CHAPTER 138

Dynamic Revetments

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Abstract

The concept of a stone revetment where rock movement is expected is relatively new. For people familiar with gravel and shingle beaches, the idea is an easily accepted extension of the traditional statically stable revetment. However, for engineers familiar with classic breakwater design, the idea that a dynamic structure can provide suitable shoreline protection raises many doubts. Recent research in the Netherlands by van der Meer (1988) and colleagues has gone a long way toward answering difficult questions regarding the development of sound design criteria for dynamic revetments. This paper discusses progress in quantifying the problem and defining solutions. Laboratory model tests were conducted at the Coastal Engineering Research Center (CERC) to confirm and extend Dutch research. These tests are described and important findings presented, including a simple quantitative method to predict the amount of stone required to protect a bulkhead, i.e. critical mass. The method to protect a bulkhead appears to have more general implications which could lead to extensive application to a wide range of shoreline erosion problems. Analysis of data is continuing.

Introduction and Background

The concept of a rubble structure having a dynamic response to wave attack has been around for quite some time. Per Bruun has commented frequently about the high stability of "S" shape profiles of some very old breakwaters in Plymouth, England, and Cherbourg, France (Bruun and Johannesson, 1976). An adaption of the "S" profile is the berm breakwater concept developed by William Baird (see Baird and Hall, (1984) for a discussion of design considerations, or Hall (1987) for background information. The basic strategy is to build an extensive stone berm which can adjust and deform in response to severe wave action. A berm is effective because a large mass of rubble near the water line is capable of

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disrupting wave action and dissipating wave energy. Stone size may be smaller than required for traditional armor, and placement does not require special care. The advantages appear to be, berm breakwaters have less expensive construction equipment requirements, are simple to construct, and use smaller stone, which is usually less expensive than conventional heavier armor stone. However, conventional breakwaters have a smaller cross section than berm breakwaters and smaller stone does not always cost less than conventional armor stone, so a berm breakwater is not always cost effective.

The idea of a dynamic revetment seems to be of more recent origin than dynamic breakwaters. van Hijum and Pilarczyk (1982) and Pilarczyk and den Boer (1983) present data and summarize some of the Dutch experience with gravel beaches and cobble-sized revetments. Recently, research has been initiated in England on the response of shingle beaches to wave action (Powell, 1988). Recent research in the Netherlands and England is motivated by a need for fundamental understanding of shingle beaches, how they might be nourished, and if shingle beaches could be used in some situations instead of a traditional statically stable riprap revetment.

With increasing interest in both dynamic breakwaters and revetments, there has been an acknowledgement that not enough was known about the durability of stone. Considerable effort has been directed in the United Kingdom towards understanding rock durability in the marine environment under dynamic conditions, (see e.g. Allsop and Latham, 1987 and Latham and Poole, 1988).

In the United States, Johnson (1987) found that gravel beaches and dumped rubble are frequently cost effective alternatives to using sand for beach nourishment and placed stone for revetments, respectively. Johnson's findings were obtained from extensive experience on Lakes Michigan and Superior where fluctuating water levels created enormous problems for conventional shoreline protection. This experience shows dynamic revetments are not vulnerable to toe scour, overtopping, or flanking. Advantages cited by Johnson for coarse material include a long residence time and an ability to stay in the vicinity of the water line. Other advantages are similar to those noted by Baird and Hall (1984), i.e., ease of placement and lower unit cost.

Comprehensive research efforts conducted recently in the Netherlands resulted in detailed and quantitative findings on dynamic stability (van der Meer, 1988). Findings are based on a very extensive amount of laboratory work and data analysis. One problem in applying van der Meer's results is that his tests were conducted in relatively deep water, and most problems in the United States involving shoreline erosion and protection are in shallow water. One goal of this study is to utilize the Dutch research to the greatest possible extent to help design dynamic revetments.

This paper provides information on laboratory tests conducted to gain familiarity with the concept of dynamic revetments and their use in shallow water. Specifically, this study was initiated to determine how dumped stone might protect a vertical bulkhead.

Test Setup, Conditions and Procedures

The model tests were conducted in CERC's 0.46x0.91x45.73 meter long glass walled wave tank (Figure 1), using an undistorted Froude scale of 1:16 (model : prototype). Irregular waves representing JONSWAP spectra were generated by a hydraulically actuated piston type wave maker. The test sections were placed approximately 35.4 meters from the wave board. Wave data were collected for each run using two Goda Arrays each consisting of three electronically driven resistance-type wave gages. Incident and reflected spectra were resolved using the method of Goda and Suzuki (1976). Wave signal generation and data acquisition were controlled using a DEC MicroVAX I computer. Data analysis was preformed on a DEC VAX 11/750.

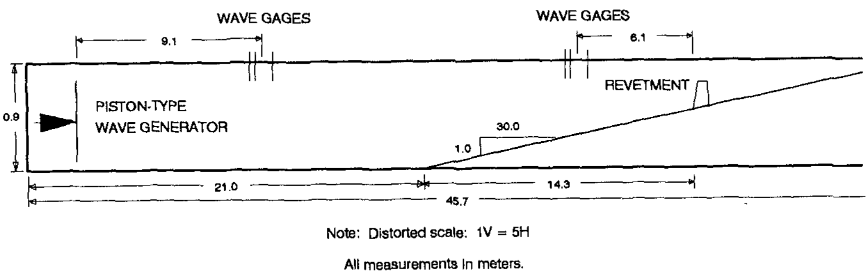

Figure 1. Profile view of wave tank setup.

Figure 2 shows a typical initial and equilibrium profile for a dynamic revetment. All initial profiles, except for Test 4, had a horizontal berm and a seaward face on a slope of 1 on 1, (vertical to horizontal). Test 4 used the equilibrium profile from Test 3 as a starting profile to determine how sensitive the equilibrium profile was to initial conditions. The influence of the initial berm width and berm height above the still water level (SWL) are two of the major variables investigated in this study. One goal of the study was to determine how much stone was required to protect a vertical bulkhead from direct wave attack. The bulkhead was simulated in the model using a plywood board to terminate the rubble on the landward side, located at 0.0 on the horizontal axis in the profile figures. Profiles shown in the figures are the average of five profile surveys along the length of the tank. There was very little across-tank variation in the profile observed during these tests. Surveys were made using a rod attached to a disc with a ball and socket connection. The disc had a diameter of 15 mm.

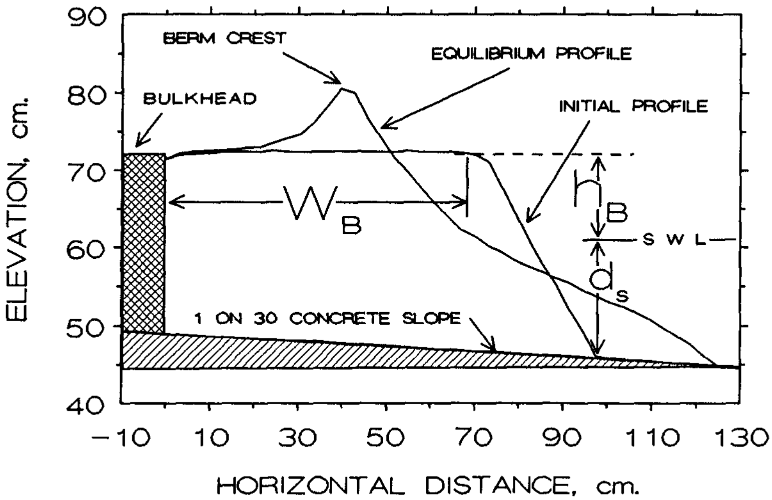

Figure 2a. Typical equilibrium profile.

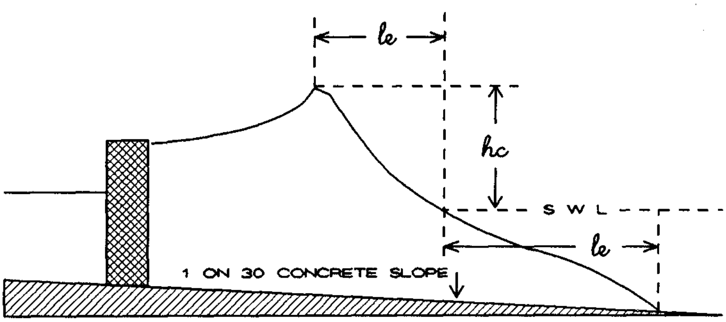

Figure 2b. Definition Sketch

Initial test conditions were first generated to simulate wave action similar to that found on Lake Michigan near a site in Chicago at Devon Avenue. The initial berm width was chosen by substituting these initial test conditions into a simplified version of the Dutch equations (van der Meer, 1988). As testing progressed, more severe wave conditions were run in the wave tank to fully test the range of circumstances for which a dynamic revetment would be suitable. Later tests examined a shorter wave period which may better represent the conditions for a structure on a body of water smaller than Lake Michigan.

Table 1 summarizes the gradations and specific gravity of the stone used in this study. Specifically the stone is a dense limestone.

Cumulative Percent Passing	Tests 1-22 Sieve size (mm)	Tests 23-26 Sieve size (mm)
2	4.8	3.1
15	5.6	4.3
50	8.1	5.6
85	11.2	7.3
98	12.7	9.3
specific gravity	rho(r) = 2.68	rho(r) = 2.72

Table 1. Characteristics of stone used in this study

Table 2 gives some of the basic data collected during this study. Number of waves is the total number of waves generated during a test based on the period of peak energy density, T_p , of the incident spectrum. T_p and H_{mo} , the incident zero-moment wave height, were measured at Goda Array 2 which is the shallow water array shown in Figure 1. Array 2 depth is the water depth at the middle gage in the three gage array. It was found that the initial profile adjusts rapidly to the incident wave conditions. For tests with $T_p = 2.5$ there was little change in the profile between 3000 and 5000 waves and for tests where $T_p = 1.75$, there was little change between 4245 and 7080 waves.

Berm width, W_B , is the horizontal length of the berm as it was constructed at the beginning of a test. Berm height, h_B , is the average vertical distance from the still water level (SWL) to the almost horizontal berm surface at the beginning of a test. Berm crest height, h_c , and berm crest length, l_b , are the vertical and horizontal distances respectively from the still water line to the conspicuous berm crest formed by the wave runup. Toe water depth, d_s , is the depth at the toe of the revetment at the beginning of a test. Erosion depth, h_e , and erosion length, l_e , are the depth and horizontal distance respectively of the toe of

Test no.	No. Waves N	Array 2 Hmo (cm)	Tp (sec)	Nominal Depth (cm)	Berm Width (cm)	Berm Height Above S.W.L. (cm)	Water Depth At Toe (cm)	Water Depth At Wave Generator (cm)	Revet-ment Re-sponse Category	Berm Crest Height (cm)	Reflec-tion Coeff. Kr
1	5000	6.83	2.53	37.19	54.86	10.77	14.81	60.96	I	19.04	0.45
1	3000	6.73	2.51	37.19	54.86	10.77	14.81	60.96	I	18.44	0.45
2	3000	13.31	2.52	37.19	54.86	10.77	14.81	60.96	F	NA	0.36
3	5000	6.92	2.57	34.14	54.86	13.82	11.71	57.91	S	17.07	0.47
3	3000	6.81	2.60	34.14	54.86	13.82	11.77	57.91	S	17.56	0.48
4	3000	13.25	2.52	34.14	54.86	13.82	11.71	57.91	I	21.12	0.36
5	3000	13.03	2.52	34.14	54.86	13.82	11.71	57.91	I	21.61	0.35
6	3000	13.62	2.65	37.19	70.10	11.28	15.30	60.96	I	21.43	0.39
7	5000	7.02	2.50	37.19	70.10	11.28	15.30	60.96	S	19.51	0.46
7	3000	6.86	2.51	37.19	70.10	11.28	15.30	60.96	S	18.50	0.46
8	5000	13.61	2.50	37.19	77.72	11.15	15.58	60.96	I	24.20	0.38
8	3000	14.07	2.64	37.19	77.72	11.15	15.58	60.96	I	23.26	0.39
9	3000	7.07	2.54	37.19	24.38	10.93	13.66	60.96	F	NA	0.49
10	3000	6.90	2.47	34.14	24.38	13.98	10.61	57.91	I	16.73	0.44
11	4286	11.62	1.81	34.14	54.86	14.25	11.51	57.91	S	17.43	0.30
12	4286	11.49	1.85	37.19	54.86	11.20	14.56	60.96	I	17.89	0.28
13	7143	11.52	1.74	34.14	24.38	13.98	10.61	57.91	I	16.73	0.27
13	4286	11.43	1.76	34.14	24.38	13.98	10.61	57.91	I	16.52	0.28
14	3000	13.46	2.54	43.28	77.72	11.27	15.23	67.06	I	24.05	0.38
15	3000	11.44	1.75	43.28	77.72	11.27	15.23	67.06	S	17.74	0.28
16	3000	13.51	2.56	37.80	92.96	11.24	15.91	67.06	I	24.51	0.38
17	4576	11.45	1.75	34.14	24.38	11.04	13.62	60.96	F	NA	0.32
17	3000	12.07	1.75	34.14	24.38	11.04	13.62	60.96	F	NA	0.30
18	3000	13.44	2.49	34.14	24.38	14.09	10.92	57.91	F	NA	0.37
19	3000	5.56	1.77	34.14	24.38	14.09	10.47	57.91	S	NA	0.35
19	4576	5.61	1.77	34.14	24.38	14.09	10.47	57.91	S	NA	0.34
20	3000	5.71	1.73	37.19	24.38	11.04	13.52	60.96	I	13.69	0.33
20	4576	5.64	1.74	37.19	24.38	11.04	13.52	60.96	I	13.62	0.33
21	3000	13.38	2.49	37.19	24.38	11.04	13.52	60.96	F	NA	0.42
22	3000	5.75	1.79	37.19	54.86	10.99	14.60	60.96	S	14.20	0.33
22	4576	5.65	1.73	37.19	54.86	10.99	14.60	60.96	S	14.48	0.32
23	2160	12.38	2.51	37.19	54.86	10.63	14.60	60.96	F	NA	0.43
24	3000	6.42	2.52	37.19	54.86	10.63	14.60	60.96	I	20.33	0.50
25	3000	10.97	1.71	37.19	54.86	10.63	14.60	60.96	I	18.99	0.30
25	4576	10.78	1.85	37.19	54.86	10.63	14.60	60.96	I	19.11	0.30
26	3000	5.45	1.72	37.19	54.86	10.63	14.60	60.96	S	14.72	0.37
26	4576	5.37	1.79	37.19	54.86	10.63	14.60	60.96	S	15.79	0.37

Table 2. Basic Data

the revetment from the still water line for the indicated number of waves. Water depth at the wave generator is the depth in the deepest part of the tank (Figure 1). Revetment Response Category (RRC) is a simple evaluation of the performance of the revetment during a test where "F" indicates the revetment failed, "S" indicates the revetment was safe, and "I" indicates an intermediate condition. The RRC and the dimensionless revetment size will be discussed further in the next section. K_r is the reflection coefficient which is defined by Goda and Suzuki (1976) as the square root of the ratio of reflected to incident wave energy.

Critical Mass Analysis

One of the goals of this study was to provide guidance on the quantity of stone, critical mass, necessary to protect a structure, such as a bulkhead, from wave attack. To accomplish this goal, all of the test results were classified into one of three categories. When wave conditions are severe in relation to the quantity of stone in the revetment wave action will erode the rubble, usually by carrying it over the bulkhead, until waves can impact directly against the bulkhead; this category was designated failure, denoted "F" in Table 2 and illustrated by Figure 3a. When the amount of stone in a revetment is large in relation to the wave conditions the development of the berm crest will have enough room so that neither stone or water will be carried over the bulkhead; this category is designated safe, denoted "S" in Table 2, and illustrated by Figure 3b. The third category fell between safe and failure and occurs when the berm crest buildup extends far enough landward to reach the bulkhead and there is at least some overtopping of the bulkhead by both water and stone; this category was designated intermediate, denoted "I" in Table 2, and illustrated by Figure 3c. The ability of a dynamic revetment to protect a bulkhead is a function of the volume of stone per unit length and the incident wave conditions.

In order to calculate the critical mass it is necessary to estimate three characteristic dimensions of a dynamic revetment, i.e., berm crest height, h_c , berm crest length, l_c , and erosion length, l_e . Regression analysis was employed to determine the following equations which are used to estimate, h_c , l_c , and l_e :

$$h_c/H_{mo} = 0.270*(H_{mo}/L_p)^{-0.645}, \ R^2 = 0.96 \tag{1}$$

$$l_c/H_{mo} = 0.677*(H_{mo}/L_p{}^{-0.521}, \ R^2 = 0.92 \tag{2}$$

$$l_e/d_s = \exp(2.24*(H_{mo}/L_p)^{0.143}, \ R^2 = 0.64 \tag{3}$$

Equations 1, 2, and 3 are based on analysis of Tests 1 through 22. R^2 values give the portion of the variance explained by the regression analysis. Tests 23, 24, 25, and 26 were conducted with somewhat smaller stone (see Table 1) and were withheld from analysis. Figures 4 shows observed data with regression trends for Equations 1, 2, and 3, respectively. Stone sizes are denoted by

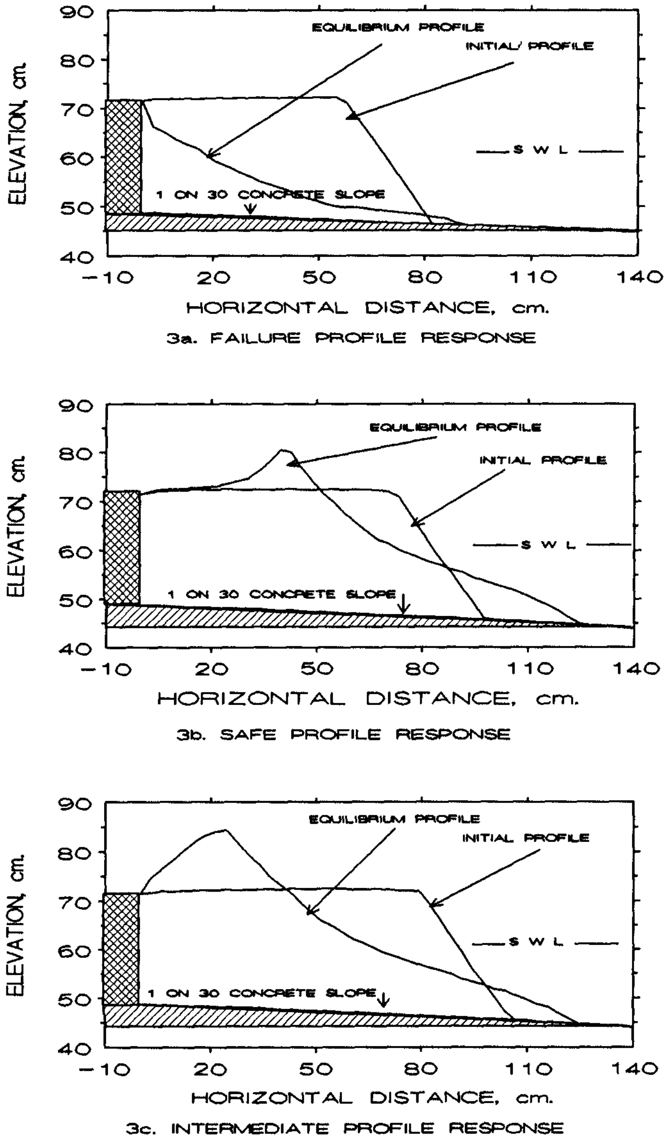

Figure 3. Dynamic Revetment Response Categories.

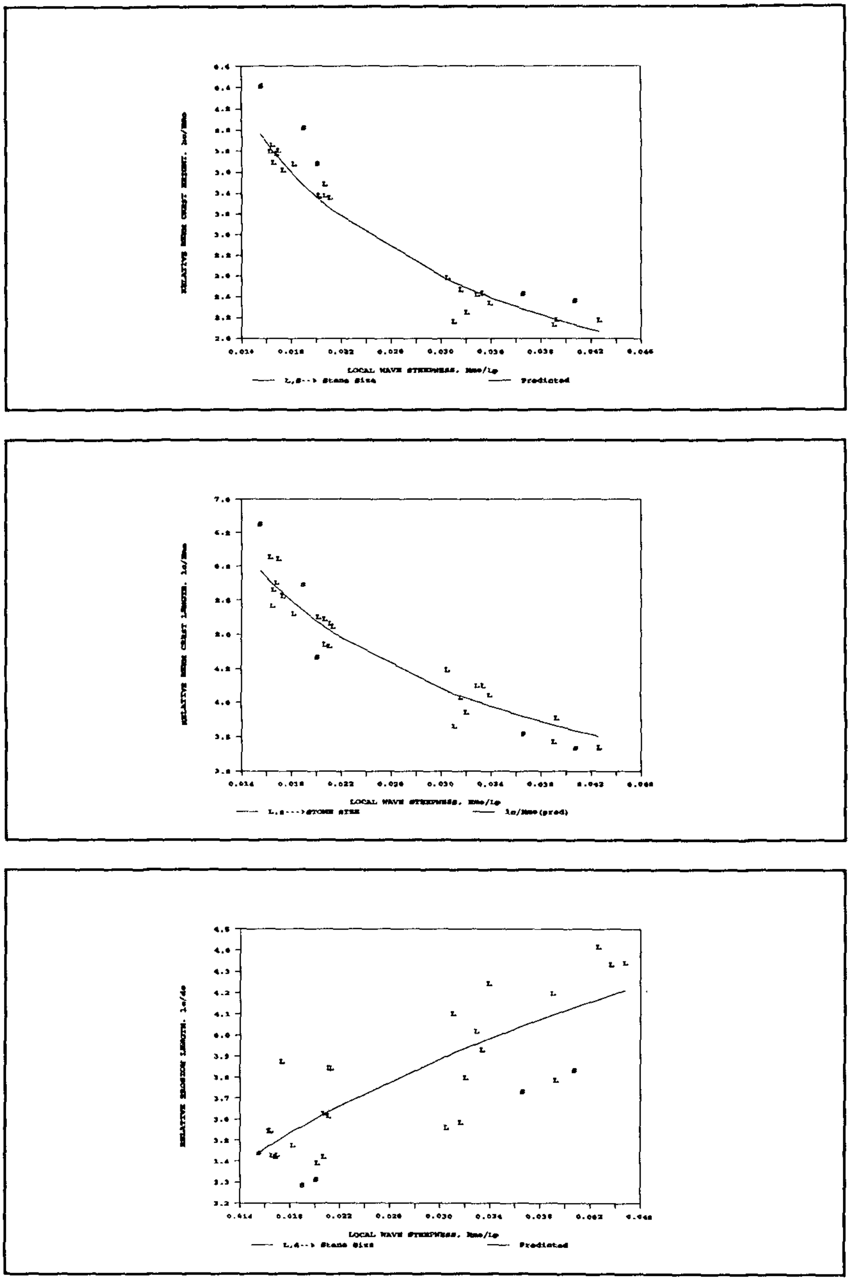

Figure 4, Characteristic dynamic revetment dimensions as a function of local wave steepness.

symbol in these figures so that the suitability of regression curves for the smaller sized stone withheld from analysis can be judged.

To specify the scale of the revetment response to wave attack it is useful to define a scale parameter, $A_s(c_m{}^3/c_m)$, where

$$A_s = (d_s + h_c)*(l_e + l_c) \tag{4}$$

and where h_c , l_c , and l_e are estimated using Equation 1, 2, and 3, respectively. The water depth at the toe of the revetment d_s is selected based on design considerations. The total volume of the revetment per unit length is denoted $A_t(c_m{}^3/c_m)$ and includes the void space of about 45 percent. Figure 5 shows the response category versus the ratio of A_t to A_s . For convenience let $A_t'(s) = A_t/A_s$. It can be seen that $A_t'(s)$ is able to properly categorize all tests including those with smaller stone withheld from analysis.

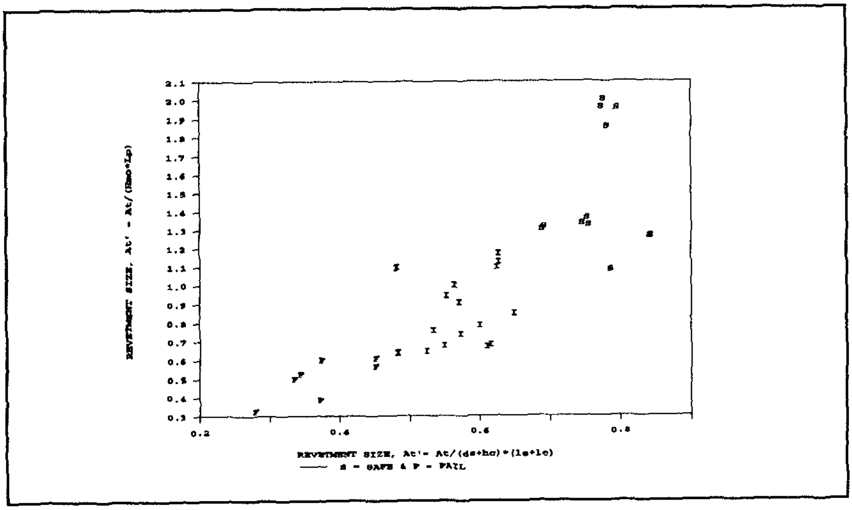

Figure 5, Dynamic revetment response categories as a function of two dimensionless revetment size variables.

A slightly easier but less accurate way to estimate the amount of stone required for a stable dynamic revetment can be obtained using the wave severity parameter, $H_{mo}*L_p$, where L_p is the Airy wave length calculated using d_s and T_p . Let $A_t'(w_s) = A_t/(H_{mo}*L_p)$. In Figure 5 it can be seen that $A_t'(s)$ is somewhat better than $A_t'(w_s)$ at predicting the proper revetment response category. The purpose of considering $A_t'(ws)$ is that it is quite simple and can provide some additional insight. It was found from the study of reef breakwaters that when wave conditions were quite severe in relation to the size of a reef, the reef could not dissipate wave energy effectively. For reefs it was found that for $(A_t/(H_{mo}*L_p)) <= 0.5$, energy dissipation was largely a function of the relative size of the structure such that the percent energy dissipation = $(A_t/(H_{mo}*L_p))*100$.

From the analysis given above there appears to be at least two ways to interpret the critical mass for a dynamic revetment: (1) It is a supply and demand relation to determine if A_t is large enough to supply the demand, characterized by A_s . (2) It is an energy dissipation relation to determine if there is enough stone A_t available to dissipate the wave energy characterized by $H_{mo}*L_p$.

Wave Reflection and Energy Dissipation

After Coda Suzuki (1976) the reflection coefficient is defined as the square root of the ratio of the reflected wave energy to the total incident wave energy. Wave reflection from dynamic revetments appears to be a function of two variables, wave steepness and relative void size. Reflection coefficients can be predicted with the following equation:

$$K_r = 1/(1.0 + (C_o*(d_{50}/L_o)^{C_1}*\exp(C_2/H_{mo}/L_o))) \qquad (5)$$

where d_{50} is the median stone size and L_o is the deep water wave length given by, $L_o = (g*(T_p^2))/2*\pi$. The dimensionless regression coefficients are given by,

$C_o = 23.4$
$C_1 = 0.312$
$C_2 = -0.00374$

Equation 5 explains about 97 percent of the variance in a sample size of 30,i.e. $R^2 = 0.97$ and $N = 30$. Tests in the failure response category were not included in the above analysis since at failure a substantial part of the reflection is from the vertical bulkhead. Percent of incident wave energy dissipated by a dynamic revetment can be estimated by using Equation 5 and the relation,

$$\%D = (1.0 - (K_r^2)) * 100$$

where %D is the percent energy dissipation. Observed data gives reflection coefficients between 0.27 and 0.50, indicating that dynamic revetments dissipate between 75 and 92 percent of the incident wave energy. By dissipating over three-quarters of the incident wave energy dynamic revetments would make good wave absorbers.

Summary and Conclusions

A series of laboratory tests were conducted to investigate the response of dynamic revetments to shallow waver wave conditions. No more laboratory tests are planned at this time but data analysis is continuing. Most tests from this study fall into the category "dynamically stable rock slopes" based on the Dutch classification system (van der Meer and Pilarczyk, 1987). For this study, the ratio of the wave height to stone dimension is in the range of roughly 5 to 16. Typically zero-damage on a conventional riprap revetment occurs when the wave is about two and a half times larger than the stone dimension.

It was found the equilibrium dynamic revetment profile was not sensitive to the initial profile. This means that construction costs can be lowered because special care is not required in the placement of the stone. The berm crest is a conspicuous feature of the profile and the crest height is strongly dependent on the product of the zeroth moment wave height and local wave length.

The concept of a critical mass for a dynamic revetment is introduced. Critical mass is the quantity of stone required to protect a unit length of a vertical bulkhead for a given water depth at the toe and given wave conditions. This quantity is found to increase with increasing water depth, and zeroth moment wave height and period of peak energy density. Two methods to calculate the critical mass are introduced and discussed.

Future work includes more analysis and greater familiarity with the extensive Dutch research (van der Meer, 1988). Surprisingly, most of the Dutch laboratory work was conducted in relatively deep water at the toe of the revetment. Part of the effort in this study involves determining to what extent the Dutch findings can be applied to shoreline erosion in the United States. A related study being conducted in England can also provide valuable information to this study (Powell, 1988).

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