ \end{cases} \end{array}\right\} \qquad \text{(VI-5-26)}$$

The minimum value of any combination of the $\gamma$-factors is 0.5.

Figure VI-5-15 shows the data basis for Eqns. VI-5-24 and VI-5-25 which includes the data of Owen (1980, 1982) for straight slopes, data of Führböter, et al. (1989) and various data sets of Delft Hydraulics. It is seen that Eqn. VI-5-24 contains some bias for small values of $q$.

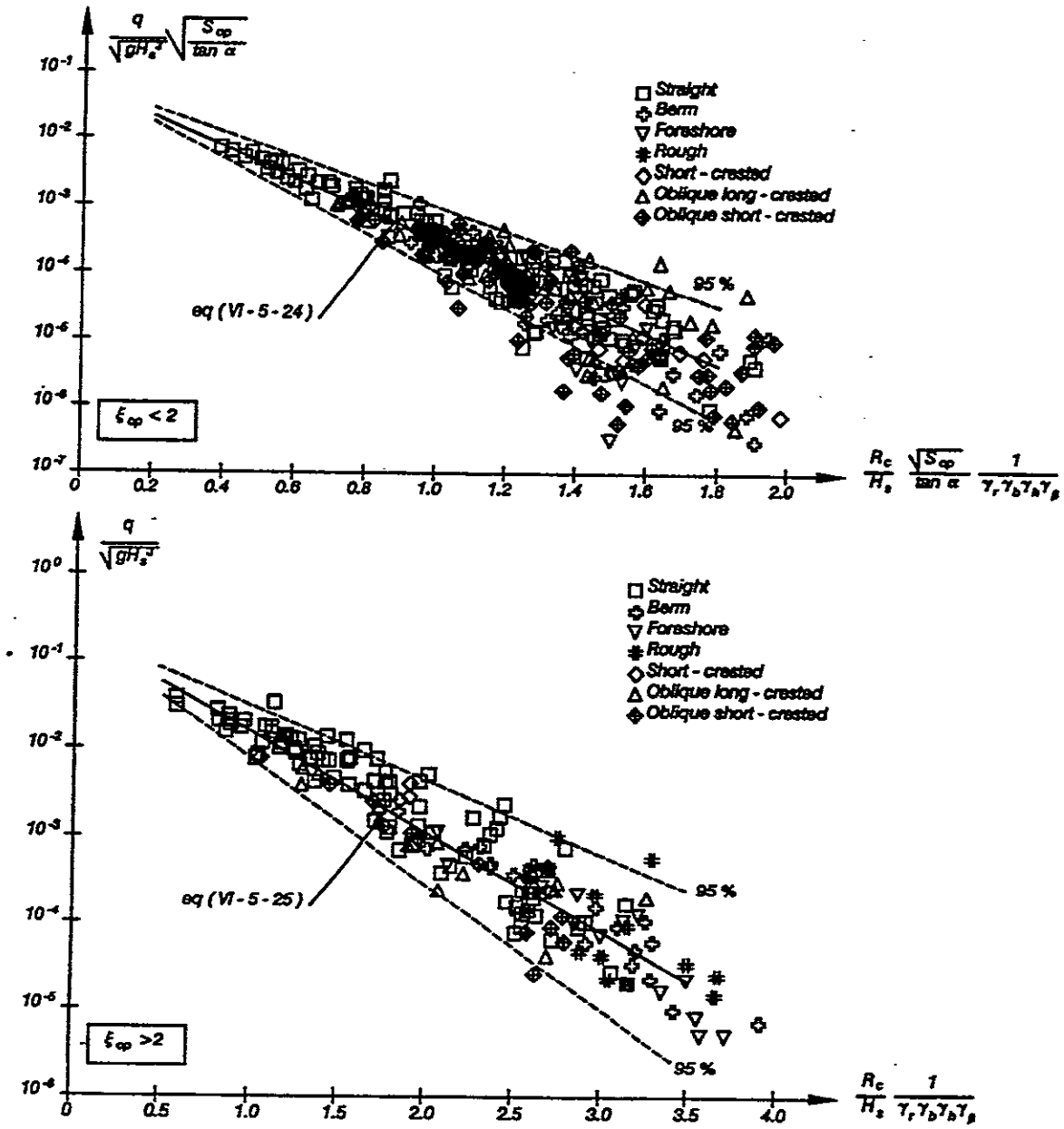


Figure VI-5-15. Wave overtopping data as basis for Eqns. VI-5-24 and VI-5-25. Fitted mean and 95% confidence bands. van der Meer and Janssen (1995).

(3) Overtopping Volumes of Individual Waves. The average overtopping discharge $q$ provides no information about the discharge intensity of the individual overtopping waves. However, such information is important because most damaging impacts on persons, vehicles, and structures are caused by overtopping of large single waves. The overtopping volume per wave has been recorded in model tests and it was found that the probability distribution function for overtopping volume per wave per unit width ($V$ m$^3$/m) follows a Weibull distribution as given in Eqn. VI-5-30 (Franco, et al. 1994; van der Meer and Jansson 1995).

$$prob\ (v > V) = \exp\left[-\left(\frac{V}{B}\right)^{3/4}\right] \qquad \text{(VI-5-30)}$$

or

$$V = B\left(-\ln[prob(v > V)]\right)^{4/3} \qquad \text{(VI-5-31)}$$

with

$$B = 0.84\,\frac{T_m q}{P_{ow}} \qquad \text{(VI-5-32)}$$

and

$prob(v > V)$ - probability of individual wave overtopping volume per unit width, $v$, being larger than the specified overtopping volume per unit width, $V$
$T_m$ - average wave period (in units of seconds)
$q$ - average overtopping discharge per unit width (in units of m$^3$/s per m)
$P_{ow}$ - probability of overtopping per incoming wave [$= N_{ow} / N_w$ ]
$N_{ow}$ - number of overtopping waves
$N_w$ - number of incoming waves

If the runup levels follow a Rayleigh distribution, the probability of overtopping per incoming wave can be estimated as

$$P_{ow} = \exp\left[-\left(\frac{R_c}{c\,H_s}\right)^2\right] \qquad \text{(VI-5-33)}$$

where

*For sloping structure, irregular waves:*

$c = 0.81\ \xi_{eq}\ \gamma_r\ \gamma_k\ \gamma_\beta$ *with a maximum of* $c = 1.62\ \gamma_r\ \gamma_k\ \gamma_\beta$

*For vertical wall structure, irregular, impermeable, long-crested, nonbreaking, head-on waves:* (VI-5-34)

$c = 0.91$

and

$R_c$ - structure crest height relative to still water level
$H_s$ - significant wave height

A first estimate of the maximum overtopping volume per unit width produced by one wave out of the total number of overtopping waves can be calculated using the expression

$$V_{max} = B\ (\ln N_{ov})^{4/3} \qquad \text{(VI-5-35)}$$

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# Armor Layer Stability

DR Basco
03/23/01

(5) Practical Formulae for Hydraulic Stability of Armor Layers. 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. Nevertheless, armor stability formulae should be applied only for conceptual design, and the uncertainty of the formulae should be considered. When the formulae do not cover the actual range of structure geometries and sea states, preliminary designs should be model tested before actual construction. Major structures should always be tested in a physical model.

Below are some of the factors by which armor stability formulae can be classified:

- o Type of armor unit
- o Deep or shallow water wave conditions
- o Armor layers crest level relative to wave run-up and still water level
- o Structures with and without superstructure

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 height 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.

Overtopping affects the armor stability. When the crest is lower than the run-up 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 nonovertopped structures. With respect to armor stability it is common to distinguish between

- o Non-overtopped or marginally overtopped structures
- o Low-crested structures, i.e., overtopped structures but with crest level above still water level
- o Submerged structures, i.e., the crest level is below still water level

The remainder of this section presents armor layer stability formulae for use in designing coastal structures. These stability formulae can be used in the context of reliability based design using the *partial safety factors* given in the tables of Chapter VI-6-6 (Partial Safety Factor System for Implementing Reliability in Design). Guidance for designing structure cross-sections is given in Section VI-5-3-e (Design of Structure Cross-Section), and complete design examples for specific structure types are given in Chapter VI-7 *(Design of Specific Project Elements)*.

(a) Structure Trunk Stability. Stability formulae for *front slope armor* on structure trunks are presented in tables as outlined below:

| Armor Unit | Non-Overtopped | Overtopped | Submerged |
|---|---|---|---|
| Rock | Tables VI-5-22/23 | Tables VI-5-24/26 | Tables VI-5-25/26 |
| Concrete Cubes | Table VI-5-29 | | |
| Tetrapods | Table VI-5-30 | | |
| Dolosse | Table VI-5-31 | | |
| ACCROPODES® | Tables VI-5-32/33 | | |
| CORE-LOC® | Table VI-5-34 | | |
| Tribars | Table VI-5-36 | | |

Information on *rear side armor stability* is given in Table VI-5-28. A formula for stability of *reef breakwater* is presented in Table VI-5-34. A formula for stability of *armor in front of a vertical wall* is presented in Table VI-5-35. Rubble-mound structure head stability is given in Tables VI-5-37/38.

*Parapet walls* are placed on top of rubble mound structures in order to reduce overtopping by deflecting the uprushing waves back into the sea. This generally reduces the front slope armor stability. A low wall behind a wide front armor berm will hardly affect the armor stability (see Figure VI-5-36a). On the other hand a high wall with a relatively deep foundation situated behind a narrow front armor berm will significantly reduce the armor stability (see Figure VI-5-36b).

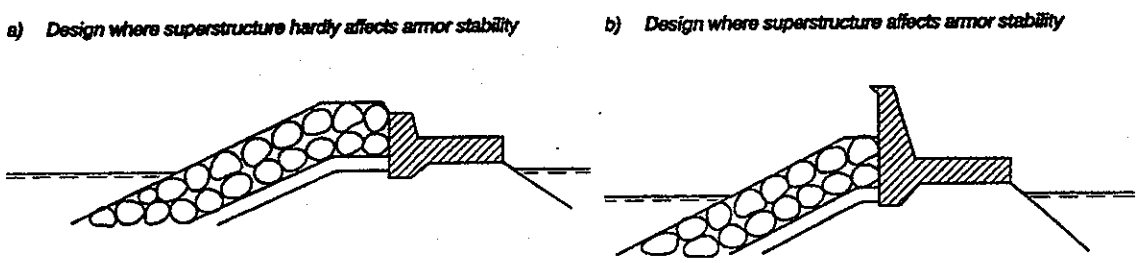


Figure VI-5-36. Illustration of superstructure designs causing insignificant and significant reduction in front slope armor stability.

No generally applicable formulae are available for reduction in front slope armor stability caused by parapet walls.

# EXAMPLE PROBLEM VI-7-10

FIND: The weight of uniform-sized armor stone placed on an impermeable revetment slope with non-overtopping waves.

GIVEN: An impermeable structure (revetment) has a slope of 1 on 2.5 and is subjected to a design, significant wave height $H_s = 2.0$ m (6.6 ft) measured at a gage located in a depth, $d = 4.5$ m (14.8 ft). Design wave peak period, $T_p = 8$ sec. Design depth at structure toe at high water is $d_{toe} = 3.0$ m (9.8 ft)

SOLUTION: From Example Problem VI-7-1, linear wave theory gives the wave height due to shoaling at the structure toe $d_s$:

$$(H_s)_{toe} = 2.16 \text{ m } (7.0 \text{ ft})$$

ASSUMPTIONS: (See Table VI-5-22 and VI-5-23)

1. Fresh water, $\rho_w = 1000$ kg/m$^3$
2. Rock, $\rho_s = 2650$ kg/m$^3$
3. Two layers, $n = 2$, random placement
4. Quarrystone, rough angular
5. No damage criteria (See Table VI-5-21 for damage values, D and S)

## 1. Hudson (1974) SPM (1984)

Use $H = H_{1/10} = 1.27 H_s$ for the Rayleigh distributed wave heights and related $K_D$-values for stability coefficient. These recommendations of SPM (1984) introduce a factor of safety compared to that recommended in the SPM (1977). The no damage range is $D = 0-5\%$.

From Eqn VI-5-67, rearranged for the median rock mass

$$M_{50} = \frac{\rho_s H^3}{K_D (\rho_s/\rho_w - 1)^3 \cot\alpha}$$

but

$$W_{50} = M_{50}\, g$$

hence,

$$W_{50} = \frac{\rho_s g H^3}{K_D (\rho_s/\rho_w - 1)^3 \cot\alpha}$$

or if

$$\gamma_s = \rho_s g = \text{unit weight of rock}$$

and

$$\rho_s/\rho_w = S, \text{ the specific gravity for rock}$$

then

$$W_{50} = \frac{\gamma_s H^3}{K_D (S-1)^3 \cot\alpha}$$

and is the common form of the Hudson formula. $K_D$ is the Hudson stability coefficient.

EXAMPLE PROBLEM VI-7-10 (continued)

(a) Non-breaking waves $K_D = 4.0$, gives

$$W_{50} = 11{,}946\ N. \quad (2597\ lbs)$$

and this gives a median rock size, equivalent cube length, $D_{n50}$ of

$$D_{n50} = 0.77\,m\ (2.5\ ft)$$

(b) Breaking waves $K_D = 2.0$, gives

$$W_{50} = 23{,}893\ N \quad (5173\ lbs)$$
$$D_{n50} = 0.97\,m\ (3.15\ ft)$$

(c) Governing wave conditions

At the toe, $\frac{H_s}{d} = \frac{2.16\,m}{3.0\,m} = 0.7$, the wave condition is close to breaking for shallow water and the wave will break as depth decreases on the sloped revetment.

2. <u>van der Meer (1988) CEM 2000</u>

Additional assumptions and data input are required. See Table VI-5-23.

(a) Notational permeability coefficient, P
As shown on Fig. VI-5-11 for impermeable rock revetments,

$$P = 0.1$$

(b) Number of waves, $N_z$
This value depends on the length of the storm and average wave period during the storm. For example, a 16-17 hour storm with average wave period, $T_m = 8$ sec would produce about 7500 waves. When $N_z > 7500$, the equilibrium damage criteria is obtained.

$$\text{Let } N_z = 7500$$

(c) Relative eroded (damage) area, S
This variable is defined by Eqn VI-5-60

$$S \equiv \frac{A_e}{D_{n50}^2}$$

where $A_e$ is the eroded cross-section area around the SWL. Thus S is a dimensionless damage parameter, independent of slope length. Table VI-5-21 presents damage levels (initial, intermediate, failure) for a two-layer armor layer (n=2). For a slope 1:2.5 (interpolate)

EXAMPLE PROBLEM VI-7-10 (continued)

Initial damage, $S = 2$ (percent)
Intermediate damage, $S = 5-7.5$ (percent)
Failure $S \geq 10$ (percent)

Hence, for initial or no damage condition, use

$$S = 2 \text{ (nominal value)}$$

For irregular waves striking the revetment at 90° (normal), the applicable formulas of van der Meer (1988) are found in Table VI-5-23. Two cases exist depending on whether the waves are (1) plunging or (2) surging against the revetment slope.

(1) Plunging Waves : $\xi_m < \xi_{mc}$ (Eqn VI-5-53)

Recall Example VI-7-1 where the surf-similarity parameter, $\xi_m$ was defined and discussed. Here, it is determined for a mean wave period, $T_m$ using the wave height at the toe, $H_{toe}$. Therefore

$$s_m = \frac{H_{toe}}{\frac{g}{2\pi}(T_m)^2} = \frac{2.16(2\pi)}{9.31(6.6)^2} = 0.03176$$

Here, we have assumed $T_m = 0.32\,T_p$ which is an average relation and slightly different than that employed for Example Problem VI-7-1.

Now we find

$$\xi_m = \frac{\tan\alpha}{\sqrt{s_m}} = \frac{1/2.5}{\sqrt{0.03176}} = 2.257$$

As discussed in Table VI-5-23, if $\xi_m < \xi_{mc}$ where

$$\xi_{mc} \equiv \left[6.2\,P^{0.31}(\tan\alpha)^{0.5}\right]^{1/(P+0.5)}$$

then, the Plunging Waves Eqn VI-5-68 is applicable. Hence,

$$\xi_{mc} = \left[6.2(0.1)^{0.31}(0.4)^{0.5}\right]^{1/0.6}$$

gives

$$\xi_{mc} = 2.97$$

Therefore, $\xi_m < \xi_{mc}$ so that the Plunging Wave conditions apply.

It is convenient to apply the stability parameter, $N_s$ form (see Eqn VI-5-58) to give

$$\frac{H_s}{\Delta D_n} = 6.2\,S^{0.2}\,P^{0.18}\,N_z^{-0.1}\,\xi_m^{-0.5} \quad \text{from}$$

Table VI-5-23. Or, $\frac{H_s}{\Delta D_n} = 6.2(2)^{0.2}(0.1)^{0.18}(7500)^{-0.1}(2.257)^{-0.5}$

EXAMPLE PROBLEM VI-7-10 (Continued)

which gives* for $H = H_s = 2.16\,m$, $\Delta = (s-1) = 1.65$

$$D_n = 1.02\,m \quad (3.306\,ft)$$

and

$$W_{50} = 27{,}345\ N \quad (5976\ lbs)$$

The stability number, $N_s = 1.29$. Statically stable breakwaters have this stability parameter in the range 1 – 4 for $H_o T_o < 100$ (van der Meer, 1990).

DISCUSSION: In summary, for the wave, storm and damage conditions, i.e.

$$H_s = 2.16\,m$$
$$T_p = 8\ sec$$
$$S = 2\ percent$$
$$N_z = 7500 \quad (16\text{-}17\ hr\ storm)$$

Hudson (1974) $W_{50} = 23{,}893\ kN$ (5173 lbs) Breaking Wave
van der Meer (1988) $W_{50} = 27{,}345\ KN$ (5976 lbs) Plunging Wave

and it can be said that both methods give simular results.

The Hudson (1974) formula and therefore, the SPM (1984) method limitations include:

- no wave period effects
- no storm duration effects
- no description of the damage level

and others as discussed in subsequent examples. The wave period effects have long been recognized as an important missing element in the Hudson (1974) formulation. For example, as shown in Example VI-7-1, as T increases, the surf similarity parameter increases. If the period in the above example was increased the following results would be obtained from the van der Meer (1988) formulation for Plunging Waves (VI-5-63)

| Period $T_p$, sec | $W_{50}$ N | Remarks |
|---|---|---|
| 9.0 | 33,150 | Plunging Waves formula okay |
| 10.0 | 38,565 | Plunging Waves formula okay |
| 11.0 | 44,363 | Use Surging Waves formula, Eqn VI-5-69 |

Example VI-7-11 demonstrates the practical importance of wave period on armor-layer stability.

* A spread sheet aids the solution.

DR Basco
03/21/01



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HUDSON(1974) SPM(1984)
Vol II,Eqn(7-116)

ARMOUR LAYER STABILITY
Rubble-Mound Structures and Non-Overtopped Slopes
(Metric System)

*Enter initial conditions*
Quarrystone,rough angular
n=2 layers,random placement
Structure Trunk
Nonbreaking waves Kd=4.0
Breaking waves Kd=2.0
Cot a=1.5 to 5.0
No damage criteria (0-5%)

| **Enter Data** | | | Units |
|---|---|---|---|
| Wave Height,Hs= | | 2.16 | meters |
| Rho for rock | = | 2650 | kg/m^3 |
| Gravity | = | 9.81 | m/sec^2 |
| Rho for water | = | 1000 | kg/m^3 |
| Kd coefficient | = | 2.0 | |
| Sr | = | 2.65 | |

| | **H=Hs,sign.wave ht.** | | **H=H one-tenth** | |
|---|---|---|---|---|
| SLOPE | **WEIGHT** | **DIAMETER** | **WEIGHT** | **DIAMETER** |
| COT Alpha | KN | meter | KN | meter |
| 1.5 | **19440.3** | **0.907674** | **39821.19** | **1.152581** |
| 2.0 | **14580.23** | **0.824677** | **29865.89** | **1.04729** |
| 2.5 | **11664.18** | **0.765563** | **23892.71** | **0.97229** |
| 3.0 | **9720.152** | **0.720423** | **19910.59** | **0.915015** |
| 4.0 | **7290.114** | **0.654548** | **14932.95** | **0.831426** |
| 5.0 | **5832.091** | **0.607629** | **11946.36** | **0.771885** |

Formula

$$\text{WEIGHT} = \frac{(\text{Rho-rock}*g)*H^3)}{Kd*(Sr-1)^3*\text{Cot(Alpha)}}$$

53.

VAN DER MEER(1988) CEM(2000)
**Table VI-5-23**
**Eqn VI-5-68 Plunging Waves**
*Enter initial conditions*
Quarry stone,rough angular
n=2 layers, random placement
Structure Trunk
COT alpha = 1.5 to 5.0
Damage conditions specified by S

ARMOUR LAYER STABILITY
Rubble-Mound Structures and Non-Overtopping Slopes
(Metric System)

| **Enter Data** | | | Units |
|---|---|---|---|
| Wave Height,Hs | = | 2.16 | meters |
| Rho for rock | = | 2650 | Kg/m^3 |
| Gravity | = | 9.81 | m/sec^2 |
| Rho for water | = | 1000 | Kg/m^3 |
| Delta = Sr-1 | = | 1.65 | |
| Gamma Z | = | 1.00 | |
| Gamma H | = | 1.00 | |

| **Additional Data** | | | Units |
|---|---|---|---|
| Nominal Damage,S | = | 2.0 | |
| Permeability,P | = | 0.1 | |
| No. of Waves,Nz | = | 7500 | |
| Wave Period,Tm | = | 6.65 | sec |

| **Calculated Information** | | | Units |
|---|---|---|---|
| Wave Length,Lom | = | 69.05 | meet |
| Steepness,Som | = | 0.0313 | |
| Peak period,Tp | = | 8.01325 | sec |
| Storm duration,t | = | 13.85 | hrs |

Remarks: Eta must be less than Eta critical

**Hs,Significant Wave Height**

| SLOPE COT alpha | SURF PRA Eta | DIAMETER meters | WEIGHT KN | Eta critical | REMARKS | |
|---|---|---|---|---|---|---|
| 1.5 | 3.7682228 | 1.3181 | **59533.3** | 4.541961 | **OK** | **0.99** |
| 2.0 | 2.8261671 | 1.1415 | **38667.2** | 3.57378 | **OK** | **1.15** |
| 2.5 | 2.2609337 | 1.021 | **27668.9** | 2.967355 | **OK** | **1.28** |
| 3.0 | 1.8841114 | 0.932 | **21045.7** | 2.54909 | **OK** | **1.40** |
| 4.0 | 1.4130835 | 0.8072 | **13672.8** | 2.005716 | **OK** | **1.62** |
| 5.0 | 1.1304668 | 0.722 | **9784.2** | 1.665372 | **OK** | **1.81** |

**Table VI-5-22**
Rock, two-layer armored non-overtopped slopes (Hudson 1974).

Irregular, head-on waves

$$\frac{H}{\Delta D_{n50}} = (K_D \cot\alpha)^{1/3} \quad \text{or} \quad M_{50} = \frac{\rho_s H^3}{K_D \left(\frac{\rho_s}{\rho_w} - 1\right)^3 \cot\alpha} \qquad \text{(VI-5-67)}$$

where
- $H$ — Characteristic wave height ($H_s$ or $H_{1/10}$)
- $D_{n50}$ — Equivalent cube length of median rock
- $M_{50}$ — Medium mass of rocks, $M_{50} = \rho_s D_{n50}^3$
- $\rho_s$ — Mass density of rocks
- $\rho_w$ — Mass density of water
- $\Delta$ — $(\rho_s/\rho_w) - 1$
- $\alpha$ — Slope angle
- $K_D$ — Stability coefficient

*$K_D$-values by SPM 1977, $H = H_s$, for slope angles* $1.5 \leq \cot\alpha \leq 3.0$. (Based entirely on regular wave tests.)

| Stone shape | Placement | Damage, $D$ (4) | | | |
|---|---|---|---|---|---|
| | | 0-5% | | 5-10% | 10-15% |
| | | Breaking waves (1) | Non-breaking waves (2) | Non-breaking waves | Non-breaking waves |
| Smooth, rounded | Random | 2.1 | 2.4 | 3.0 | 3.6 |
| Rough angular | Random | 3.5 | 4.0 | 4.9 | 6.6 |
| Rough angular | Special (3) | 4.8 | 5.5 | | |

*$K_D$-values by SPM 1984, $H = H_{1/10}$.*

| Stone shape | Placement | Damage, $D$ (4) = 0-5% | |
|---|---|---|---|
| | | Breaking waves (1) | Non-breaking waves (2) |
| Smooth rounded | Random | 1.2 | 2.4 |
| Rough angular | Random | 2.0 | 4.0 |
| Rough angular | Special (3) | 5.8 | 7.0 |

(1) Breaking waves means depth-limited waves, i.e., wave breaking takes place in front of the armor slope. (Critical case for shallow-water structures.)

(2) No depth-limited wave breaking takes place in front of the armor slope.

(3) Special placement with long axis of stone placed perpendicular to the slope face.

(4) $D$ is defined according to SPM 1984 as follows: The percent damage is based on the volume of armor units displaced from the breakwater zone of active armor unit removal for a specific wave height. This zone extends from the middle of the breakwater crest down the seaward face to a depth equivalent to the wave height causing *zero* damage below still water level.

SPM 1977 vs. SPM 1984: When considering that $H_{1/10} = 1.27H_s$ for Rayleigh distributed wave heights (non-depth-limited waves) it is seen that the recommendations of SPM 1984 introduce a considerable safety factor compared to the practice based on SPM 1977.

Uncertainty of the formula: The coefficient of variation of Eqn. VI-5-67 is estimated to be 18% by van der Meer (1988). Melby and Mlaker (1997) reported a coefficient of variation for $K_D$ of 25% for stone and 20% for dolosse.

**Table VI-5-23**
Rock, two-layer armored non-overtopped slopes (van der Meer 1988)

Irregular, head-on waves

$$\frac{H_s}{\Delta D_{n50}} = 6.2 \cdot S^{0.2} P^{0.18} N_z^{-0.1} \xi_m^{-0.5} \qquad \text{Plunging waves}: \ \xi_m < \xi_{mc} \qquad \text{(VI-5-68)}$$

$$\frac{H_s}{\Delta D_{n50}} = 1.0 \cdot S^{0.2} P^{-0.13} N_z^{-0.1} (\cot\alpha)^{0.5} \xi_m^{P} \qquad \text{Surging waves}: \ \xi_m > \xi_{mc} \qquad \text{(VI-5-69)}$$

$$\xi_m = s_m^{-0.5} \tan\alpha \qquad \xi_{mc} = \left(6.2\, P^{0.31} (\tan\alpha)^{0.5}\right)^{1/(P+0.5)}$$

where

| | |
|---|---|
| $H_s$ | Significant wave height in front of breakwater |
| $D_{n50}$ | Equivalent cube length of median rock |
| $\rho_s$ | Mass density of rocks |
| $\rho_w$ | Mass density of water |
| $\Delta$ | $(\rho_s/\rho_w) - 1$ |
| $S$ | Relative eroded area (see Table VI-5-21 for nominal values) |
| $P$ | Notional permeability (see Figure VI-5-11) |
| $N_z$ | Number of waves |
| $\alpha$ | Slope angle |
| $s_m$ | Wave steepness, $s_m = H_s/L_{om}$ |
| $L_{om}$ | Deepwater wave length corresponding to mean wave period |

Validity:

1) Equations VI-5-68 and VI-5-69 are valid for non-depth-limited waves. For depth-limited waves $H_s$ is replaced by $H_{2\%}/1.4$.
2) For $\cot\alpha \geq 4.0$ only Eqn VI-5-68 should be used.
3) $N_z \leq 7,500$ after which number equilibrium damage is more or less reached.
4) $0.1 \leq P \leq 0.6$ , $0.005 \leq s_m \leq 0.06$ , $2.0\ tonne/m^3 \leq \rho \leq 3.1\ tonne/m^3$
5) For the 8 tests run with depth-limited waves, breaking conditions were limited to spilling breakers which are not as damaging as plunging breakers. Therefore, Eqns. VI-5-68 and VI-5-69 may not be conservative in some breaking wave conditions.

Uncertainty of the formula: The coefficient of variation on the factor 6.2 in Eqn. VI-5-68 and on the factor 1.0 in Eqn. VI-5-69 are estimated to be 6.5% and 8%, respectively.

Test program: See Table VI-5-4.

# EXAMPLE PROBLEM VI-7-11

FIND: 1. The design wave height for a stable, uniform-sized armor stone placed on an impermeable revetment slope with non-overtopping waves.
2. Study the evolution in design waves since the 1960's including such factors as alteration in coefficients, wave period and partial safety factors for design.

GIVEN: In the early 1960's, the Chesapeake Bay Bridge Tunnel (CBBT) islands (see Fig. VI-7-X1) were constructed with ten ton (US units) armor stones on a 2:1 slope (single layer) as a revetment for storm protection. Fig. VI-7-X2 is a cross-section of the original design for a typical island. The CBBT revetments have been relatively stable and survived many northeasters and hurricane events.

On October 31, 1991 the famous Halloween storm caused severe damage to the revetment. (This storm has been the subject of a best selling novel "The Perfect Storm", Junger 1997 and a Hollywood movie "The Storm of the Century"). Hydrographic surveys determined the extent of damage as discussed in Example Problem VI-7-12.

Wave conditions measured at the Corps', Field Research Facility (FRF) located 65 miles south in 8 m water depth were $H_s = 4.6$ m, $T_p = 22$ sec. At the Virginia Beach wave gauge, VA001 located in 8 m depth, $H_s = 2.6$ m and $T_p$ about 22 sec under peak conditions. These waves came from 90° (True North) direction and lasted about 12 hours. The measured storm surge at Hampton Roads tide gauge (Sewells Pt) was 0.85 m.

ASSUMPTIONS: (See Tables VI-5-22 and VI-5-23)

1. Sea water, $\rho_w = 1030$ kg/m³
2. Rock $\rho_s = 2650$ kg/m³
3. One layer, $n = 1$, tough angular, random placement
4. No damage criteria (see Table VI-5-21 for D and S damage values

SOLUTION:

1. Hudson (1974) SPM (1977)

Estimate the stable design wave height $H = H_s$. From Eqn VI-5-67

$$\frac{H}{\Delta D_{n50}} = (K_D \cot\alpha)^{1/3}$$

Therefore, when $W = (\rho_s g)(D_{n50})^3 = 10$ tons $= 20{,}000$ lbs $(89{,}000$ N$)$ gives using English units

$$\rho_s g = \gamma_s = 165 \text{ lb/ft}^3 \ (10{,}104 \text{ N/m}^3)$$

EXAMPLE PROBLEM VI-7-11 (continued)

therefore, $D_{n50} = \left(\frac{20,000}{165}\right)^{1/3} = 4.95$ ft (1.508m)

Now, considering only wave breaking events on the revetment, as seen in Table VI-5-22, $K_D$ values employed in 1977 were $K_D = 3.5$ gives

$$H_s = \left(\frac{\rho_s}{\rho_w} - 1\right)(4.95)(3.5 \cdot 20)^{1/3}$$

$$H_s = (1.57)(4.95)(1.913)$$

$$H_s = 14.9 \text{ ft } (4.53 \text{ m})$$

If the stones were smooth and rounded, $K_D = 2.1$ giving $H_s = 12.5$ ft (3.82m) These 10 ton stones would only be stable under reduced wave conditions

2. Hudson (1974) SPM (1984)

The SPM (1984) took $H_{1/10} = 1.27\, H_s$ from the Rayleigh Distribution, and reduced the Hudson coefficients to introduce some safety factor into the design. Hence

$$H_{1/10} = \Delta D_{n50} (K_D \cdot \cot\alpha)^{1/3}$$

$$H_{1/10} = 1.57\,(4.95)(2.0 \cdot 2.0)^{1/3}$$

$$H_{1/10} = 12.3 \text{ ft } (3.76 \text{ m})$$

so that $H_s = 9.7$ ft (2.95m) for the stable conditions

Note also that for smooth stones ($K_D = 1.2$) gives $H_{1/10} = 10.4$ ft (3.17m) and $H_s = 8.19$ ft (2.50m)

These results should be interpreted to demonstrate that for a given armor stone weight the design wave height for the stable, no damage condition has decreased by about 35 percent using the SPM (1984). However, these wave heights were approximately those measured at Duck, NC ($H_s = 4.6$m) and at Virginia Beach, VA, during the Halloween Storm event. Yet severe damage occurred to the 10 ton armor stone revetment. This indicates that other factors may have been present that are not taken into consideration by the Hudson (1974) formula.

The Halloween storm event was unique to the Atlantic Ocean, East Coast for the very long period swell waves ($T_p > 20$ sec) generated and recorded. Wave period is not a variable in the Hudson formula

3. van der Meer (1988) CEM 2000

Now consider for irregular, head-on waves on rock, non-overtopping

EXAMPLE PROBLEM VI-7-11 (continued)

slopes, the formulas of van der Meer, 1988 as shown in Table VI-5-23.

Assume as additional, needed variables

- $P = 0.1$ for impermeable, rock revetments
- $N_z = \frac{t}{T_m} = \frac{12 \text{ hours } (3600 \text{ sec/hr})}{18.3 \text{ sec}} = 2360$ waves
- $S = 2$ (nominal value) for the initial, no damage condition
- $T_p = 22.0$ sec ($T_m \doteq 18.3$ sec)

(a) Assume Eqn VI-5-68 for Plunging Waves is applicable

Because of the very long wave period, $T_p = 22$ sec giving $T_m \doteq 18.3$ sec, the surf-similarity parameter, $\xi_m$ given by

$$\xi_m = \frac{\tan\alpha}{\sqrt{S_m}}$$

with

$$S_m = \frac{H_{so}}{\frac{g}{2\pi}(T_m)^2}$$

gives a relatively large value of $\xi_m$. For example, taking $H_s = 5.75$ ft (1.75 m) gives $\xi_m = 3.57$

But the critical $\xi_{mc}$ is found from Table VI-5-23

$$\xi_{mc} = \left(6.2\, P^{0.31} (\tan\alpha)^{0.5}\right)^{1/P+0.5}$$

$$\xi_{mc} = 3.57$$

Therefore, since $\xi_m > \xi_{mc}$, Table VI-5-23 requires that the Surging Waves, Eqn VI-5-69 be employed. Note that taking

$$H_s = 5.75 \text{ ft } (1.75 \text{ m})$$
$$T_p = 22.0 \text{ sec}$$

and the above values for $P$, $N_z$ and $S$ gives for Plunging Waves

$$W_{50} = 20{,}121 \text{ lbs } (89{,}500 \text{ N})$$

$$D_{50} = 4.95 \text{ ft } (1.5 \text{ m})$$

as the wave height for stable, 10 ton (20,000 lb) armor stones. The stability number, $N_s = 0.74$ and indicates unstable conditions.

EXAMPLE PROBLEM VI-7-11 (continued)

(b) Use Surging Waves, Eqn VI-5-69 ($\xi_m > \xi_{mc}$)

The stability parameter, $N_s$ form in Table VI-5-23 is

$$\frac{H_s}{\Delta D_n} = 1.0\, S^{0.2} P^{-0.13} N_z^{-0.1} (\cot\alpha)^{0.5} \xi_m^{P}$$

giving

$$= 1.0\,(2)^{0.2}(0.1)^{-0.13}(2360)^{-0.1}(2.0)^{0.5}(\xi_m)^{0.1}$$

so that a value of $H_s$ is needed to solve for $D_n$ and $W_n$. By trial and error or a spread sheet solution, we find

$$H_s = 9.5 \text{ ft } (2.90 \text{ m})$$

giving

$$D_n = 4.96 \text{ ft } (1.51 \text{ m})$$

$$W_n = 20{,}168 \text{ lbs } (89{,}750 \text{ N})$$

The stability number, $N_s = 1.22$, i.e. $N_s > 1$ for stable conditions.

Now for this same wave height, $H_s = 9.5$ ft (2.90 m), what armor layer weight, $W_{50}$ is required for shorter wave periods, $T_p$ to remain stable?

| Wave Period $T_p$, sec | Weight $W_{50}$ lbs | Remarks |
|---|---|---|
| 20.0 | 20,770. | OK - Surging Eqn VI-5-69 |
| 15.0 | 22,640. | " " " |
| 12.0 | 24,206. | " " " |
| 10.0 | 25,580 | Use Plunging Eqn VI-5-68 |

Now we see that for surging waves, lowering the wave period, increases the stone weight, $W_{50}$ for stability up to some point where the conditions for the Surging Wave Eqn. are no longer applicable. This is the opposite trend as shown in Example VI-7-10 for the case where the Plunging Wave Eqn was applicable. In general, each equation is only applicable for the special conditions

$\xi_m < \xi_{mc}$ Use Plunging Waves, Eqn VI-5-68

$\xi_m > \xi_{mc}$ Use Surging Waves, Eqn VI-5-69

and the wave period, $T_p$ is an important variable in the equation for $\xi_m$.

All the above does not address the need for some safety factors in applying the van der Meer formulas for design



22-141 50 SHEETS
22-142 100 SHEETS
22-144 200 SHEETS
AMPAD

EXAMPLE PROBLEM VI-7-11 (continued)

4. Partial Safety Factors, The CEM Part VI, Chapter 6

The theory behind the inclusion of partial safety factors for the stable design of armor stone is found in Part VI, Chapter 6 of the CEM. In general the safety factors increase

1. as our knowledge of the wave height conditions decrease, and
2. as our desire for a risk free, low failure probability increases.

Table VI-6-6 presents the Partial Safety Factors ranging up to 1.9 for Surging Wave conditions on non-overtopping slopes using the van der Meer, 1988 Eqn VI-6-45.

Here, we consider the influence of the partial safety factors on the wave heights for a stable armor stone weight, 10 tons on a 2:1 slope for this example. In all cases, also take $P=0.1$, $S=2$, and $N_z = 2360$ waves for $T_p = 22.0$ sec. Consider two cases:

(a) Excellent Knowledge of Wave Conditions ($\sigma = 0.05$) at Site.

| Failure Probability | $P_f$ | $\gamma_H$ | $\gamma_z$ | $H_s$ ft (m) | Rmks |
|---|---|---|---|---|---|
| Low | 0.01 | 1.7 | 1.00 | 5.73 (1.75) | |
| Medium | 0.10 | 1.3 | 1.02 | 7.25 (2.21) | |
| High | 0.40 | 1.0 | 1.08 | 8.85 (2.70) | |
| No. S.F. | ? | 1.0 | 1.0 | 9.5 (2.90) | |

Decreasing the degree of risk of failure (i.e. including safety factors) means lowering the wave height design conditions for the same armor stone weight and revetment slope.

(b) Relatively Poor Knowledge of Wave Conditions ($\sigma = 0.2$) at Site

| Failure Probability | $P_f$ | $\gamma_H$ | $\gamma_z$ | $H_s$ ft (m) | Rmks |
|---|---|---|---|---|---|
| Low | 0.01 | 1.9 | 1.02 | 5.05 (1.54) | |
| Medium | 0.10 | 1.4 | 1.04 | 6.65 (2.03) | |
| High | 0.40 | 1.1 | 1.00 | 8.70 (2.65) | |

To insure a low failure probability means using 10 ton armor stone on a 2:1 slope in regions with long period waves but wave heights only in the 5-6 ft range.

Clearly, since these armor stones were severly damaged in the October 31, 1991 storm, the storm wave heights must have been greater than all those calculated above.

EXAMPLE PROBLEM VI-7-11 (continued)

5. Summary:

Given:

$W_{50}$ = 10 tons = 20,000 lbs (89,000 KN)
$D_{50}$ = 4.95 ft (1.51 m)
$P$ = 0.1
$S$ = 2
$N_z$ = 2360
$T_p$ = 22 sec ($T_m$ = 18.3 sec)

| Armor-Layer Stability Formula | Stable, Significant Wave Height, Hs | | Rmks |
|---|---|---|---|
| | feet | meters | |
| Hudson (1974) SPM (1977) | 14.9 | 4.53 | No period effects<br>No safety factor |
| Hudson (1974) SPM (1988) | 9.7 | 2.95 | No period effects<br>Revised coefficients, conservative |
| van der Meer (1988) (no safety factor) | 9.5 | 2.90 | $T_p$ = 22.0 sec<br>No safety factor |
| van der Meer (1988) (with safety factors) | 5-9 | 1.5-2.7 | $T_p$ = 22.0 sec<br>Includes partial safety factors, Part VI, Chapter 6 |

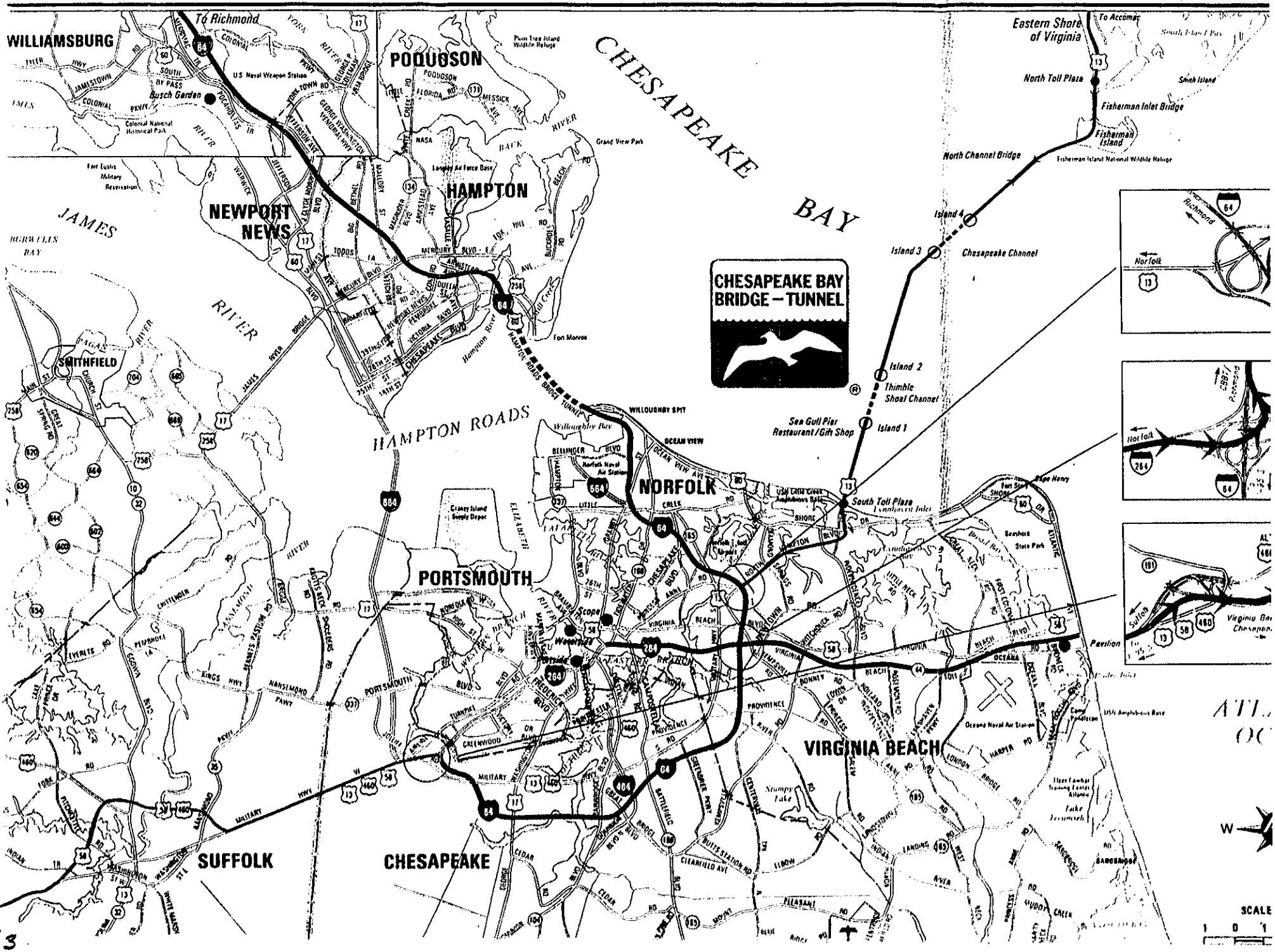
Clearly, a wide range of wave heights are possible based on these formulas. Example VI-7-12 considers the damage experienced by the CBBT island revetments to determine the design wave conditions. Example VI-7-13 considers the size (weight) of the armor stones for repair

D.R. Basco
03/22/01



22-141 50 SHEETS
22-142 100 SHEETS
22-144 200 SHEETS
AMPAD

# Southeastern Virginia



Orginal Dwg. for Armor Revetment, Tunnel Islands (Sverdrup and Parcel, Consulting Engrs, Oct 25, 1960)

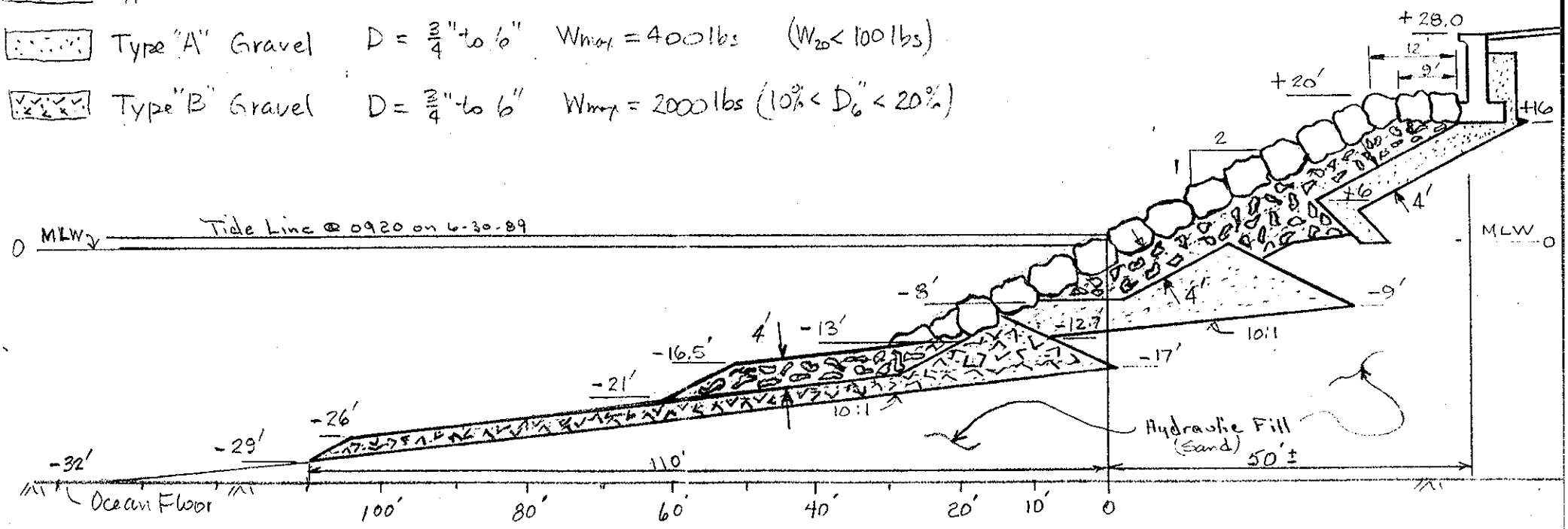
## Materials

Type A Heavy Rip-Rap (Stone) $W = 10$ tons ($6t < W_{10} < 10t$)

Type B Quarry Run Rock $W_{50} = 1000$ lbs ($500 < W < 2000$ lbs) ($100 < W_{10} < 500$ lbs)

Type "A" Gravel $D = \frac{3}{4}$" to 6" $W_{max} = 400$ lbs ($W_{20} < 100$ lbs)

Type "B" Gravel $D = \frac{3}{4}$" to 6" $W_{max} = 2000$ lbs ($10\% < D_6'' < 20\%$)




Type "A" Quarry Run-Type "B" Gravel-Type "A" Gravel-Type "B"

# Typical Island Cross-Section

Chesapeake Bay Bridge Tunnel

Scale 1" = 20'

0 10' 20'

64

Drawn July 6, 1989

VAN DER MEER(1988) CEM(2000)
**Table VI-5-23**
**Eqn VI-5-69 Surging Waves**
*Enter initial conditions*
Quarry stone,rough angular
n=2 layers, random placement
Structure Trunk
COT alpha = 1.5 to 5.0
Damage conditions specified by S

ARMOUR LAYER STABILITY
Rubble-Mound Structures and Non-Overtopping Slopes
(English System)

| **Enter Data** | | | Units |
|---|---|---|---|
| Wave Height,Hs | = | 9.50 | feet |
| Rho for rock | = | 5.141 | slugs/ft^3 |
| Gravity | = | 32.17 | ft/sec^2 |
| Rho for water | = | 2.0 | slugs/ft^3 |
| Delta = Sr-1 | = | 1.5705 | |
| Gamma Z | = | 1.00 | |
| Gamma H | = | 1.00 | |

| **Additional Data** | | | Units |
|---|---|---|---|
| Nominal Damage,S | = | 2.0 | |
| Permeability,P | = | 0.1 | |
| No. of Waves,Nz | = | 2360 | |
| Wave Period,Tm | = | 18.3 | sec |

| **Calculated Information** | | | Units |
|---|---|---|---|
| Wave Length,Lom | = | 1714.64 | feet |
| Steepness,Som | = | 0.0055 | |
| Peak period,Tp | = | 22.0515 | sec |
| Storm duration,t | = | 11.997 | hrs |

Remarks: Eta must be greater than Eta critical

**Hs,Significant Wave Height**

| SLOPE COT alpha | SURF PRA Eta | DIAMETER feet | WEIGHT lbs | Eta critical | REMARKS | |
|---|---|---|---|---|---|---|
| 1.5 | 8.9893 | 5.564 | **28487.87** | 4.542 | **OK** | 1.09 |
| 2.0 | 6.742 | 4.959 | **20168.84** | 3.5738 | **OK** | 1.22 |
| 2.5 | 5.3936 | 4.535 | **15425.19** | 2.9674 | **OK** | 1.33 |
| 3.0 | 4.4947 | 4.216 | **12393.69** | 2.5491 | **OK** | 1.43 |
| 4.0 | 3.371 | 3.758 | **8777.46** | 2.0057 | **OK** | 1.61 |
| 5.0 | Use Plunging Waves Equation | | | | | |

**Table VI-6-4**
Partial safety factors for stability failure of rock armor. Hudson formula. Design without model tests.

Design equation (cf. Table VI-5-22)

$$G = \frac{1}{\gamma_Z}\hat{\Delta}\hat{D}_n(K_D\ c\hat{o}t\ \alpha)^{1/3} - \gamma_H \hat{H}_S^T \qquad \text{(VI-6-43)}$$

| | $\sigma'_{FH_S} = 0.05$ | | $\sigma'_{FH_S} = 0.2$ | |
|---|---|---|---|---|
| $P_f$ | $\gamma_H$ | $\gamma_Z$ | $\gamma_H$ | $\gamma_Z$ |
| 0.01 | 1.7 | 1.04 | 2.0 | 1.00 |
| 0.05 | 1.4 | 1.06 | 1.6 | 1.02 |
| 0.10 | 1.3 | 1.04 | 1.4 | 1.06 |
| 0.20 | 1.2 | 1.02 | 1.3 | 1.00 |
| 0.40 | 1.0 | 1.08 | 1.1 | 1.00 |

**Table VI-6-5**
Partial safety factors for stability failure of rock armor. Plunging waves. Van der Meer formula. Design without model tests.

Design equation (cf. Table VI-5-23)

$$G = \frac{1}{\gamma_Z}6.2\ \hat{S}^{0.2}\ \hat{P}^{0.18}\ \hat{\Delta}\hat{D}_n\hat{f}\ (c\hat{o}t\ \alpha)^{0.5}\ (\hat{s}_{om})^{0.25}\ \hat{N}_z^{-0.1} - \gamma_H \hat{H}_S^T \qquad \text{(VI-6-44)}$$

where the factor $f$ models the effect of low crested breakwaters:

$$\hat{f} = \frac{1}{1.25 - 4.8\frac{R_c}{\hat{H}_S^T}\sqrt{\frac{\hat{s}_{om}}{2\pi}}}$$

| | $\sigma'_{FH_S} = 0.05$ | | $\sigma'_{FH_S} = 0.2$ | |
|---|---|---|---|---|
| $P_f$ | $\gamma_H$ | $\gamma_Z$ | $\gamma_H$ | $\gamma_Z$ |
| 0.01 | 1.6 | 1.04 | 1.9 | 1.00 |
| 0.05 | 1.4 | 1.02 | 1.5 | 1.06 |
| 0.10 | 1.3 | 1.00 | 1.3 | 1.10 |
| 0.20 | 1.2 | 1.00 | 1.2 | 1.06 |
| 0.40 | 1.0 | 1.08 | 1.0 | 1.10 |

**Table VI-6-6**
Partial safety factors for stability failure of rock armor. Surging waves. Van der Meer formula. Design without model tests.

Design equation (cf. Table VI-5-23)

$$G = \frac{1}{\gamma_Z} \hat{S}^{0.2}\, \hat{P}^{-0.13}\, \hat{\Delta}\, \hat{D}_n\, \hat{f}\, (c\hat{o}t\, \alpha)^{(0.5-P)} (\hat{s}_{om})^{-0.5P}\, \hat{N}_z^{-0.1} - \gamma_H \hat{H}_S^T \qquad \text{(VI-6-45)}$$

where

$$\hat{f} = \frac{1}{1.25 - 4.8 \frac{R_c}{\hat{H}_S^T} \sqrt{\frac{\hat{s}_{om}}{2\pi}}}$$

| | $\sigma'_{FH_S} = 0.05$ | | $\sigma'_{FH_S} = 0.2$ | |
|---|---|---|---|---|
| $P_f$ | $\gamma_H$ | $\gamma_Z$ | $\gamma_H$ | $\gamma_Z$ |
| 0.01 | 1.7 | 1.00 | 1.9 | 1.02 |
| 0.05 | 1.3 | 1.10 | 1.6 | 1.00 |
| 0.10 | 1.3 | 1.02 | 1.4 | 1.04 |
| 0.20 | 1.1 | 1.10 | 1.2 | 1.08 |
| 0.40 | 1.0 | 1.08 | 1.1 | 1.00 |

**Table VI-6-7**
Partial safety factors for stability failure of cube block armor. Van der Meer formula. Design without model tests.

Design equation (cf. Table VI-5-29)

$$G = \frac{1}{\gamma_Z} \left( 6.7 \frac{(\hat{N}_{od})^{0.4}}{(\hat{N}_z)^{0.3}} + 1.0 \right) (\hat{s}_{om})^{-0.1} \hat{\Delta} \hat{D}_n - \gamma_H \hat{H}_S^T \qquad \text{(VI-6-46)}$$

| | $\sigma'_{FH_S} = 0.05$ | | $\sigma'_{FH_S} = 0.2$ | |
|---|---|---|---|---|
| $P_f$ | $\gamma_H$ | $\gamma_Z$ | $\gamma_H$ | $\gamma_Z$ |
| 0.01 | 1.5 | 1.10 | 1.8 | 1.04 |
| 0.05 | 1.3 | 1.08 | 1.5 | 1.04 |
| 0.10 | 1.3 | 1.00 | 1.4 | 1.02 |
| 0.20 | 1.2 | 1.00 | 1.2 | 1.06 |
| 0.40 | 1.0 | 1.08 | 1.0 | 1.10 |

EXAMPLE PROBLEM VI-7-12

FIND: The damage curve relationships for wave erosion above the design wave height for uniform-sized armor stone placed on an impermeable revetment slope with non-overtopping waves.

GIVEN: (a) The same conditions as found in Example VI-7-10 for the generic, 2.5:1 sloped revetment, $H_{toe} = 2.16$ m, $T_p = 9$ sec

(b) The same conditions as found in Example VI-7-11 for the Chesapeake Bay Bridge Tunnel (CBBT) island revetments with 2:1 sloped revetment, $T_p = 22.0$ sec, but uncertain knowledge of the wave height, $H_s$.

SOLUTION:

(a) Example VI-7-10

1. The SPM (1984)

Table 7-9 (Vol II, p. 7-211) presents the following generic, $H/H_d$-vs-damage D in percent relationships for rough, quarry stone revetments. (two layers, random placed, nonbreaking waves, minor overtopping)

| $\frac{H}{H_{D=0}}$ | Damage % | $H_s$ (m) | Rmks |
|---|---|---|---|
| 1.0 | 0 - 5 | 2.16 | No damage criteria can actually be up to 5 %. |
| 1.08 | 5 - 10 | 2.33 | In general, the percent damage is based on |
| 1.19 | 10 - 15 | 2.57 | the volume of armor units displaced between |
| 1.27 | 15 - 20 | 2.74 | the middle of the breakwater crest down to |
| 1.37 | 20 - 30 | 2.96 | the seaward face to a depth equal to one, |
| 1.47 | 30 - 40 | 3.18 | zero-damage wave height $H_{D=0}$ below |
| 1.56 | 40 - 50 | 3.37 | the SWL. (See Table VI-5-21) |

No upper bound limit for the damage level to cause "failure", i.e. exposure of the sublayers was discussed.

2. van der Meer (1988)

Table VI-5-21 presents damage level, S for two-layer armor where the inital, intermediate and failure damage levels depend on the slope of the revetment. Here, S is defined as (Eqn VI-5-60)

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

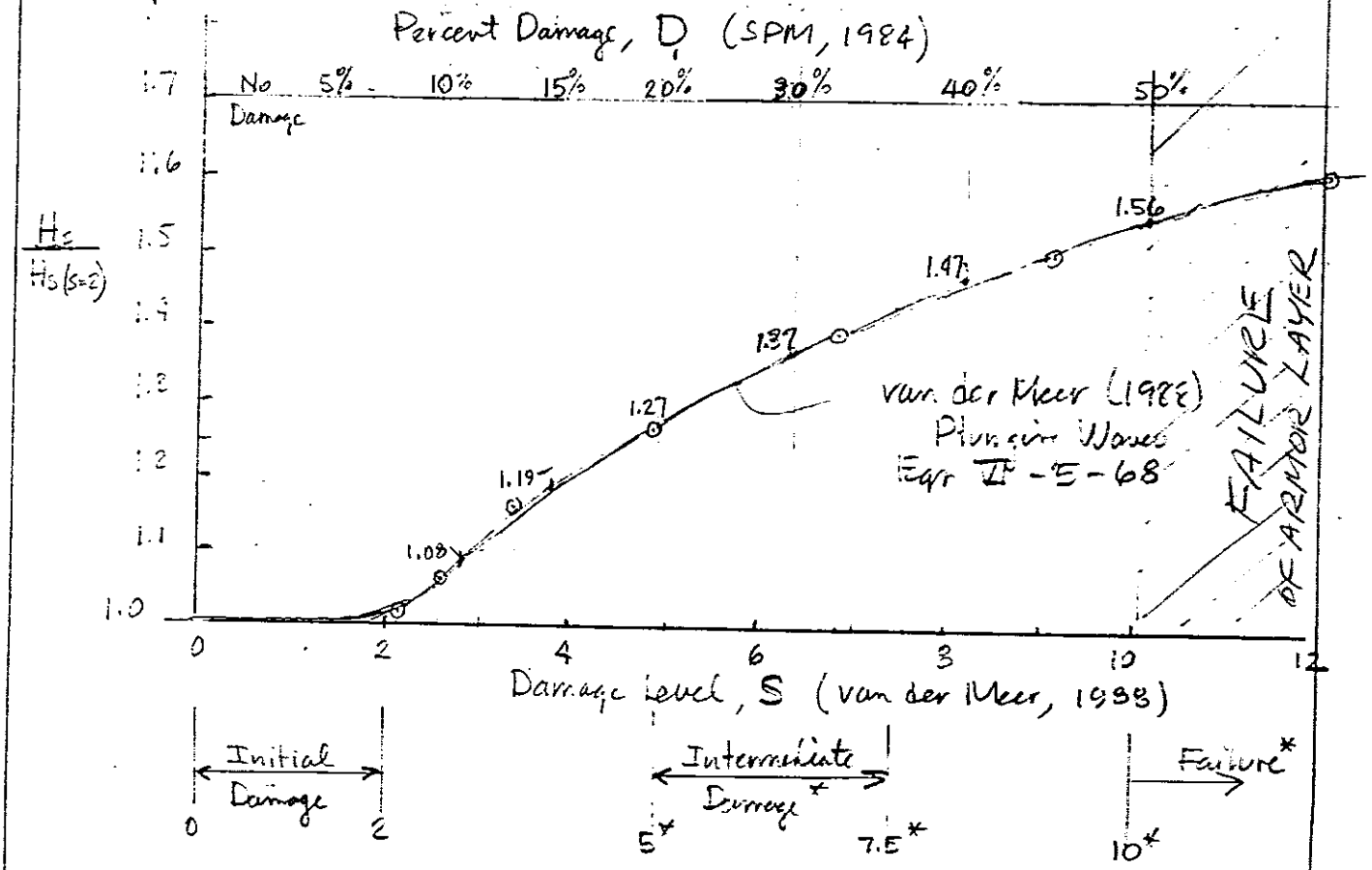
where:
- $A_e$ is the eroded, cross-sectional area near the SWL as illustrated in Table VI-5-19, and
- S can be interpreted as the number of squares with side length $D_{n50}$ which fit into the eroded area.

EXAMPLE PROBLEM VI-7-12 (continued)

We can now employ Eqn VI-5-68 for plunging waves and vary the significant wave height, $H_s$ to calculate, S, the damage level. We will keep $D_{n50}$ = 1.02 m and $W_{50}$ = 27,345 KN (5976 lbs)

| $H_s$ m | $H/H_{s(s=2)}$ | S percent | Relative Damage level | Remarks |
|---|---|---|---|---|
| 2.16 | 1.00 | 1.9703 | 0.99 | Slight rounding error, S = 2 |
| 2.20 | 1.02 | 2.118 | 1.06 | |
| 2.30 | 1.06 | 2.499 | 1.25 | |
| 2.50 | 1.16 | 3.41 | 1.70 | |
| 2.75 | 1.27 | 4.89 | 2.44 | Intermediate damage levels |
| 3.00 | 1.39 | 6.77 | 3.38 | |
| 3.25 | 1.50 | 9.13 | 4.56 | |
| 3.50 | 1.62 | 12.05 | 6.02 | Failure is expected |

Values of $H/H_{s(s=2)}$ -vs- S are plotted below




(* Values interpolated from data for 1:2 and 1:3 slopes in Table VI-5-21)

If we superimpose the $H/H_{D=0}$ values from the SPM (1984) we can construct an approximate percent damage, D scale to compare with the definition of damage level, S. We clearly see that the definitions of intermediate and failure damage levels, S are far different than the percentage damage relations as found in the SPM (1984). Note also that S depends on cot α.

EXAMPLE PROBLEM VI-7-12 (continued)

(b) Example VI-7-11 The CERT Island Revetment

1. The SPM (1984)

The same, generic Table 7-9 (vol II, p. 7-211) applies as presented above. Now the value of H depends on what level $H_{D=0}$ is used for a stable design. From the Hudson formula and SPM (1984) $H_s = 9.7$ feet gives for a 2:1 slope

| $\frac{H}{H_{D=0}}$ | Damage, D percent | $H_s$ feet |
|---|---|---|
| 1.0 | 0-5 | 9.7 |
| 1.08 | 5-10 | 10.48 |
| 1.19 | 10-15 | 11.54 |
| 1.27 | 15-20 | 12.30 |
| 1.37 | 20-30 | 13.29 |
| 1.47 | 30-40 | 14.26 |
| 1.56 | 40-50 | 15.13 |

2. van der Meer (1988)

We now apply Eqn VI-5-69 for surging waves with $H_s(S=2) = 9.5$ and again vary the significant wave height, $H_s$ to calculate S, the damage level. We keep $D_{n50} = 4.95$ ft and $W_{50} = 10$ tons (20,000 lbs)

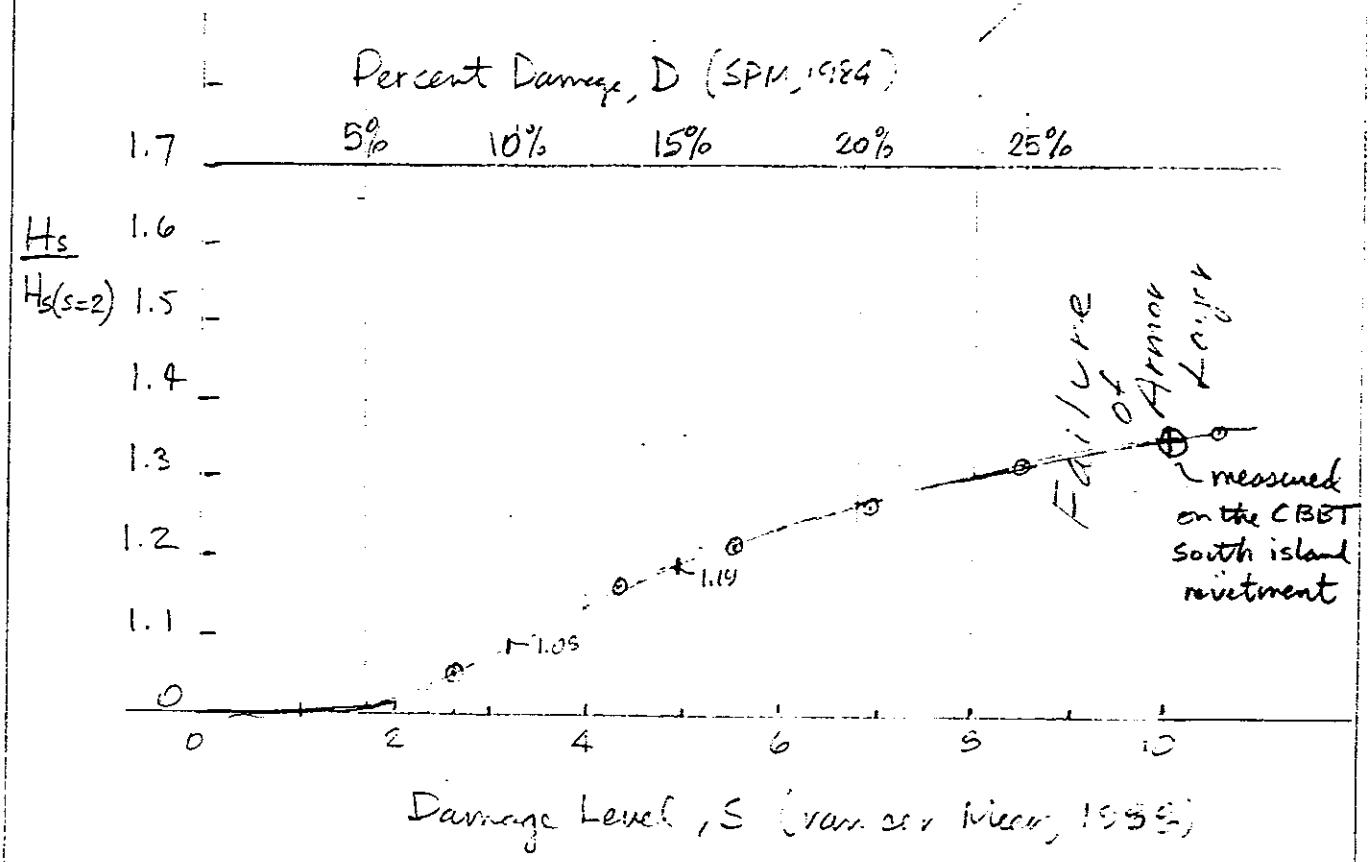
| $H_s$ feet | $\frac{H}{H_s(S=2)}$ | S percent | Relative Damage level | Remarks |
|---|---|---|---|---|
| 9.5 | 1.00 | 2.018 | 1.009 | Slight rounding error, S=2 |
| 10.0 | 1.05 | 2.643 | 1.32 | |
| 11.0 | 1.16 | 4.36 | 2.18 | Intermediate Damage S = 4-6 |
| 11.5 | 1.21 | 5.51 | 2.76 | |
| 12.0 | 1.26 | 6.89 | 3.45 | |
| 12.5 | 1.32 | 8.54 | 4.27 | Failure, S = 8 of Armor Layer |
| 13.0 | 1.37 | 10.5 | 5.25 | |
| 14.0 | 1.47 | 15.5 | 7.75 | Underlayer exposed to direct wave attack |
| 15.0 | 1.58 | 22.2 | 11.1 | |
| 16.0 | 1.68 | 31.2 | 15.6 | |

From Table VI-5-21, van der Meer (1988) gives the following guidelines for 2:1 Slopes

| | | |
|---|---|---|
| Initial Damage | S = 2 | Initial damage - no displacement |
| Intermediate Damage | S = 4-6 | Units displaced but without underlayer exposu |
| Failure (of armor layer) | S = 8 | The underlayer is exposed to direct wave atta |

EXAMPLE PROBLEM VI-7-12 (continued)

Values of $H/H_s(s=2)$ -vs- $S$ are plotted below. Also, approx note percentage damage, D scales are constructed for comparison.




Now note that failure here ($S \geq 8$, $D \doteq 25\%$) is far different than for case (a) where $S \geq 10$, $D = 50\%$ for failure. This simply means that the generic damage curves for D from the SPM (1984) are less accurate to account for all the <u>various factors</u> producing damage (revetment slope, wave period, breaker type, permeability, length of storm, etc.) that are accounted for in the van der Meer (1988) formulation.

EXAMPLE PROBLEM VI-7-12 (continued)

3. Damage Measurements on the CBBT Islands

Damage profile surveys taken by the Engineering staff, CBBT District (East, 1994) have been analyzed to learn that S = 10 from the October 31, 1991 Halloween Storm northeaster. Some underlayers were exposed and this level of the damage parameter is consistent with the criteria for "failure" as shown in Table VI-5-21.

This damage occurred on the South Island on a curved section where the armor stones are more vulnerable, i.e. on the head of the structure rather than within the trunk section. Table VI-5-37 presents a method to estimate stability of rock breakwaters as proposed by Carver and Heimbaugh, 1989.

From the analysis for S = 10, $H/H_{s(s=2)} = 1.35$ giving

$$H_s = 12.8 \text{ ft} \quad (3.9 \text{ m})$$

to produce this level of damage using the van der Meer (1988) formulation for Surging Waves. As shown in Example Problem VI-7-11, wave heights were measured at Duck NC as 15.1 ft (4.6 m). These wave conditions are possible at the Chesapeake Bay entrance with relatively deep (d = 12 m) water.

Example VI-7-12 considers what level of Hs to use to determine the size and weight of armor stone for repairs of the CBBT island.

D.R Basco
03/22/01



22-141 50 SHEETS
22-142 100 SHEETS
22-144 200 SHEETS
AMPAD

VAN DER MEER(1988) CEM(2000)
**Table VI-6-5**
**Eqn VI-6-44 Plunging Waves**
*Enter initial conditions*
Quarry stone,rough angular
n=2 layers, random placement
Structure Trunk
COT alpha = 1.5 to 5.0
Damage conditions specified by S

[redacted]
Rubble-Mound Structures and Non-Overtopping Slopes
(Metric System)

| **Enter Data** | | | Units |
|---|---|---|---|
| Wave Height,Hs | = | 2.16 | meters |
| Rho for rock | = | 2650 | Kn/m^3 |
| Gravity | = | 9.81 | m/sec^2 |
| Rho for water | = | 1000 | Kn/m^3 |
| Delta = Sr-1 | = | 1.65 | |
| Gamma Z | = | 1.00 | |
| Gamma H | = | 1.00 | |

| **Additional Data** | | | Units |
|---|---|---|---|
| Nominal Diameter,Dn= | | [redacted] | meters |
| Permeability,P | = | 0.1 | |
| No. of Waves,Nz | = | 7500 | |
| Wave Period,Tm | = | 6.6 | sec |

| **Calculated Information** | | | Units |
|---|---|---|---|
| Wave Length,Lom | = | 68.01 | meters |
| Steepness,Som | = | 0.0318 | |
| Peak period,Tp | = | 7.953 | sec |
| Storm duration,t | = | 13.75 | hrs |

Remarks: Eta must be less than Eta critical

**Hs,Significant Wave Height**

| SLOPE COT alpha | SURF PRA Eta | [redacted] | Rel Dam Zero=2.0 | Eta critical | REMARKS |
|---|---|---|---|---|---|
| 1.5 | 3.738481 | 7.0658 | **3.5329** | 4.541961 | **OK** |
| 2.0 | 2.8038608 | 3.442 | **1.721** | 3.57378 | **OK** |
| 2.5 | 2.2430886 | 1.9703 | **0.98515** | 2.967355 | **OK** |
| 3.0 | 1.8692405 | 1.2491 | **0.62455** | 2.54909 | **OK** |
| 4.0 | 1.4019304 | 0.6085 | **0.30425** | 2.005716 | **OK** |
| 5.0 | 1.1215443 | 0.3483 | **0.17415** | 1.665372 | **OK** |

**Partial Safety Factors**

| | Sigma Waves = 0.5 | | Sigma Waves = 0.2 | |
|---|---|---|---|---|
| **Pf** | Gamma H | Gamma Z | Gamma H | Gamma Z |
| 0.01 | 1.6 | 1.04 | 1.9 | 1.00 |
| 0.05 | 1.4 | 1.02 | 1.5 | 1.06 |
| 0.10 | 1.3 | 1.00 | 1.3 | 1.10 |
| 0.20 | 1.2 | 1.00 | 1.2 | 1.06 |
| 0.40 | 1.0 | 1.08 | 1.0 | 1.10 |

VAN DER MEER(1988) CEM(2000)
**Table VI-6-5**
**Eqn VI-6-44 Plunging Waves**
*Enter initial conditions*
Quarry stone,rough angular
n=2 layers, random placement
Structure Trunk
COT alpha = 1.5 to 5.0
Damage conditions specified by S

[redacted]
Rubble-Mound Structures and Non-Overtopping Slopes
(Metric System)

| **Enter Data** | | | Units |
|---|---|---|---|
| Wave Height,Hs | = | 2.50 | meters |
| Rho for rock | = | 2650 | Kn/m^3 |
| Gravity | = | 9.81 | m/sec^2 |
| Rho for water | = | 1000 | Kn/m^3 |
| Delta = Sr-1 | = | 1.65 | |
| Gamma Z | = | 1.00 | |
| Gamma H | = | 1.00 | |

| **Additional Data** | | | Units |
|---|---|---|---|
| Nominal Diameter,Dn= | | [redacted] | meters |
| Permeability,P | = | 0.1 | |
| No. of Waves,Nz | = | 7500 | |
| Wave Period,Tm | = | 6.6 | sec |

| **Calculated Information** | | | Units |
|---|---|---|---|
| Wave Length,Lom | = | 68.01 | meters |
| Steepness,Som | = | 0.0368 | |
| Peak period,Tp | = | 7.953 | sec |
| Storm duration,t | = | 13.75 | hrs |

Remarks: Eta must be less than Eta critical

**Hs,Significant Wave Height**

| SLOPE COT alpha | SURF PRA Eta | [redacted] | Rel Dam Zero=2.0 | Eta critical | REMARKS |
|---|---|---|---|---|---|
| 1.5 | 3.4752402 | 12.2268 | **6.1134** | 4.541961 | **OK** |
| 2.0 | 2.6064302 | 5.9562 | **2.9781** | 3.57378 | **OK** |
| 2.5 | 2.0851441 | 3.4095 | **1.70475** | 2.967355 | **OK** |
| 3.0 | 1.7376201 | 2.1614 | **1.0807** | 2.54909 | **OK** |
| 4.0 | 1.3032151 | 1.0529 | **0.52645** | 2.005716 | **OK** |
| 5.0 | 1.0425721 | 0.6027 | **0.30135** | 1.665372 | **OK** |

**Partial Safety Factors**

| | Sigma Waves = 0.5 | | Sigma Waves = 0.2 | |
|---|---|---|---|---|
| **Pf** | Gamma H | Gamma Z | Gamma H | Gamma Z |
| 0.01 | 1.6 | 1.04 | 1.9 | 1.00 |
| 0.05 | 1.4 | 1.02 | 1.5 | 1.06 |
| 0.10 | 1.3 | 1.00 | 1.3 | 1.10 |
| 0.20 | 1.2 | 1.00 | 1.2 | 1.06 |
| 0.40 | 1.0 | 1.08 | 1.0 | 1.10 |

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VAN DER MEER(1988) CEM(2000)
**Table VI-6-6**
**Eqn VI-6-45 Surging Waves**
*Enter initial conditions*
Quarry stone,rough angular
n=2 layers, random placement
Structure Trunk
COT alpha = 1.5 to 5.0
Damage conditions specified by S

ARMOUR LAYER STABILITY

[redacted]

(English System)

**Enter Data**

| | | | Units |
|---|---|---|---|
| Wave Height,Hs | = | 9.50 | feet |
| Rho for rock | = | 5.141 | slugs/ft^3 |
| Gravity | = | 32.17 | ft/sec^2 |
| Rho for water | = | 2.0 | slugs/ft^3 |
| Delta = Sr-1 | = | 1.5705 | |
| Gamma Z | = | 1.00 | |
| Gamma H | = | 1.00 | |

**Additional Data**

| | | | Units |
|---|---|---|---|
| Nominal Diameter,Dn= | | [redacted] | feet |
| Permeability,P | = | 0.1 | |
| No. of Waves,Nz | = | 2360 | |
| Wave Period,Tm | = | 18.3 | sec |

**Calculated Information**

| | | | Units |
|---|---|---|---|
| Wave Length,Lom | = | 1714.64 | feet |
| Steepness,Som | = | 0.0055 | |
| Peak period,Tp | = | 22.0515 | sec |
| Storm duration,t | = | 11.997 | hrs |

Remarks: Eta must be greater than Eta critical

**Hs,Significant Wave Height**

| SLOPE COT alpha | SURF PRA Eta | [redacted] | Rel Dam Zero=2.0 | Eta critical | **REMARKS** |
|---|---|---|---|---|---|
| 1.5 | 8.9893 | **3.587** | 1.7935 | 4.542 | **OK** |
| 2.0 | 6.742 | **2.018** | 1.009 | 3.5738 | **OK** |
| 2.5 | 5.3936 | **1.291** | 0.6455 | 2.9674 | **OK** |
| 3.0 | 4.4947 | **0.897** | 0.4485 | 2.5491 | **OK** |
| 4.0 | 3.371 | **0.504** | 0.252 | 2.0057 | **OK** |
| 5.0 | Use Plunging Waves Equation | | | | |

**Partial Safety Factor**

| | Sigma Waves = 0.05 | | Sigma Waves = 0.2 | |
|---|---|---|---|---|
| **Pf** | Gamma H | Gamma Z | Gamma H | Gamma Z |
| 0.01 | 1.7 | 1.00 | 1.9 | 1.02 |
| 0.05 | 1.3 | 1.10 | 1.6 | 1.00 |
| 0.10 | 1.3 | 1.02 | 1.4 | 1.04 |
| 0.20 | 1.1 | 1.10 | 1.2 | 1.08 |
| 0.40 | 1.0 | 1.08 | 1.1 | 1.00 |

VAN DER MEER(1988) CEM(2000)
**Table VI-6-6**
**Eqn VI-6-45 Surging Waves**
*Enter initial conditions*
Quarry stone,rough angular
n=2 layers, random placement
Structure Trunk
COT alpha = 1.5 to 5.0
Damage conditions specified by S

ARMOUR LAYER STABILITY
[redacted]
(English System)

**Enter Data**

| | | | Units |
|---|---|---|---|
| Wave Height,Hs | = | 12.00 | feet |
| Rho for rock | = | 5.141 | slugs/ft^3 |
| Gravity | = | 32.17 | ft/sec^2 |
| Rho for water | = | 2.0 | slugs/ft^3 |
| Delta = Sr-1 | = | 1.5705 | |
| Gamma Z | = | 1.00 | |
| Gamma H | = | 1.00 | |

**Additional Data**

| | | | Units |
|---|---|---|---|
| Nominal Diameter,Dn= | | [redacted] | feet |
| Permeability,P | = | 0.1 | |
| No. of Waves,Nz | = | 2360 | |
| Wave Period,Tm | = | 18.3 | sec |

**Calculated Information**

| | | | Units |
|---|---|---|---|
| Wave Length,Lom | = | 1714.64 | feet |
| Steepness,Som | = | 0.007 | |
| Peak period,Tp | = | 22.0515 | sec |
| Storm duration,t | = | 11.997 | hrs |

Remarks: Eta must be greater than Eta critical

**Hs,Significant Wave Height**

| SLOPE COT alpha | SURF PRA Eta | [redacted] | Rel Dam Zero=2.0 | Eta critical | REMARKS |
|---|---|---|---|---|---|
| 1.5 | 7.9682 | **12.253** | 6.1265 | 4.542 | **OK** |
| 2.0 | 5.9761 | **6.892** | 3.446 | 3.5738 | **OK** |
| 2.5 | 4.7809 | **4.411** | 2.2055 | 2.9674 | **OK** |
| 3.0 | 3.9841 | **3.063** | 1.5315 | 2.5491 | **OK** |
| 4.0 | 2.9881 | **1.723** | 0.8615 | 2.0057 | **OK** |
| 5.0 | Use Plunging Waves Equation | | | | |

**Partial Safety Factor**

| | Sigma Waves = 0.05 | | Sigma Waves = 0.2 | |
|---|---|---|---|---|
| **Pf** | Gamma H | Gamma Z | Gamma H | Gamma Z |
| 0.01 | 1.7 | 1.00 | 1.9 | 1.02 |
| 0.05 | 1.3 | 1.10 | 1.6 | 1.00 |
| 0.10 | 1.3 | 1.02 | 1.4 | 1.04 |
| 0.20 | 1.1 | 1.10 | 1.2 | 1.08 |
| 0.40 | 1.0 | 1.08 | 1.1 | 1.00 |

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Station # 281

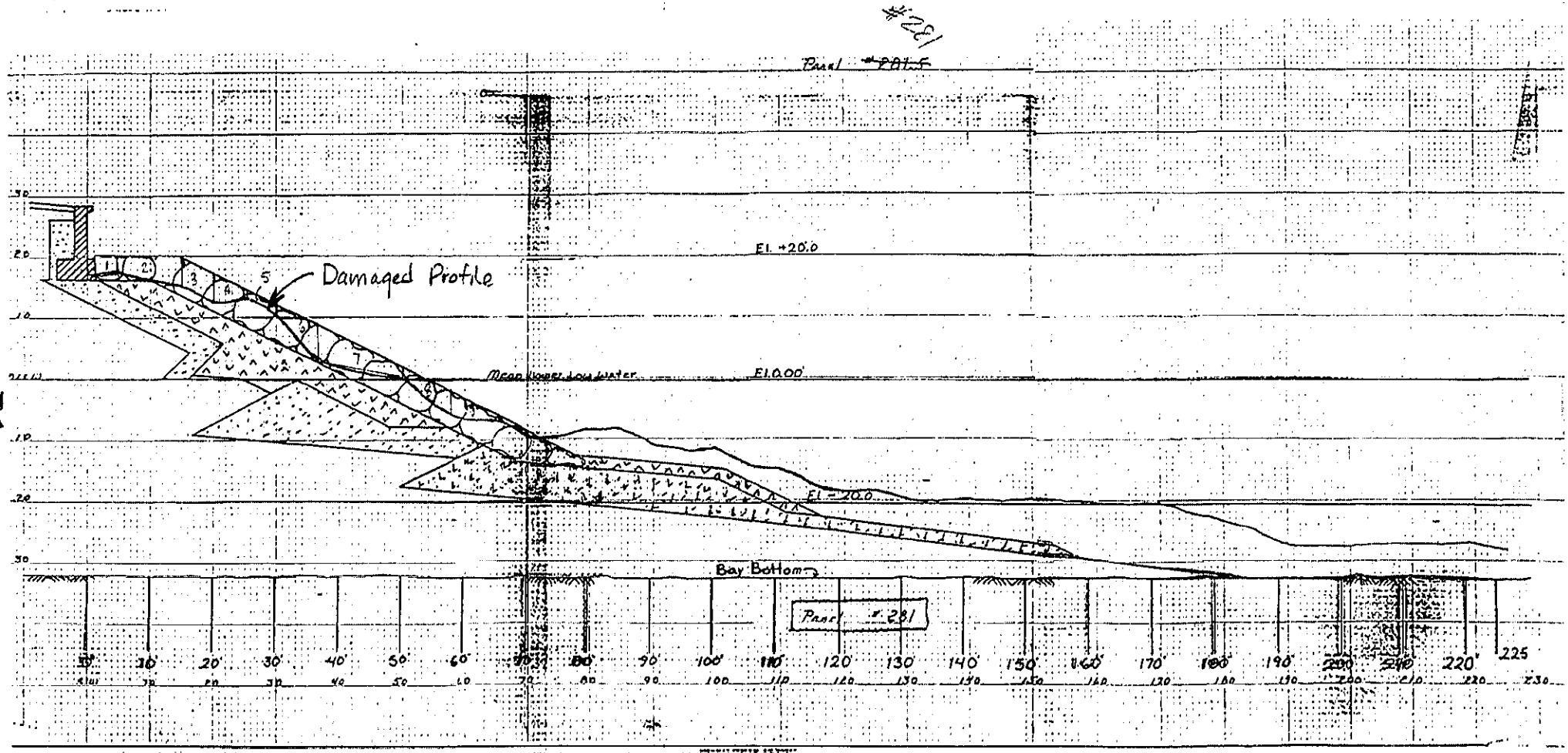


PLATE 3-FULL CROSS SECTION-FULL 001

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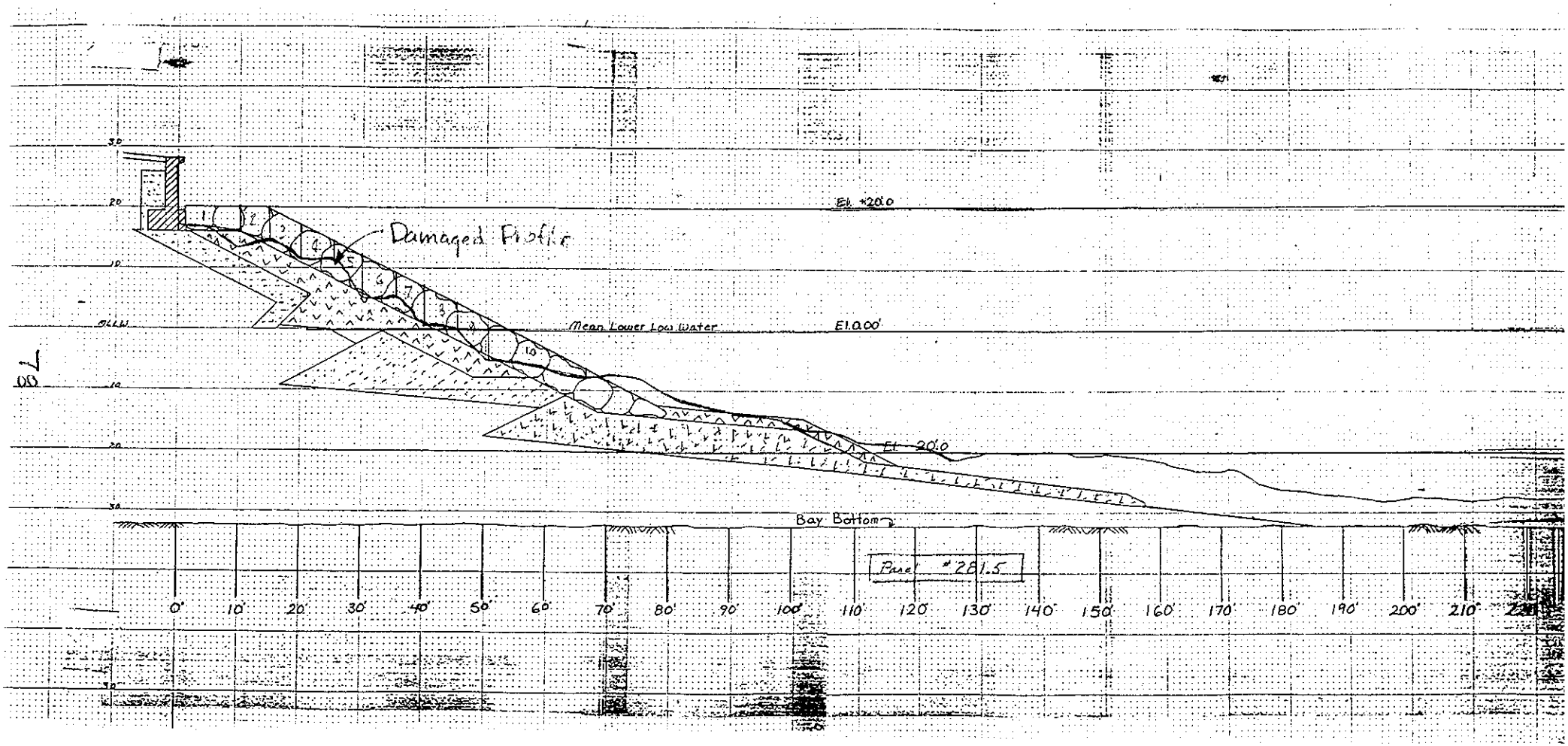
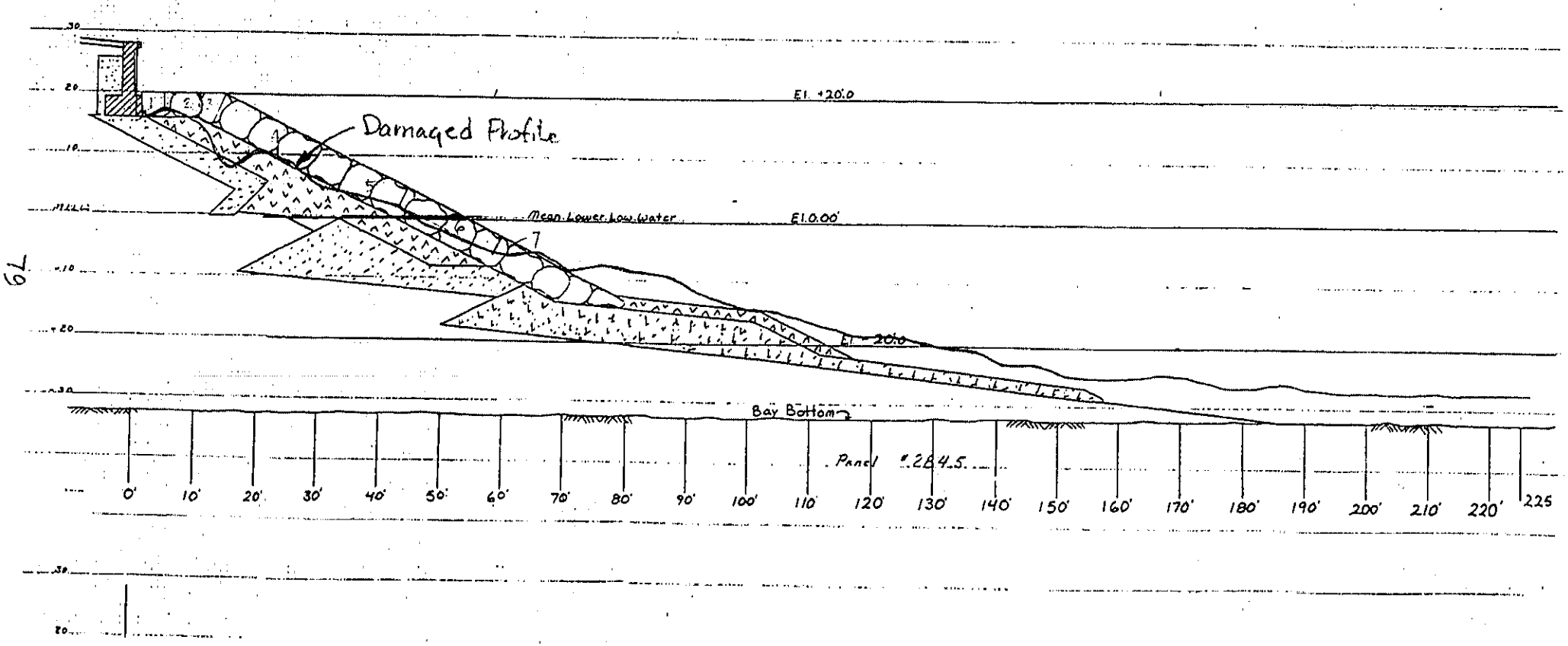
Station No. 281.5



Station No. 284.5



# EXAMPLE PROBLEM VI-7-13

FIND: The weight of armor stone to repair the damage to the CBBT Island revetment.

GIVEN:
- The results of Example Problems VI-7-11 and VI-7-12(b)
- Wave hindcast information, WIS for nearby locations
- Other extreme wave condition measurements / criteria
- Appropriate partial safety factors for wave conditions

1. WIS Station 2059 (Brooks and Brandon, 1995)
   $d = 14\,m$ (46.0 ft) Lat 37.00° N Lon 75.75° W

| Extreme Prob. Dist | Spectral Significant Wave Height, $H_{mo}$, (m) | | | | | |
|---|---|---|---|---|---|---|
| | Recurrence Interval, $T_r$, yrs | | | | | |
| Fisher-Tippett | 2 | 5 | 10 | 20 | 25 | 50 |
| Type I | 5.37 | 6.67 | 7.22 | 7.76 | 7.93 | 8.46 |
| Type II | 5.37 | 6.98 | 7.90 | 8.90 | 9.25 | 10.41 |

2. Virginia Beach Hurricane Protection Project (Corps of Engineers)
   1 % chance storm each year (100 yr recurrence interval
   $d = 30\,ft$ (9.1 m)
   $H_s = 15.8\,ft$ (4.8 m)
   $T = 13.7\,sec$
   Storm surge elevation = 8.7 ft above NGVD (1929)

SOLUTION:

As shown in Example Problem VI-7-12 when $H_s = 12.8\,ft$, the damage level parameter, S in the van der Meer (1988) for surging waves was about 10 and this was also the average damage level measured by survey. For redesign, consider the following four cases:

a. Wave Height, $H_s = 13.0\,ft$
   Wave Period, $T_p = 22.0\,sec$

b. Consider what other wave periods will have on the armor stone size, keeping $H_s = 13.0\,ft$

c. Consider what effect storm duration up to $N_{max}$ = 7500 waves will have on the stone size for $H_s = 13.0\,ft$ and the critical periods.

d. Consider what effect some increase in allowable damage level, S has on these results.

Case a. From Eqn VI-6-45 in Table VI-6-6, for $H_s = 13.0\,ft$ and $T_p = 22.0\,sec$ ($T_m = 18.3\,sec$)
$W_{50} = 54{,}236\,lbs$ (27 tons)
$D_{n50} = 6.9\,ft$
with no safety factor.

EXAMPLE PROBLEM VI-7-13 (continued)

Assuming our knowledge of wave conditions is fairly good ($C_w = 0.05$) and using a failure probability ($P_f$) of 0.10 gives $\gamma_p = 1.3$ and $\gamma_z = 1.02$. For these partial safety factors in Eqn VI-6-45 gives

$$W_{50} = 126{,}490 \text{ lbs } (63 \text{ tons})$$
$$D_{n50} = 9.1 \text{ ft}$$

This is not a practical size in the quarry and for construction

Case b. From Eqn VI-6-45 for $H_s = 13.0$ ft and other wave periods, $T_p$ with no partial safety factors

| Mean Wave Period $T_m$, sec | Peak Wave Period $T_p$, sec | Armor Layer: Stone wt, $W_{50}$, lbs | Armor Layer: Stone Diam $D_n$, ft | van der Meer Formula S = Surging P = Plunging | $N_z = 2660$ Storm Duration t, Hrs | Remarks |
|---|---|---|---|---|---|---|
| 18.3 | 22 | 54,236 | 6.90 | S | 12.0 | |
| 16.7 | 20 | 55,736 | 6.96 | S | 10.9 | The Hudson formula does not consider wave period |
| 14.9 | 18 | 57,656 | 7.04 | S | 9.2 | |
| 13.3 | 16 | 59,695 | 7.12 | S | 8.7 | |
| 11.6 | 14 | 62,194 | 7.22 | S | 7.6 | |
| 10.0 | 12 | 53,237 | 6.85 | P | 6.6 | |
| 8.3 | 10 | 40,232 | 6.24 | P | 5.6 | |

Lowering the wave period with $H_s = 13.0$ ft slightly <u>increases</u> the armor stone weight by 15% for $T_p$ from 22 → 14 sec. The Surging equations govern.

Case c. As noted in Table VI-5-23 for $N_z \approx 7{,}500$ waves, the equilibrium damage level is approximately reached.

| Wave Period, $T_p$ sec | Armor Stone Weight $W_{50}$ lbs: $N_z = 2360$ (t = hrs) | $N_z = 3500$ (t = hrs) | $N_z = 5000$ (t = hrs) | $N_z = 7500$ (t = hrs) | Remarks |
|---|---|---|---|---|---|
| 22 | 54,236 (12.0) | 61,064 (17.8) | 67,946 (25.4) | 76,746 (38.1) | The very long durations are not physically realistic. Long durations at long periods also not realistic |
| 18 | 57,656 (9.8) | 64,868 (14.5) | 72,200 (20.7) | 81,592 (31.0) | |
| 14 | 62,194 (7.6) | 70,000 (11.3) | 77,832 (16.1) | 87,958 (24.2 | |

Again, as expected, increasing the storm duration increases the weight of the armor stone required for stable weight at the S = 2 level. Note however, that some storm durations (t > 18 hrs) are physically unrealistic for sustained, long period waves and wave heights, $H_s = 13.0$ ft.

EXAMPLE PROBLEM VI-7-13 (continued)

Case d. Consider realistic wave periods, $T_p$ = 14 - 22 sec for storms lasting 3 to 18 hours with $H_s$ = 13.0 ft. Now vary the damage level allowable to the intermediate range, S = 4 - 6 (say S = 5) for rock on COT$\alpha$ = 2 slopes:

| Damage Parameter S | ARMOR STONE WEIGHT, $W_{50}$ lbs | | | | | |
|---|---|---|---|---|---|---|
| | $N_z$ = 2360 | | | $N_z$ = 3500 | | |
| | $T_p$ = 22 sec | $T_p$ = 18 sec | $T_p$ = 14 sec | $T_p$ = 22 sec | $T_p$ = 18 sec | $T_p$ = 14 sec |
| 2 | 54,236 | 57,656 | 62,194 | 61,064 | 64,868 | 70,000 |
| 3 | 42,527 | 45,188 | 48,768 | 47,873 | 50,863 | 54,876 |
| 4 | 35,794 | 38,040 | 41,020 | 40,281 | 42,808 | 46,178 |
| 5 | 31,310 | 33,263 | 35,884 | 35,225 | 37,447 | 40,396 |
| 6 | 28,060 | 29,813 | 32,168 | 31,589 | 33,554 | 36,207 |
| 7 | 25,582 | 27,187 | 29,325 | 28,796 | 30,596 | 33,000 |

Allowing the damage level to rise up to S = 6 in effect means using roughly one-half, the weight of armor stones needed when S = 2. This is a tremendous reduction and cost savings for initial construction costs and repair costs in a balanced design (see, eg. Example Problem VI-7-XX for the procedure for optimum design of rubble mound structures when damage level expected is employed in the design).

[Note that using the Hudson (1977) formula and SPM (1984) methodology $H_{1/10}$ = 1.27 $H_s$, $K_D$ = 2.0, gives $W_{50}$ = 48,036 lbs. This is roughly equivalent to that obtained above for $T_p$ = 14 sec, S = 3 with an 11+ hour storm event]

The CBBT Island revetment was repaired in August 1994. (See Technical Specs. for repair, CBBT District, 1994). The $W_{50}$ was 13.5 tons (27,000 lbs) with allowable range 12-15 tons. No stones $W_{50}$ < 12 ton were allowed. As demonstrated above, this repair stone weight, $W_{50}$ = 13.5 tons (35% increase in weight) is on the border required but with some damage expected, again in the future. All the above is with No Partial Safety Factors in the design.

Alternatives for repair would be to use artifically manufactured concrete cubes (Table VI-5-29), Tetrapods (Table VI-5-30) or the Corps of Engineers new CORE-LOC design (Table VI-5-34). These units have greater interlocking abilities and are stable with less weight. A detailed cost analysis is necessary to justify the additional repair expense.

SUMMARY:

1. All the above was calculated keeping $H_s$ = 13.0 ft for design. Wave period, storm duration and allowable damage level are all

EXAMPLE PROBLEM VI-7-13

additional, important factors, but are not considered in the Hudson formula (1977) nor in the SPM (1984).

2. As shown in Table VI-6-6 for Eqn VI-6-45 Surging Waves and the van der Meer (1988) formulation, not including any partial safety factors (i.e. taking $\gamma_H = \gamma_F = 1.0$) is saying

   - our knowledge of wave height conditions is good ($C_W = 0.05$)
   - but we expect a damage probability, $P_f > 40\%$ sometime during the life time of the structures. This is acceptable if the damage can be repaired (economically) and if the extra risk is understood.

3. Using artifically manufactured units (concrete cubes, Tetrapods, CORE-LOC, etc.) can greatly reduce the level of risk by allowing $P_f$ to decrease (say $P_f = 0.05$) including the appropriate $\gamma_H$ and $\gamma_F$ factors and repairing the damage on the CBST Island revetments. This will be shown in Example Problem VI-7-XX.

D.R. Basco
03-22-01

EXAMPLE PROBLEM VI-7-14

FIND: The weight of armor stone placed as a permeable, nearshore breakwater with overtopped wave conditions

GIVEN: A permeable structure (nearshore, detached breakwater) has a slope of 1 on 2.5 and is subject to a design, significant wave height, $H_s$ of 2.0 m (6.56 ft) measured at a gauge located in a depth, d = 4.5 m (14.8 ft). Design wave peak period, $T_p$ = 9 sec. Design water depth at the structure toe, $d_{toe}$ = 3.0 m (9.8 ft). From Example Problem, VI-7-2 for these conditions, runup, $R_{u2\%}$ is 3.5-4.5 m above the SWL, hence some wave overtopping occurs.

SOLUTION: From Example Problem VI-7-1 (see also VI-7-10), linear wave theory shoaling to the structure toe gives

$$(H_s)_{toe} = 2.16 \text{ m } (7.0 \text{ ft})$$

ASSUMPTIONS: (See Tables VI-5-22, VI-5-24 and VI-5-25 for conventional, two-layer armor stone designs. See also Fig. VI-5-11 for notational permeability coefficients)

1. Fresh water, $\rho_w$ = 1000 kg/m³
2. Rock $\rho_r$ = 2650 kg/m³
3. Two-layers, n = 2, random placement
4. Quarry stone rough, angular
5. No damage criteria (See Table VI-5-21 for values, D and S)

1. Hudson (1974) SPM (1984)

See Example Problem VI-7-10 for results which do not change for rubble-mound revetment or nearshore breakwaters: Repeat here.

(a) Non breaking Waves $K_D$ = 4

$W_{50}$ = 11,946 N (2537 lb)

$D_{n50}$ = 0.77 m (2.5 ft)

(b) Breaking Waves, $K_D$ = 2

$W_{50}$ = 23,893 N. (5173 lbs)

$D_{n50}$ = 0.97 m (3.15 ft)

(c) Governing Conditions

Wave close to breaking since $(H_s/d)_{toe}$ = 0.7

The Hudson formula was originally for non-overtopped slopes, but has been often mis-applied to cases with moderate to substantial wave overtopping.

2. van der Meer (1991) CEM 2000

van der Meer (1991) developed a reduction factor, $f_i$ given by Eqn VI-5-71 in Table VI-5-24 to modify the original, van der Meer (1988)

EXAMPLE PROBLEM VI-7-14 (continued)

stability formula for non-overtopping rock sloped revetments. The $D_{n50}$ value is replaced by $f_i D_{n50}$ and the relative freeboard, $R_c/H_s$ plays an important role in the factor, $f_i$. Here, $R_c$ is the same as defined for overtopping depicted in Figure VI-5-14.

(a) non-overtopping conditions — impermeable revetment

Recall from Example Problem VI-7-10 when

$$P = 0.1 \quad \text{(impermeable)}$$
$$N_z \geq 7500 \quad (t = 16\text{-}17 \text{ hrs})$$
$$S = 2 \quad \text{(no damage condition)}$$
$$\xi_m < \xi_{mc}$$

then for Plunging Wave condition:

$$D_{n50} = 1.02 \text{ m} \quad (3.306 \text{ ft})$$
$$W_{50} = 27{,}345 \text{ kN} \quad (5976 \text{ lbs})$$

(b) overtopping conditions

The van der Meer (1991) equations can be written

$$\frac{H_s}{\Delta f_i D_{n50}} = 6.2\, S^{0.2} P^{0.18} N_z^{-0.1} \xi_m^{-0.5} \qquad \text{Plunging } (\xi_m < \xi_{mc})$$

and/or

$$\frac{H_s}{\Delta f_i D_{n50}} = 1.0\, S^{0.2} P^{-0.13} N_z^{-0.1} (\cot\alpha)^{0.5} \xi_m^{P} \qquad \text{Surging } (\xi_m > \xi_{mc})$$

where:

$$f_i = \left(1.25 - 4.8 \frac{R_c}{H_s}\sqrt{\frac{S_{op}}{2\pi}}\right)^{-1} \qquad (\text{VI-5-71})$$

within limits

$$0 < \frac{R_c}{H_s}\sqrt{\frac{S_{op}}{2\pi}} < 0.052$$

Note that now, the peak period wave steepness, $S_{op}$ is employed. It is convenient to set up a spread sheet solution to investigate how $D_{n50}$ and $W_{50}$ vary with relative freeboard, $R_c/H_s$. We first consider the case (unlikely) for an impermeable, nearshore breakwater design

(i) Impermeable, $P = 0.1$

| Relative Freeboard, $R_c/H_s$ | $R_c$ m | $D_{n50}$ m | $W_{50}$ N | $f_i$ | Remarks |
|---|---|---|---|---|---|
| 1.0 | 2.16 | 1.021 | 27,663 | 1.000 | Exceeds Limit 0.052 |
| 0.89 | 1.9224 | 1.021 | 27,663 | 1.0002 | Limit Value = 0.052116 |
| 0.75 | 1.62 | 0.9825 | 24,655 | 0.962 | |
| 0.50 | 1.08 | 0.920 | 20,263 | 0.901 | |
| 0 | 0 | 0.317 | 14,167 | 0.800 | Limit Value = 0 |

EXAMPLE PROBLEM VI-7-14 (continued)

We see that as the relative freeboard, $R_c/H_s$ decreases, more wave overtopping occurs, and the stable armor-layer weight also decreases. Over the limiting factor range $0.8 \leq f_i \leq 1.0$. Note that for $R_c/H_s > 0.89$, $f_i = 1.0$ limit is reached and the van der Meer (1991) formulation returns to that originally proposed for the non-overtopping conditions.

(ii) Permeable, P = 0.4 or 0.5

The great achievement of the original van der Meer (1988) formulation with modification by the reduction factor, $f_i$ is in applications to permeable structures, such as nearshore breakwaters. We repeat the above results but now for permeable structures.

P = 0.4 (See Fig VI-5-11)

| Relative Freeboard $R_c/H_s$ | $R_c$ (m) | $D_{n50}$ (m) | $W_{50}$ N | $f_i$ | Limit Parameter | Remarks |
|---|---|---|---|---|---|---|
| 1.0 | NOT APPLICABLE | | | | >0.052 | |
| 0.89 | 1.9224 | 0.796 | 13,087 | 1.0002 | 0.0521 | Going from P=0.1 (impermeable) to P=0.4 (permeable) produces a 50% or one-half lower weight requirement. |
| 0.85 | 1.836 | 0.787 | 12,662 | 0.989 | 0.0497 | |
| 0.80 | 1.728 | 0.776 | 12,147 | 0.976 | 0.047 | |
| 0.75 | 1.62 | 0.766 | 11,661 | 0.962 | | |
| 0.50 | 1.08 | 0.717 | 9,582 | 0.901 | | |
| 0.25 | 0.54 | 0.674 | 7,970 | 0.848 | | |
| 0.10 | 0.216 | 0.651 | 7,175 | 0.818 | | |
| 0.05 | 0.108 | 0.644 | 6,933 | 0.809 | | |
| 0 | 0 | 0.636 | 6,700 | 0.800 | 0 | |

P = 0.5 (Permeable, $D_{core} = 0.3\ D_{armor}$

| | | | | | | |
|---|---|---|---|---|---|---|
| 1.0 | NOT APPLICABLE | | | | >0.052 | |
| 0.89 | Same | 0.764 | 11,602 | 1.0002 | | Going from P=0.4 to P=0.5 gives a 11.3% drop in $W_{50}$ |
| 0.85 | | | | | | |
| 0.80 | | 0.746 | 10,771 | 0.976 | | |
| 0.75 | | | | | | |
| 0.50 | | 0.689 | 8,496 | 0.901 | | |
| 0.25 | | | | | | |
| 0.10 | | 0.625 | 6,359 | 0.818 | | |
| 0.05 | | | | | | |
| 0 | | 0.611 | 5,941 | 0.800 | | |

The value of P=0.6 is reserved for permeable breakwaters built with no core and homogeneous sized units as discussed in Example Problem VI-7-15.

We now see the significant (50% or more) reduction in the stable weight requirements due to the permeability, P of the typical, nearshore breakwater designs with a core layer



22-141 50 SHEETS
22-142 100 SHEETS
22-144 200 SHEETS
AMPAD

EXAMPLE PROBLEM VI-7-14 (continued)

SUMMARY :

1. The Hudson formula is not applicable for wave over topping conditions
2. Use the van der Meer (1991) formula with $D_{n50} = f_i D_{n50}$ as an extension of van der Meer (1988).
3. The reduction factor lies in the range $0.8 < f_i < 1.0$ where

$$f_i = 0.8 \quad \text{at } R_c/H_s = 0, \quad \text{zero freeboard}$$

and

$$f_i = 1.0 \quad \text{at } \frac{R_c}{H_s}\sqrt{\frac{S_{op}}{2\pi}} = 0.052 \quad \text{limit}$$

4. At limit of zero freeboard, $R_c = 0$, $f_i = 0.8$

$$W_{50} \sim D_{n50}^3$$

$\therefore$ Weight reduction $= (0.8)^3 = 0.512$ or almost a 50% drop.

5. At limit $f_i = 1.0$

$$S_{op} = \frac{H_s}{L_{op}} = \frac{H_s}{\frac{g}{2\pi} T_p^2}$$

therefore

$$\text{Lim} = \frac{R_c}{H_s}\left(\frac{H_s}{\frac{g}{2\pi} T_p^2 \, 2\pi}\right)^{1/2}$$

$$\text{Lim} = \frac{R_c}{H_s}\left(\frac{H_s}{g T_p^2}\right)^{1/2} < 0.052$$

so that for a range of practical wave heights $1.0 < H_s < 10.0$ m and peak periods, $3 < T_p < 21$ sec, a table can be prepared to calculate the maximum values of $R_c/H_s$ at the 0.052 limit. This gives a practical range of $(R_c/H_s)_{max}$ values as shown in the table below

| $H_s$, m | $(R_c/H_s)_{max}$ Values at limit = 0.052 — Peak Period, $T_p$ | | | | | | |
|---|---|---|---|---|---|---|---|
| | 3 | 6 | 9 | 12 | 15 | 18 | 21 |
| 1.0 | 0.49 | 0.98 | 1.49 | 1.95 | 2.44 | 2.93 | 3.42 |
| 2.0 | 0.35 | 0.69 | 1.04 | 1.38 | 1.73 | 2.07 | 2.42 |
| 3.0 | 0.28 | 0.56 | 0.85 | 1.13 | 1.41 | 1.69 | 1.97 |
| 4.0 | 0.24 | 0.49 | 0.73 | 0.98 | 1.22 | 1.47 | 1.71 |
| 5.0 | 0.22 | 0.44 | 0.66 | 0.88 | 1.09 | 1.31 | 1.53 |
| 7.0 | 0.19 | 0.37 | 0.55 | 0.74 | 0.92 | 1.11 | 1.29 |
| 10.0 | 0.15 | 0.31 | 0.46 | 0.61 | 0.77 | 0.93 | 1.08 |

For $(R_c/H_s)$ values greater than in the above table, non-overtopping conditions prev

**Table VI-5-24**
Rock, two-layer armored overtopped, but not submerged, low-crested slopes.

Powell and Allsop (1985) analyzed data by Allsop (1983) and proposed the stability formula

$$\frac{N_{od}}{N_a} = a \exp\left[b\, s_p^{-1/3} H_s / (\Delta D_{n50})\right] \quad \text{or} \quad \frac{H_s}{\Delta D_{n50}} = \frac{s_p^{1/3}}{b} \ln\left(\frac{1}{a}\frac{N_{od}}{N_a}\right) \qquad \text{(VI-5-70)}$$

where values of the empirical coefficients $a$ and $b$ are given in the table as functions of freeboard $R_c$ and water depth $h$. $N_{od}$ and $N_a$ are the number of units displaced out of the armor layer and the total number of armor layer units, respectively.

**Values of coefficients $a$ and $b$ in Eqn. VI-5-70.**

| $R_c/h$ | $a \cdot 10^4$ | $b$ | wave steepness $H_s/L_p$ |
|---|---|---|---|
| 0.29 | 0.07 | 1.66 | <0.03 |
| 0.39 | 0.18 | 1.58 | <0.03 |
| 0.57 | 0.09 | 1.92 | <0.03 |
| 0.38 | 0.59 | 1.07 | >0.03 |

van der Meer (1991) suggested that the van der Meer stability formulae for non-overtopped rock slope, Eqns. VI-5-68 and VI-5-69, be used with $D_{n50}$ replaced by $f_i D_{n50}$. The reduction factor $f_i$ is given as

$$f_i = \left(1.25 - 4.8\,\frac{R_c}{H_s}\sqrt{\frac{s_{op}}{2\pi}}\right)^{-1} \qquad \text{(VI-5-71)}$$

where $R_c$ is the freeboard, $s_{op} = H_s/L_{op}$, and $L_{op}$ is deep water wave length corresponding to the peak wave period. Limits of Eqn. VI-5-71 are given by

$$0 < \frac{R_c}{H_s}\sqrt{\frac{s_{op}}{2\pi}} < 0.052$$

VAN DER MEER(1991) CEM(2000)
**Table VI-5-24**
**Eqn VI-5-71 Overtopped-Plunging**
*Enter initial conditions*
Quarry stone,rough angular
n=2 layers, random placement
Structure Trunk
COT alpha = 1.5 to 5.0
Damage conditions specified by S

ARMOUR LAYER STABILITY
Rubble-Mound Structures and Overtopped,But Not Submerged
(Metric System)

| **Enter Data** | | | Units | **Additional Data** | | | Units |
|---|---|---|---|---|---|---|---|
| Wave Height,Hs | = | 2.16 | meters | Nominal Damage,S = | | 2.0 | |
| Rho for rock | = | 2650 | Kn/m^3 | Permeability,P | = | 0.1 | |
| Gravity | = | 9.81 | m/sec^2 | No. of Waves,Nz | = | 7500 | |
| Rho for water | = | 1000 | Kn/m^3 | Wave Period,Tm | = | 6.65 | sec |
| Delta = Sr-1 | = | 1.65 | | Freeboard,Rc | = | 0.000 | meters |
| Gamma Z | = | 1.00 | | **Calculated Information** | | | Units |
| Gamma H | = | 1.00 | | Wave Length,Lom | = | 69.05 | meters |
| | | | | Steepness,Som | = | 0.03130 | |
| | | | | Peak period,Tp | = | 8.01325 | sec |
| | | | | Storm duration,t | = | 13.8542 | hrs |
| | | | | Peak Steepness,Sop | = | 0.021545 | |

Remarks: Eta must be less than Eta critical
Limiting Value < 0.052
**Hs,Significant Wave Height**

| SLOPE COT alpha | SURF PRA Eta | DIAMETER meters | WEIGHT KN | Eta critical | REMARKS | |
|---|---|---|---|---|---|---|
| 1.5 | 3.7682228 | 1.0545 | **30482.8** | 4.541961 | **OK** | 1.24 |
| 2.0 | 2.8261671 | 0.9132 | **19797.6** | 3.57378 | **OK** | 1.43 |
| 2.5 | 2.2609337 | 0.8168 | **14166.5** | 2.967355 | **OK** | 1.60 |
| 3.0 | 1.8841114 | 0.7456 | **10775.4** | 2.54909 | **OK** | 1.76 |
| 4.0 | 1.4130835 | 0.6457 | **6998.6** | 2.005716 | **OK** | 2.03 |
| 5.0 | 1.1304668 | 0.5776 | **5009.5** | 1.665372 | **OK** | 2.27 |

**REDUCTIONFACTOR f** 0.8 **LIMIT**
Limiting Value = 0 0.052
Set fi = 0.8000

VAN DER MEER(1991) CEM(2000)
**Table VI-5-24**
**Eqn VI-5-71 Overtopped-Plunging**
*Enter initial conditions*
Quarry stone,rough angular
n=2 layers, random placement
Structure Trunk
COT alpha = 1.5 to 5.0
Damage conditions specified by S

ARMOUR LAYER STABILITY
Rubble-Mound Structures and Overtopped,But Not Submerged
(Metric System)

| **Enter Data** | | | Units |
|---|---|---|---|
| Wave Height,Hs | = | 2.16 | meters |
| Rho for rock | = | 2650 | Kn/m^3 |
| Gravity | = | 9.81 | m/sec^2 |
| Rho for water | = | 1000 | Kn/m^3 |
| Delta = Sr-1 | = | 1.65 | |
| Gamma Z | = | 1.00 | |
| Gamma H | = | 1.00 | |

| **Additional Data** | | | Units |
|---|---|---|---|
| Nominal Damage,S | = | 2.0 | |
| Permeability,P | = | 0.1 | |
| No. of Waves,Nz | = | 7500 | |
| Wave Period,Tm | = | 6.65 | sec |
| Freeboard,Rc | = | 1.920 | meters |
| **Calculated Information** | | | Units |
| Wave Length,Lom | = | 69.05 | meters |
| Steepness,Som | = | 0.03130 | |
| Peak period,Tp | = | 8.01325 | sec |
| Storm duration,t | = | 13.8542 | hrs |
| Peak Steepness,Sop | = | 0.021545 | |

Remarks: Eta must be less than Eta critical
Limiting Value < 0.052
**Hs,Significant Wave Height**

| SLOPE COT alpha | SURF PRA Eta | DIAMETER meters | WEIGHT KN | Eta critical | REMARKS | |
|---|---|---|---|---|---|---|
| 1.5 | 3.7682228 | 1.3179 | **59506.2** | 4.541961 | **OK** | 0.99 |
| 2.0 | 2.8261671 | 1.1413 | **38646.9** | 3.57378 | **OK** | 1.15 |
| 2.5 | 2.2609337 | 1.0208 | **27652.7** | 2.967355 | **OK** | 1.28 |
| 3.0 | 1.8841114 | 0.9319 | **21038.9** | 2.54909 | **OK** | 1.4 |
| 4.0 | 1.4130835 | 0.807 | **13662.7** | 2.005716 | **OK** | 1.62 |
| 5.0 | 1.1304668 | 0.7218 | **9776.1** | 1.665372 | **OK** | 1.81 |

**REDUCTIONFACTOR f** 0.999846 **LIMIT**
Limiting Value = 0.052051 0.052
Set fi = 0.9998

VAN DER MEER(1991) CEM(2000)
**Table VI-5-24**
**Eqn VI-5-71 Overtopped-Plunging**
*Enter initial conditions*
Quarry stone,rough angular
n=2 layers, random placement
Structure Trunk
COT alpha = 1.5 to 5.0
Damage conditions specified by S

ARMOUR LAYER STABILITY
Rubble-Mound Structures and Overtopped,But Not Submerged
(Metric System)

| **Enter Data** | | | Units | **Additional Data** | | | Units |
|---|---|---|---|---|---|---|---|
| Wave Height,Hs | = | 2.16 | meters | Nominal Damage,S = | | 2.0 | |
| Rho for rock | = | 2650 | Kn/m^3 | Permeability,P | = | 0.5 | |
| Gravity | = | 9.81 | m/sec^2 | No. of Waves,Nz | = | 7500 | |
| Rho for water | = | 1000 | Kn/m^3 | Wave Period,Tm | = | 6.65 | sec |
| Delta = Sr-1 | = | 1.65 | | Freeboard,Rc | = | 0.000 | meters |
| Gamma Z | = | 1.00 | | **Calculated Information** | | | Units |
| Gamma H | = | 1.00 | | Wave Length,Lom | = | 69.05 | meters |
| | | | | Steepness,Som | = | 0.03130 | |
| | | | | Peak period,Tp | = | 8.01325 | sec |
| | | | | Storm duration,t | = | 13.8542 | hrs |
| | | | | Peak Steepness,Sop = | | 0.021545 | |

Remarks: Eta must be less than Eta critical
Limiting Value < 0.052
**Hs,Significant Wave Height**

| SLOPE COT alpha | SURF PRA Eta | DIAMETER meters | WEIGHT KN | Eta critical | REMARKS | |
|---|---|---|---|---|---|---|
| 1.5 | 3.7682228 | 0.7893 | **12783.2** | 4.083445 | **OK** | 1.66 |
| 2.0 | 2.8261671 | 0.6835 | **8301.0** | 3.536368 | **OK** | 1.92 |
| 2.5 | 2.2609337 | 0.6114 | **5941.4** | 3.163023 | **OK** | 2.14 |
| 3.0 | 1.8841114 | 0.5581 | **4519.1** | 2.887432 | **OK** | 2.35 |
| 4.0 | 1.4130835 | 0.4833 | **2934.7** | 2.500589 | **OK** | 2.71 |
| 5.0 | 1.1304668 | 0.4323 | **2100.2** | 2.236595 | **OK** | 3.03 |

**REDUCTIONFACTOR f** 0.8 **LIMIT**
Limiting Value = 0 0.052
Set fi = 0.8000

VAN DER MEER(1991) CEM(2000)
**Table VI-5-24**
**Eqn VI-5-71 Overtopped-Plunging**
*Enter initial conditions*
Quarry stone,rough angular
n=2 layers, random placement
Structure Trunk
COT alpha = 1.5 to 5.0
Damage conditions specified by S

ARMOUR LAYER STABILITY
Rubble-Mound Structures and Overtopped,But Not Submerged
(Metric System)

| **Enter Data** | | | Units |
|---|---|---|---|
| Wave Height,Hs | = | 2.16 | meters |
| Rho for rock | = | 2650 | Kn/m^3 |
| Gravity | = | 9.81 | m/sec^2 |
| Rho for water | = | 1000 | Kn/m^3 |
| Delta = Sr-1 | = | 1.65 | |
| Gamma Z | = | 1.00 | |
| Gamma H | = | 1.00 | |

| **Additional Data** | | | Units |
|---|---|---|---|
| Nominal Damage,S | = | 2.0 | |
| Permeability,P | = | 0.5 | |
| No. of Waves,Nz | = | 7500 | |
| Wave Period,Tm | = | 6.65 | sec |
| Freeboard,Rc | = | 1.920 | meters |
| **Calculated Information** | | | Units |
| Wave Length,Lom | = | 69.05 | meters |
| Steepness,Som | = | 0.03130 | |
| Peak period,Tp | = | 8.01325 | sec |
| Storm duration,t | = | 13.8542 | hrs |
| Peak Steepness,Sop | = | 0.021545 | |

Remarks: Eta must be less than Eta critical
Limiting Value < 0.052
**Hs,Significant Wave Height**

| SLOPE COT alpha | SURF PRA Eta | DIAMETER meters | WEIGHT KN | Eta critical | REMARKS | |
|---|---|---|---|---|---|---|
| 1.5 | 3.7682228 | 0.9864 | **24950.2** | 4.083445 | **OK** | 1.33 |
| 2.0 | 2.8261671 | 0.8543 | **16208.6** | 3.536368 | **OK** | 1.53 |
| 2.5 | 2.2609337 | 0.7641 | **11597.5** | 3.163023 | **OK** | 1.71 |
| 3.0 | 1.8841114 | 0.6975 | **8821.6** | 2.887432 | **OK** | 1.88 |
| 4.0 | 1.4130835 | 0.6041 | **5731.1** | 2.500589 | **OK** | 2.17 |
| 5.0 | 1.1304668 | 0.5403 | **4100.3** | 2.236595 | **OK** | 2.42 |

**REDUCTIONFACTOR f** 0.999846 **LIMIT**
Limiting Value = 0.052051 0.052
Set fi = 0.9998

The core permeability in Table VI-5-4 refers to the structures shown in details *a*, *c* and *d* of Figure VI-5-11, taken from van der Meer (1988). The figure provides definition of a notational permeability parameter *P* which is used in various formulae by van der Meer to take into account the effect of permeability on response to wave action. The value $P = 0.4$ in Figure VI-5-11, detail *b*, is not identified by tests, but instead is an estimated value.

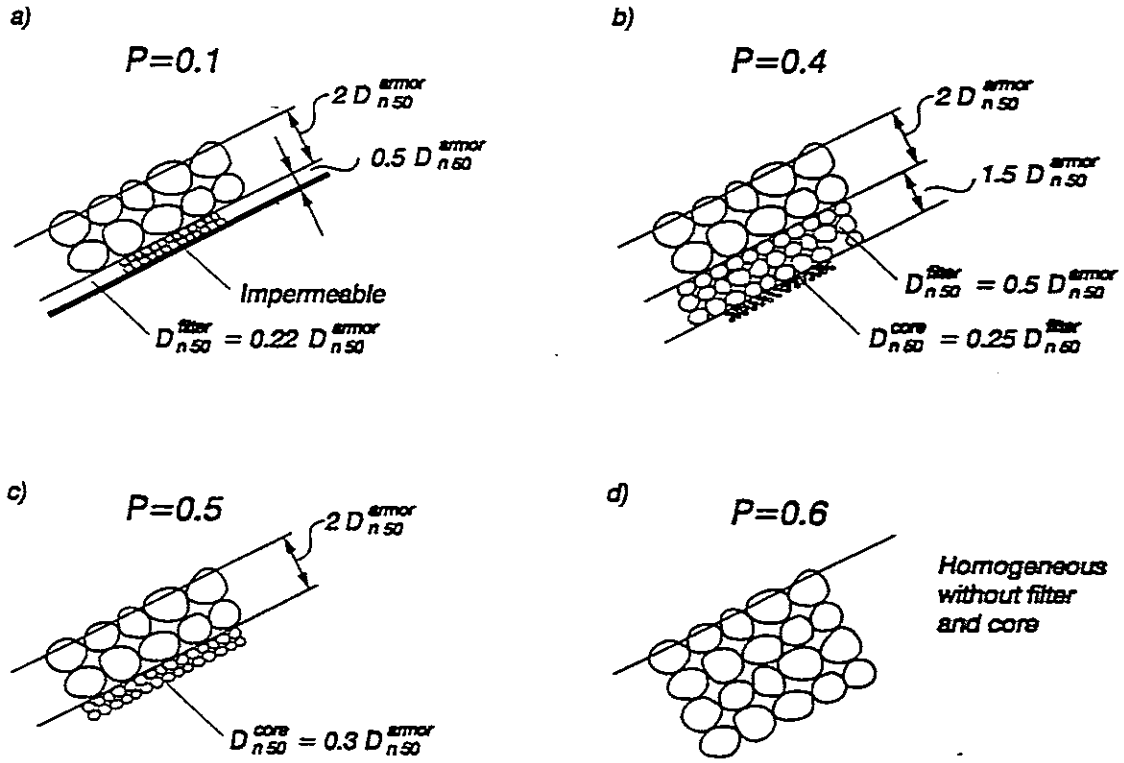



Figure VI-5-11. Notational permeability coefficients. van der Meer (1988).

EXAMPLE PROBLEM VI-7-15

FIND: The weight of graded riprap placed as a permeable, nearshore breakwater with <u>submerged</u> water level conditions.

GIVEN: The same data as for EXAMPLE PROBLEM VI-7-14 except now the design water depth, $d_{toe}$ at the structure toe <u>increases</u> to submerge the structure, i.e.

$\cot\alpha = 2.5$

$(H_s)_{toe} = 2.16\,m$ (7.0 ft)

$T_p = 8$ sec

and $\rho_w = 1000\ kg/m^3$ (fresh water)

$\rho_r = 2650\ kg/m^3$ (rock)

$n = 2$ layers, random placement

Quarrystone, rough angular

No damage criteria, $S = 2$

$d_{toe} = 3.0\,m$ (in EX. PROB. VI-7-14)

SOLUTION:

1. <u>van der Meer (1991) CEM 2000</u>

For irregular, head-on waves and data for $\cot\alpha = 1.5, 2$ slopes, van der Meer (1991) developed the formula

$$\frac{h_c'}{h} = (2.1 + 0.1\,S)\exp(-0.14\,N_s^*) \qquad (VI\text{-}5\text{-}72)$$

where

$h_c'$ = crest height of structure above sea bed

$h$ = water depth

$\therefore\ h_c' - h$ = water depth over the structure crest

$S$ = relative eroded area, (damage level)

and

$N_s^*$ = spectral stability number

$$= \frac{H_s}{\Delta D_{n50}} S_p^{-1/3} \qquad VI\text{-}5\text{-}72(a)$$

with $S_p = H_s/L_{op}$

To determine the stable, armor stone diameter and weight, Eqn VI-5-72 is solved first for a given $h_c'/h$ ratio and $S$ value to calculate $N_s^*$. Then, Eqn VI-5-72(a) is solved for $D_{n50}$. A spread sheet solution aids the calculation process. Note that the slope does not enter into the calculation and the empirical formula VI-5-72 has only been developed for two slopes.

Assumptions:
1. Assume VI-5-72 also applicable when $\cot\alpha = 2.5$
2. Using $S = 2$ gives the no damage results
3. Assume crest height, $h_c' = 5.0\,m$ above sea bed.

EXAMPLE PROBLEM VI-7-15 (continued)

The inverse of the $h_c'/h$ ratio in Eqn VI-5-72 is the relative submergence ratio $h/h_c' \geq 1.0$. At $h/h_c' = 1.0$ the water level is at the structure crest and this condition is equal to $R_c = 0$ or relative freeboard, $R_c/H_s$ in Eqn VI-5-71 when $f_i = 0.8$. (see previous Example Problem VI-7-14). Since the given data is the same for both Example Problem VI-7-14 and this problem VI-7-15, then the rock size $D_{n50}$ and weight, $W_{50}$ should coincide at these extreme limits of these equations.

The table below demonstrates that $D_{n50}$ and $W_{50}$ are far too big at the extreme end where $h/h_c' = 1.0$ using the submerged Eqn VI-5-72.

| $\frac{h_c'}{h}$ | $\frac{h}{h_c'}$ | Water Depth, h (m) | Diameter $D_{n50}$, (m) | Weight $W_{50}$, KN | $N_s^*$ | Remarks |
|---|---|---|---|---|---|---|
| 1.1 | Overtopped, not submerged. Use Eqn VI-5-71, $h_c' = 5.0$ | | | | | |
| 1.0 | 1.000 | 5.00 | 0.791 | 12,866 | 5.95 | $R_c = 0$ |
| 0.998 | 1.002 | 5.01 | 0.789 | 12,769 | 5.96 | |
| 0.91 | 1.10 | 5.50 | 0.710 | 9,304 | 6.63 | |
| 0.833 | 1.20 | 6.00 | 0.649 | 7,106 | 7.25 | |
| 0.77 | 1.30 | 6.50 | 0.602 | 5,672 | 7.82 | |
| 0.71 | 1.40 | 7.00 | 0.564 | 4,663 | 8.35 | |
| 0.66 | 1.50 | 7.50 | 0.532 | 3,914 | 8.85 | |
| 0.625 | 1.60 | 8.00 | 0.506 | 3,368 | 9.3 | |
| 0.555 | 1.80 | 9.00 | 0.464 | 2,597 | 10.15 | |
| 0.50 | 2.00 | 10.00 | 0.432 | 2,096 | 10.94 | |

Note that $N_s^*$ max tested was about 13.5 @ $h_c'/h = 0.4$.

Now from Eqn VI-5-71 at the limit when $R_c/H_s = 0$ (zero freeboard) as shown in Example Problem VI-7-14

| P | $D_{n50}$ (m) | $W_{50}$ KN | Remarks |
|---|---|---|---|
| 0.1 | 0.817 | 14,167 | Similar to 12,866 kN above |
| 0.4 | 0.636 | 6,700 | Way low |
| 0.5 | 0.611 | 5,941 | Way low |

We now make two assumptions that need further confirmation:

1. Eqn VI-5-72 for submerged conditions has been developed for impermeable breakwaters, i.e. implied is $P = 0.1$. The notational permeability parameter does not appear in Eqn VI-5-72.

2. We modify Eqn VI-5-72 to

$$\frac{h_c'}{h} = \left[(2.0 + P) + 0.1S\right] \exp\left(-0.14 N_s^*\right) \qquad \text{VI-5-72}^*$$

EXAMPLE PROBLEM VI-7-15 (continued)

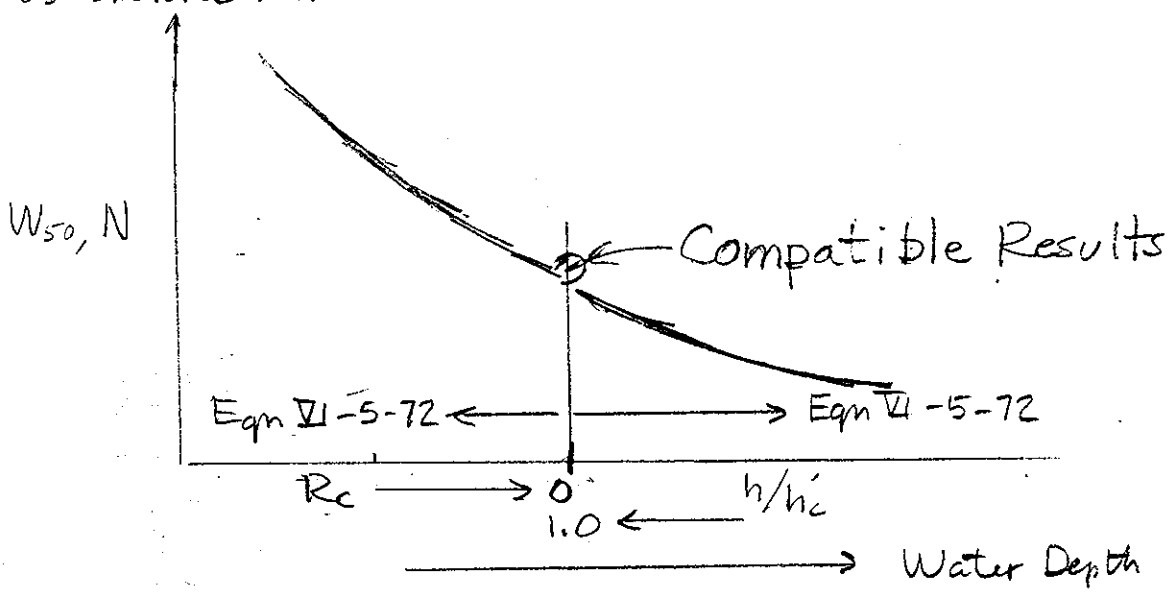
to account for the permeability of rubble-mound, nearshore breakwaters that become submerged. In this case, when P = 0.1 the original Eqn VI-5-72 is recovered. And, for permeable breakwaters say P = 0.5, recalculation of the above table gives

| $\frac{h_c'}{h}$ | $\frac{h}{h_c'}$ | Water Depth, h (m) | Diam $D_{n50}$ (m) | Weight $W_{50}$ N | $N_s^*$ | Rmks |
|---|---|---|---|---|---|---|
| 1.1 | Overtopped, not submerged, Use Eqn VI-5-71, $h_c' = 5.0$ | | | | | |
| 1.0 | 1.000 | 5.00 | 0.664 | 7610. | 1.97 | Use Overtopping |
| 0.998 | 1.002 | 5.01 | 0.662 | 7542. | 1.98 | |
| 0.91 | 1.10 | 5.50 | 0.605 | 5757. | 2.16 | |
| 0.833 | 1.20 | 6.00 | 0.561 | 4590. | 2.33 | |
| 0.770 | 1.30 | 6.50 | 0.525 | 3762. | 2.49 | |
| 0.710 | 1.40 | 7.00 | 0.496 | 3172. | 2.64 | |
| 0.66 | 1.50 | 7.50 | 0.471 | 2716. | 2.78 | |
| 0.625 | 1.60 | 8.00 | 0.450 | 2368. | 2.91 | |
| 0.555 | 1.80 | 9.00 | 0.417 | 1885. | 3.14 | |
| 0.50 | 2.00 | 10.00 | 0.391 | 1554. | 3.35 | |

We now have compatibility for the same water levels and notational permeability conditions, e.g.

| Equation | P | Water Depth (m) | $R_c/H_s$ ($h/h_c'$) | $D_{n50}$ (m) | $W_{50}$ (N) |
|---|---|---|---|---|---|
| VI-5-71 | 0.4-0.5 | 5 | 0.0 | 0.636-0.611 | 67,00 - 59,40 |
| VI-5-72 | 0.5 | 5 | (1.0) | 0.664 | 7610 |

The results are not exact because the formulations, data and coefficients are completely different but they are compatiable as sketched below




D.R. Basco
~~03/22/01~~
05/06

VAN DER MEER(1991) CEM(2000)
**Table VI-5-25**
**Eqn VI-5-72 Submerged Breakwters**
*Enter initial conditions*
Quarry stone,rough angular
n=2 layers, random placement
Structure Trunk
COT alpha = 1.5 to 5.0
Damage conditions specified by S

ARMOUR LAYER STABILITY
Rubble-Mound Structures and Submerged Slopes
(Metric System)

| **Enter Data** | | | Units |
|---|---|---|---|
| Wave Height,Hs | = | 2.16 | meters |
| Rho for rock | = | 2650 | Kn/m^3 |
| Gravity | = | 9.81 | m/sec^2 |
| Rho for water | = | 1000 | Kn/m^3 |
| Delta = Sr-1 | = | 1.65 | |
| Gamma Z | = | 1.00 | |
| Gamma H | = | 1.00 | |

| **Additional Data** | | | Units |
|---|---|---|---|
| Nominal Damage,S = | | 2.0 | |
| Water Depth, h | = | 7.50 | meters |
| Crest Ele. hc prime = | | 5.00 | meters |
| Wave Period,Tm | = | 6.65 | sec |
| Number of Waves | = | 7500.0 | |
| **Calculated Information** | | | Units |
| Wave Length,Lom | = | 69.05 | meters |
| Steepness,Som | = | 0.0313 | |
| Peak period,Tp | = | 8.01325 | sec |
| Storm duration,t | = | 13.854 | hrs |
| Peak Steepness,Sop = | | 0.0215 | |
| ***Relative Submergence, h/hc prim*** | | | 1.5 |
| **Notational Permeability,P=** | | | **0.5** |

| SLOPE COT alpha | **Hs,Significant Wave Height** Spectral No N(star)s | DIAMETER meters | WEIGHT KN | | **REMARKS** |
|---|---|---|---|---|---|
| 1.5 | 9.9908 | 0.471 | **2716.3** | 2.78 | **OK** |
| 2.0 | 9.9908 | 0.471 | **2716.3** | 2.78 | **OK** |
| 2.5 | 9.9908 | 0.471 | **2716.3** | 2.78 | **OK** |
| 3.0 | 9.9908 | 0.471 | **2716.3** | 2.78 | **OK** |
| 4.0 | 9.9908 | 0.471 | **2716.3** | 2.78 | **OK** |
| 5.0 | 9.9908 | 0.471 | **2716.3** | 2.78 | **OK** |

VAN DER MEER(1991) CEM(2000)
**Table VI-5-25**
**Eqn VI-5-72 Submerged Breakwters**
*Enter initial conditions*
Quarry stone,rough angular
n=2 layers, random placement
Structure Trunk
COT alpha = 1.5 to 5.0
Damage conditions specified by S

ARMOUR LAYER STABILITY
Rubble-Mound Structures and Submerged Slopes
(Metric System)

| **Enter Data** | | | Units |
|---|---|---|---|
| Wave Height,Hs | = | 2.16 | meters |
| Rho for rock | = | 2650 | Kn/m^3 |
| Gravity | = | 9.81 | m/sec^2 |
| Rho for water | = | 1000 | Kn/m^3 |
| Delta = Sr-1 | = | 1.65 | |
| Gamma Z | = | 1.00 | |
| Gamma H | = | 1.00 | |

| **Additional Data** | | | Units |
|---|---|---|---|
| Nominal Damage,S = | | 2.0 | |
| Water Depth, h | = | 7.50 | meters |
| Crest Ele. hc prime = | | 5.00 | meters |
| Wave Period,Tm | = | 6.65 | sec |
| Number of Waves | = | 7500.0 | |
| **Calculated Information** | | | Units |
| Wave Length,Lom | = | 69.05 | meters |
| Steepness,Som | = | 0.0313 | |
| Peak period,Tp | = | 8.01325 | sec |
| Storm duration,t | = | 13.854 | hrs |
| Peak Steepness,Sop = | | 0.0215 | |
| ***Relative Submergence, h/hc prim*** | | | 1.5 |
| **Notational Permeability,P=** | | | **0.1** |

| SLOPE COT alpha | **Hs,Significant Wave Height** Spectral No N(star)s | DIAMETER meters | WEIGHT KN | | **REMARKS** |
|---|---|---|---|---|---|
| 1.5 | 8.8455 | 0.532 | **3914.26** | 2.46 | **OK** |
| 2.0 | 8.8455 | 0.532 | **3914.26** | 2.46 | **OK** |
| 2.5 | 8.8455 | 0.532 | **3914.26** | 2.46 | **OK** |
| 3.0 | 8.8455 | 0.532 | **3914.26** | 2.46 | **OK** |
| 4.0 | 8.8455 | 0.532 | **3914.26** | 2.46 | **OK** |
| 5.0 | 8.8455 | 0.532 | **3914.26** | 2.46 | **OK** |

VAN DER MEER(1991) CEM(2000)
**Table VI-5-25**
**Eqn VI-5-72 Submerged Breakwters**
*Enter initial conditions*
Quarry stone,rough angular
n=2 layers, random placement
Structure Trunk
COT alpha = 1.5 to 5.0
Damage conditions specified by S

ARMOUR LAYER STABILITY
Rubble-Mound Structures and Submerged Slopes
(Metric System)

**Enter Data**

| | | | Units |
|---|---|---|---|
| Wave Height,Hs | = | 2.16 | meters |
| Rho for rock | = | 2650 | Kn/m^3 |
| Gravity | = | 9.81 | m/sec^2 |
| Rho for water | = | 1000 | Kn/m^3 |
| Delta = Sr-1 | = | 1.65 | |
| Gamma Z | = | 1.00 | |
| Gamma H | = | 1.00 | |

**Additional Data**

| | | Units |
|---|---|---|
| Nominal Damage,S = | 2.0 | |
| Water Depth, h = | 5.01 | meters |
| Crest Ele. hc prime = | 5.00 | meters |
| Wave Period,Tm = | 6.65 | sec |
| Number of Waves = | 7500.0 | |

**Calculated Information**

| | | Units |
|---|---|---|
| Wave Length,Lom = | 69.05 | meters |
| Steepness,Som = | 0.0313 | |
| Peak period,Tp = | 8.01325 | sec |
| Storm duration,t = | 13.854 | hrs |
| Peak Steepness,Sop = | 0.0215 | |
| ***Relative Submergence, h/hc prim*** | | 1.002 |

**Notational Permeability,P=** **0.1**

| SLOPE COT alpha | **Hs,Significant Wave Height** Spectral No N(star)s | DIAMETER meters | WEIGHT KN | | **REMARKS** |
|---|---|---|---|---|---|
| 1.5 | 5.9636 | 0.789 | **12768.68** | 1.66 | **OK** |
| 2.0 | 5.9636 | 0.789 | **12768.68** | 1.66 | **OK** |
| 2.5 | 5.9636 | 0.789 | **12768.68** | 1.66 | **OK** |
| 3.0 | 5.9636 | 0.789 | **12768.68** | 1.66 | **OK** |
| 4.0 | 5.9636 | 0.789 | **12768.68** | 1.66 | **OK** |
| 5.0 | 5.9636 | 0.789 | **12768.68** | 1.66 | **OK** |

VAN DER MEER(1991) CEM(2000)
**Table VI-5-25**
**Eqn VI-5-72 Submerged Breakwters**
*Enter initial conditions*
Quarry stone,rough angular
n=2 layers, random placement
Structure Trunk
COT alpha = 1.5 to 5.0
Damage conditions specified by S

ARMOUR LAYER STABILITY
Rubble-Mound Structures and Submerged Slopes
(Metric System)

| **Enter Data** | | | Units |
|---|---|---|---|
| Wave Height,Hs | = | 2.16 | meters |
| Rho for rock | = | 2650 | Kn/m^3 |
| Gravity | = | 9.81 | m/sec^2 |
| Rho for water | = | 1000 | Kn/m^3 |
| Delta = Sr-1 | = | 1.65 | |
| Gamma Z | = | 1.00 | |
| Gamma H | = | 1.00 | |

| **Additional Data** | | Units |
|---|---|---|
| Nominal Damage,S = | 2.0 | |
| Water Depth, h = | 5.01 | meters |
| Crest Ele. hc prime = | 5.00 | meters |
| Wave Period,Tm = | 6.65 | sec |
| Number of Waves = | 7500.0 | |
| **Calculated Information** | | Units |
| Wave Length,Lom = | 69.05 | meters |
| Steepness,Som = | 0.0313 | |
| Peak period,Tp = | 8.01325 | sec |
| Storm duration,t = | 13.854 | hrs |
| Peak Steepness,Sop = | 0.0215 | |
| ***Relative Submergence, h/hc prim*** | | 1.002 |
| **Notational Permeability,P=** | | **0.5** |

| SLOPE COT alpha | **Hs,Significant Wave Height** Spectral No N(star)s | DIAMETER meters | WEIGHT KN | | REMARKS |
|---|---|---|---|---|---|
| 1.5 | 7.1089 | 0.662 | **7542.04** | 1.98 | **OK** |
| 2.0 | 7.1089 | 0.662 | **7542.04** | 1.98 | **OK** |
| 2.5 | 7.1089 | 0.662 | **7542.04** | 1.98 | **OK** |
| 3.0 | 7.1089 | 0.662 | **7542.04** | 1.98 | **OK** |
| 4.0 | 7.1089 | 0.662 | **7542.04** | 1.98 | **OK** |
| 5.0 | 7.1089 | 0.662 | **7542.04** | 1.98 | **OK** |

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**Table VI-5-25**
Rock, submerged breakwaters with two-layer armor on front, crest and rear slope (van der Meer 1991).

Irregular, head-on waves

$$\frac{h'_c}{h} = (2.1 + 0.1\ S)\ \exp(-0.14\ N_s^*) \qquad \text{(VI-5-72)}$$

where $h$ Water depth

$h'_c$ Height of structure over sea bed level ($h - h'_c$ is the water depth over the structure crest).

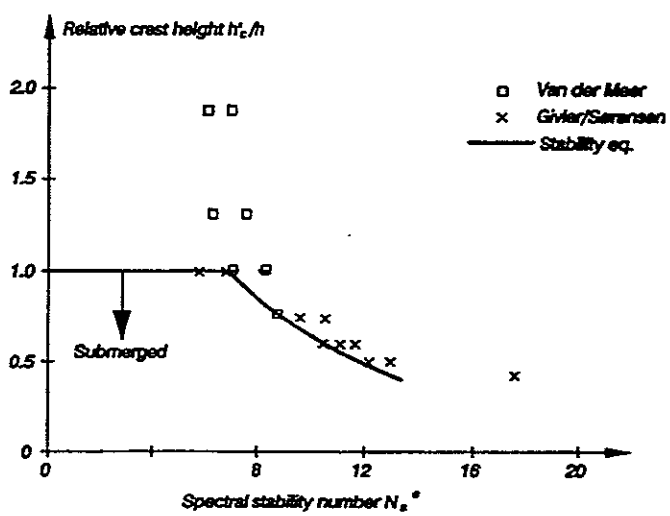
$S$ Relative eroded area

$N_s^*$ Spectral stability number, $N_s^* = \frac{H_s}{\Delta D_{n50}} s_p^{-1/3}$

Uncertainty of the formula: The uncertainty of Eqn. VI-5-72 can be expressed by considering the factor 2.1 as a Gaussian distributed stochastic variable with mean of 2.1 and standard deviation of 0.35, i.e., a coefficient of variation of 17%.

Data source: Givler and Sorensen (1986): regular head-on waves, slope 1:1.5

van der Meer (1991): irregular head-on waves, slope 1:2



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# EXAMPLE PROBLEM VI-7-16

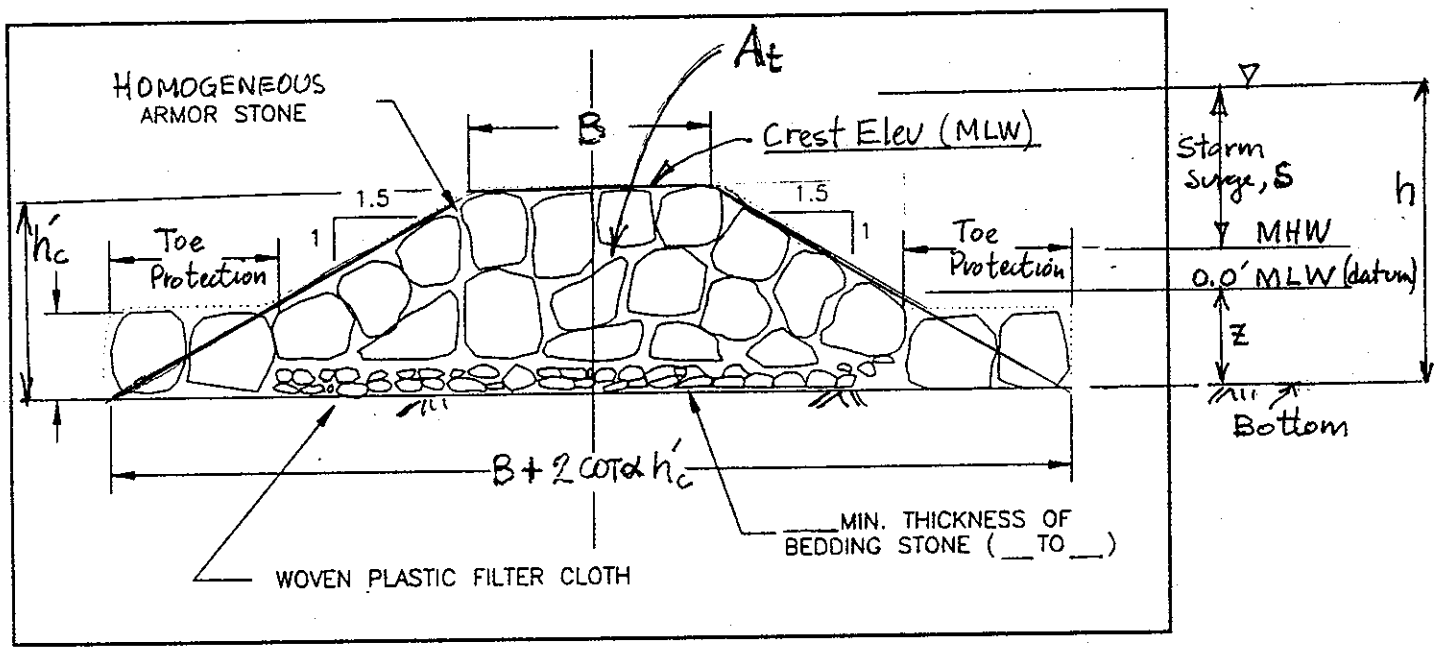
FIND: Select the armor stone size to withstand the design water level and wave conditions for a rubble-mound, nearshore breakwater constructed of homogeneous units with no core, i.e. permeable.

GIVEN: Breakwater crest elevations: +4.0 ft, +5.0 ft, +6.0 ft (MLW)

| Design Water Levels | | Design Wave Conditions | | Remarks |
|---|---|---|---|---|
| Recurrence Interval, Tr yrs | Storm Surge, S ft (MLW) | Wave Ht, $H_s$ (ft) | Wave Period, T (sec) | |
| 1 | 3.6 | 5.0 | 5.6 | Storm surge and waves at structure toe |
| 10 | 4.5 | 5.6 | 6.5 | |
| 25 | 5.2 | 6.5 | 7.6 | |
| 50 | 6.0 | 7.2 | 9.7 | |

(See sketch below)
Tidal Range, MTR = 1.0 ft
Elevation at BW location = −3.5 ft (MLW) = Z
Vertical Datum, MLW = 0.0 ft
Assume Crest Width, B = 5.0 ft

SKETCH:




## OVERTOPPING and SUBMERGENCE

Because of storm surge, some conditions will produce complete submergence so the breakwater acts as a low-crested reef. Table VI-5-27 gives Eqn VI-5-73 as proposed by van der Meer (1990) using data by Ahrens (1987) and van der Meer (1990).

$$h_c = \sqrt{\frac{A_t}{\exp(a N_s^*)}} \qquad \text{VI-5-73 (a)}$$

where $h_c = h_c'$, the no damage condition. The equilibrium profile ...

EXAMPLE PROBLEM VI-7-16 (continued)

change when excess energy is present to lower the crest elevation to height, $h_c$ (damaged) but the equilibrium area, $A_t$ remains unchanged.

And, where
$$N_s^* = \frac{H_s}{\Delta D_{n50}} s_p^{-1/3} \qquad (\text{VI}-5-73(b))$$

with
$$a = -0.028 + 0.045\frac{A_t}{(h_c')^2} + 0.034\frac{h_c'}{h} \qquad (\text{VI}-5-73(c))$$

Note that the stable stone size, $D_{n50}$, hence stable weight, $W_{50}$ is found by:

(i) specifying the reef breakwater dimensions, $h_c'$, B, and cot$\alpha$, hence first calculating $A_t$

(ii) calculating a from Eqn VI-5-73(c)* for a given water depth, h
*(here we have truncated the formula, omitting the small last term)

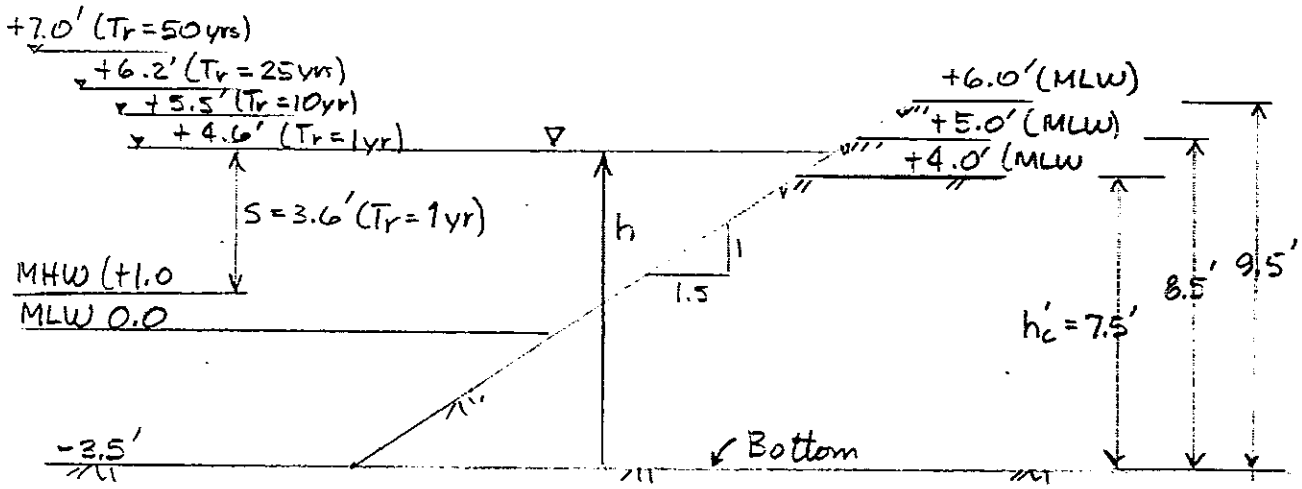
(iii) calculating $N_s^*$ from Eqn VI-5-73(a) when $h_c = h_c'$

(iv) calculating $s_p = H_s/L_{op}$, the deepwater, peak period, wave steepness

(v) calculating $D_n$ from Eqn VI-5-73(b), i.e.
$$D_n = \frac{H_s}{\Delta N_s^*} s_p^{-1/3} \qquad (\text{VI}-5-73(d))$$

(vi) and finally, calculating the homogeneous stone weight, W from
$$W = \rho_r g (D_n)^3$$

Again, a spread sheet solution aids the calculation process.
The sketch below illustrates the possible cases for design calculation.



EXAMPLE PROBLEM VI-7-16 (continued)

We now see that for the three design crest elevations and four design recurrence intervals, Equations VI-5-71 (Overtopping), Eqn VI-5-72 (modified and submerged) and Eqn VI-5-73 (reef type) are all applicable under some combinations of crest height and water level. All three should be studied to determine the condition producing the largest stones for stability.

CASE 1 Crest Elevation +4.0 ft (MLW) $h_c' = 7.5$ ft

| Recurrence Interval Tr, yrs | Water Depth, h feet | Wave Condition Hs ft | Wave Condition Tp, sec (Tm) | EQUATIONS VI-5-71 Overtopping, P=0.6 Dn, ft | VI-5-71 W, lbs | VI-5-72* Submerged, P=0.5 Dn, ft | VI-5-72* W, lbs | VI-5-73 Reef (Homogeneous) Dn, ft | VI-5-73 W, lbs | Rmks |
|---|---|---|---|---|---|---|---|---|---|---|
| 1.0 | 8.1 | 5.0 | 5.6 (4.65) | NOT APPLICABLE | | 1.26 | 330 | 1.26 | 330 | Sub |
| 10.0 | 9.0 | 5.6 | 6.5 (5.4) | | | 1.38 | 425 | 1.45 | 505 | Sub |
| 25.0 | 9.7 | 6.5 | 7.6 (6.3) | | | 1.57 | 645 | 1.76 | 902 | Sub |
| 50.0 | 10.5 | 7.2 | 9.7 (8.04) | | | 1.90 | 1070 | 2.20 | 1640 | Sub |

CASE 2 Crest Elevation +5.0 ft (MLW) $h_c' = 8.5$ ft

| | | | | VI-5-71 | | VI-5-72* | | VI-5-73 | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 1.0 | 8.1 | 5.0 | 5.6 | 1.65 | 740 | NOT APPLICABLE | | | | Overtop |
| 10.0 | 9.0 | 5.6 | 6.5 | Not Applicable | | 1.53 | 595 | 1.53 | 590 | Sub |
| 25.0 | 9.7 | 6.5 | 7.6 | | | 1.75 | 885 | 1.83 | 1010 | Sub |
| 50.0 | 10.5 | 7.2 | 9.7 | | | 2.06 | 1490 | 2.25 | 1875 | Sub |

CASE 3 Crest Elevation +6.0 ft (MLW) $h_c' = 9.5$ ft

| | | | | VI-5-71 | | VI-5-72* | | VI-5-73 | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 1.0 | 8.1 | 5.0 | 5.6 | 1.80 | 970 | NOT APPLICABLE | | | | Overtop |
| 10.0 | 9.0 | 5.6 | 6.5 | 2.00 | 1320 | | | | | Overtop |
| 25.0 | 9.7 | 6.5 | 7.6 | NOT APPLICABLE | | 1.94 | 1210 | 1.92 | 1170 | Sub |
| 50.0 | 10.5 | 7.2 | 9.7 | | | 2.27 | 1925 | 2.35 | 2150 | Sub |



22-141 50 SHEETS
22-142 100 SHEETS
22-144 200 SHEETS
AMPAD

EXAMPLE PROBLEM VI-7-16 (continued)

As expected the rarer events with low exceedance probabilities each year (higher recurrence intervals) with larger wave heights require the largest and heaviest stones for stability. The crest elevation selected depends upon economics, wave transmission and resulting erosional impacts between each nearshore breakwater structure.

This Example Problem is taken from Appendix A "Case Design Example of a Detached Breakwater Project" found in Chasten et al. (1993). It is a real, constructed project for the community of Bay Ridge, Anne Arundel County, Maryland on the western shore of the Chesapeake Bay near Annapolis, MD. Example Problem V-3-1 (see Part V) describes the functional design of the breakwater layout (spacing, distance offshore, length, etc.). A crest elevation of +4.0 ft (MLW) was selected for design. Structural stability design conditions are also given for the homogenous sized stone. Design conditions selected were:

$$T_r = 25 \text{ yrs}$$
$$H_s = 6.5 \text{ ft}$$
$$T_p = 7.6 \text{ sec}$$
$$DSWL = +5.2 \text{ ft (MLW)}$$
$$h = 9.7 \text{ ft (MLW)}$$

The Hudson formula (1977) and SPM (1984) coefficients employed in the analysis. The breaking wave Hudson coefficient, $K_D = 2.0$ gave

$$D_n = 2.71 \text{ ft}$$
$$W = 3{,}300 \text{ lbs}$$

The range of stone sizes accepted was 2500-4500 lbs. The cross-sectional sketch above is taken from Chasten et al. (1993) Appendix A. It shows the large stone placed at the bottom for toe protection. The crest width B was 9.0 ft using 3 stone widths

We now see that use of the Hudson formula for Case 1 with completely submerged conditions gives stone weights over three times heavier than required for these design conditions (902 lbs -vs- 3300 lbs). Even for the 50 year recurrence interval conditions the Hudson formula results are roughly twice heavier (1640 lbs - vs - 3300 lbs). The crest width, based on 3 stone widths could also be reduced (6 ft -vs- 9 ft) which translates into less stone volume required, and less costs.

Summary:

1. Equations VI-5-72 (modified for permeability) and Eqn-5-73 give similar results for submerged, breakwater conditions.
2. The Hudson formula was not meant to be applied for these conditions and is far too conservative.
3. The new CEM presents stone stability formulations that give practical information for a range of water depths and waves impacting nearshore breakwaters



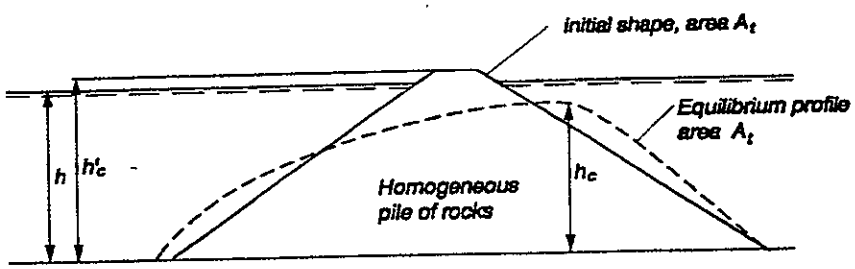
22-141 50 SHEETS
22-142 100 SHEETS
22-144 200 SHEETS
AMPAD

**Table VI-5-27**
Rock, low-crested reef breakwaters built using only one class of stone.

Irregular, head-on waves

Van der Meer (1990)

Trunk cross section of reef breakwater




The equilibrium height of the structure

$$h_c = \sqrt{\frac{A_t}{\exp(aN_s^*)}} \quad \text{with a maximum of } h'_c \qquad \text{(VI–5–73)}$$

where $A_t$ area of initial cross section of structure

$h$ water depth at toe of structure

$h'_c$ initial height of structure

$N_s^* = \frac{H_s}{\Delta D_{n50}} s_p^{-1/3}$

$a = -0.028 + 0.045\frac{A_t}{(h'_c)^2} + 0.034\frac{h'_c}{h} - 6 \times 10^{-9}\frac{A_t^2}{D_{n50}^4}$

Data source: Ahrens (1987), van der Meer (1990)

Powell and Allsop (1985) analyzed data by Ahrens, et al. (1982) and Ahrens (1984) and proposed the stability formula

$$\frac{N_{od}}{N_a} = a \exp\left[b H_s / (\Delta D_{n50})\right] \quad \text{or} \quad \frac{H_s}{\Delta D_{n50}} = \frac{1}{b}\ln\left(\frac{1}{a}\frac{N_{od}}{N_a}\right) \qquad \text{(VI–5–74)}$$

where values of the empirical coefficients $a$ and $b$ are given in the table as functions of freeboard $R_c$ and water depth $h$. $N_{od}$ and $N_a$ are the number of displaced rocks and the total number of rocks in the mound, respectively.

**Values of coefficients $a$ and $b$ in Eqn. (VI-5-74).**

| $R_c/h$ | $a \cdot 10^4$ | $b$ |
|---|---|---|
| 0.0 | 15 | 0.31 |
| 0.2 | 17 | 0.33 |
| 0.4 | 4.8 | 0.53 |

Valid for $0.0012 < H_s/L_p < 0.036$