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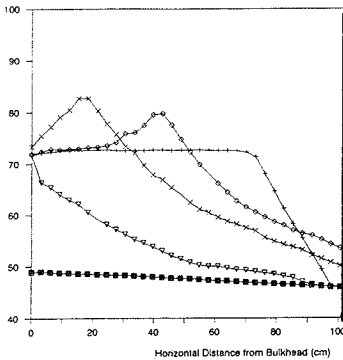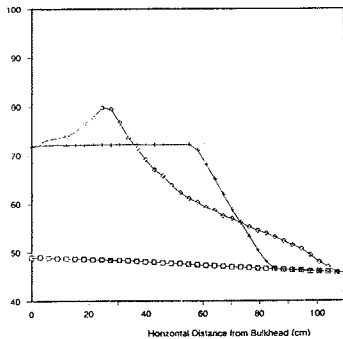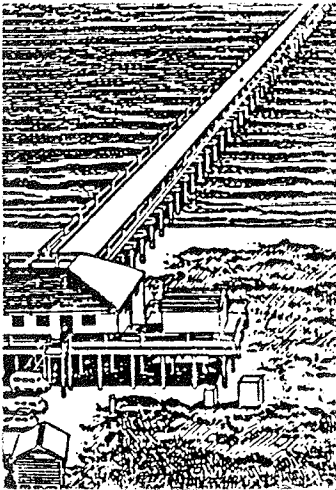
LABORATORY STUDY OF A DYNAMIC BERM REVETMENT

by

Donald L. Ward, John P. Ahrens

Coastal Engineering Research Center

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
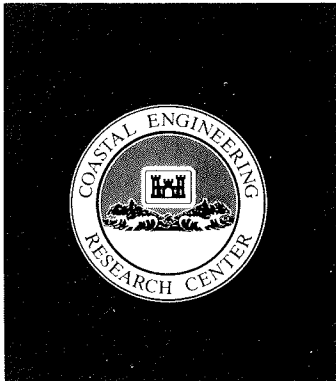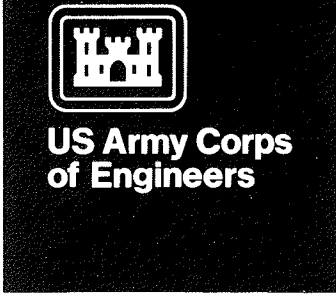3909 Halls Ferry Road, Vicksburg, Mississippi 39180-6199

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A series of physical model tests was conducted in a wave flume to study the effect of dynamic rubble protection in front of a vertical bulkhead. This rubble berm revetment was placed in front of a bulkhead with stone sized such that wave action would reshape the berm into an equilibrium profile. Profiles of the berm were taken before and after testing, and berms classified as "safe," "intermediate," or "failed" depending on protection provided to the bulkhead. Regression analysis was used to determine the quantity of stone necessary to provide protection for a wide range of storm conditions. An equation also was developed to quantify the reflected wave energy and thus determine the energy dissipation by the berm.

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PREFACE

The investigation described in this report was authorized as a part of the Civil Works Research and Development Program by Headquarters, US Army Corps of Engineers (HQUSACE). Work was performed under Work Unit 32432, "Design of Revetments and Seawalls," at the Coastal Engineering Research Center (CERC), US Army Engineer Waterways Experiment Station (WES). The HQUSACE Technical Monitors were Messrs. John H. Lockhart, Jr.; John G. Housley; James E. Crews; and Robert H. Campbell. Dr. C. Linwood Vincent was CERC Program Monitor.

The study was conducted by personnel of CERC under the general direction of Dr. James R. Houston, Chief, CERC, and Mr. Charles C. Calhoun, Jr., Assistant Chief, CERC. Direct supervision was provided by Messrs. C. E. Chatham, Chief, Wave Dynamics Division (WDD), and D. Donald Davidson, Chief, Wave Research Branch (WRB), WDD, CERC. This report was prepared by Messrs. Donald L. Ward, Principal Investigator, WRB, and John P. Ahrens, Research Oceanographer, WRB. The model was operated by Mr. Willie G. Dubose, Engineering Technician, WRB. Assistance with data analysis and graphics was provided by Mr. John M. Heggins, Computer Technician, WRB. This report was typed by Ms. Myra E. Willis, WRB, and edited by Ms. Lee T. Byrne, Information Technology Laboratory, WES.

COL Larry B. Fulton, EN, was Commander and Director of WES during report publication. Dr. Robert W. Whalin was Technical Director.

CONTENTS

LABORATORY STUDY OF A DYNAMIC BERM REVETMENT

PART I: INTRODUCTION

Background

1. Conventional revetments are designed to be statically stable; that
is, no motion of the armor stone is anticipated. Stones in the armor layer
are sized and placed such that their weight and interlocking will preclude
movement during wave attack. In contrast, a dynamic revetment is designed to
allow wave action to rearrange the stones into an equilibrium profile.
Because stones are allowed to move, a smaller stone size is used for the
dynamic revetment than for the static revetment, but dynamic revetments
require a larger quantity of stone to allow for the reshaping of the revetment
into an equilibrium profile. The dynamic revetment is effective because the
large mass of stone near the still-water level (SWL) disrupts the wave action
and dissipates wave energy. Although dynamic revetments require a larger
quantity of stone, these costs may be offset by the typically lower cost of
smaller stone, and, because size is less critical, a more cost-effective use
may be made of quarry output. In addition, smaller stone is less expensive to
handle, and, since initial placement is not critical, dynamic revetments may
be dumped in place rather than the stones being individually placed.

2. The concept of a rubble breakwater having a dynamic response to
wave attack is not new. 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), and the berm
breakwater concept developed by William Baird (Baird and Hall 1984, Hall
1987) is an adaptation of this "S" profile. The idea of a dynamic revetment,
however, seems to be of more recent origin. 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, and
research has been initiated in England on the response of shingle beaches
to wave action (Channell, Stevenson, and Brown 1985; 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.

3.   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 indicated dynamic revetments were not vulnerable to toe scour, overtopping, or flanking. Advantages cited by Johnson for coarse material on beaches 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.

Problem

4.   Comprehensive research efforts conducted recently in The Netherlands resulted in detailed and quantitative findings on dynamic stability (van der Meer 1988). Although the findings were based on extensive laboratory work and data analysis, the data are of limited applicability in the United States because van der Meer's tests were conducted in relatively deep water, whereas most US Army Corps of Engineers (USACE) problems involving shoreline erosion and protection are in shallow water. There is no design guidance on the use of dynamic revetments for coastal protection that is applicable to the shallow waters typically encountered in USACE projects.

Purpose

5.   The purpose of this study was to determine how dumped stone might protect a vertical bulkhead in shallow water, and particularly to determine a means of calculating the minimum quantity of stone necessary to provide adequate protection (the "critical mass"). However, the information on reshaping, equilibrium profile, and dynamic stability is of general applicability and should prove of value to a wide range of coastal projects.

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PART II: DEFINITION OF TEST PARAMETERS

6. Inconsistencies among authors in notations, definitions of parameters, and the methods by which a value for a parameter is obtained greatly complicate the task of comparing results from different studies. In this report, notations will follow guidelines published by the International Association for Hydraulic Research in its "List of Sea State Parameters" (1986) where applicable. Additional parameters, definitions, and method used to determine the value of certain parameters are given in the following section.

Wave and Spectral Parameters

7. Wave heights used in this report are the heights of the zeroth moment (H_{mo})* and are obtained as four times the square root of the zeroth moment of the potential energy spectrum. The H_{mo}'s of the incident spectra are separated from the H_{mo}'s of the reflected spectra by the method of Goda and Suzuki (1976), using a three-gage array. Two arrays are used, one to measure the H_{mo}'s near the wave generator (Array 1) and one near the structure toe (Array 2).

8. Peak period (T_p) is the wave period associated with the highest energy density of the spectrum. This was obtained by dividing the spectrum into 256 bands and finding the period causing the highest energy density over 11 adjacent bandwidths.

9. Peak period was used to estimate the desired length of a test run by multiplying the desired number of waves by the peak period.

10. Reflection coefficient is commonly defined as the ratio of reflected wave height to incident wave height. This is clearly inappropriate when incident and reflected wave heights are described by different spectra. Reflection coefficients were therefore determined by the energy of the respective spectra, following the method of Goda and Suzuki (1976).

$$K_r = \left(\frac{E_R}{E_I}\right)^{1/2} \tag{1}$$

---

* For convenience, symbols and abbreviations are listed in the Notation (Appendix C).

where K_r is the reflection coefficient and E_R and E_I are the energy of the reflected and incident spectra, respectively.

11. Reflected wave height is obtained as the product of reflection coefficient and incident wave height.

$$H_r = K_r H_{mo} \qquad (2)$$

where H_r is reflected wave height.

12. Wave heights and periods are frequently reported in other investigations in terms of significant wave height (H_s) and average wave period (T_z), where H_s is the average of the one-third highest waves. Both H_s and T_z are included in the data in this report to simplify comparison with other investigations. Because the measured H_s includes both incident and reflected wave energy, the incident H_s is estimated from the reflection coefficient as

$$\frac{H_{s(t)}}{\sqrt{1 + K_r^2}} \qquad (3)$$

where $H_{s(i)}$ is the incident significant wave height and $H_{s(t)}$ is the combined incident and reflected significant wave height. $H_{s(t)}$ was determined as the average from the three gages in the array. Average wave periods in this report were determined as

$$T_z = \left(\frac{m_o}{m_2} \right)^{1/2} \qquad (4)$$

where m_o and m_2 are the zeroth and second moments of the incident potential energy spectrum, respectively.

13. The spectral width or peakedness determined from the wave record is given as Q_p, defined by Goda (1970) as

$$Q_p = \frac{2}{(m_o)^2} \int_o^\infty f[S(f)]^2 \, df \qquad (5)$$

where f is frequency and $S(f)$ is the wave spectral density function for the given frequency.

Material Parameters

14.   Small sizes of stone, such as those used in the current study, are
frequently measured by sieve analysis. Larger stones are described by their
nominal diameter d_n defined as

$$d_n = \left(\frac{W}{w_r}\right)^{1/3} \tag{6}$$

where W is the weight of the stone and w_r is the specific weight of the
material. The nominal diameter of the median stone weight is $d_{n(50)}$.

Berm Parameters

15.   Figure 1 illustrates the major berm parameters prior to a test run.
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 SWL to the horizontal berm surface at the beginning of a test.

16.   A typical after-test profile is shown in Figure 2. Berm crest
height h_c and berm crest length l_c are the vertical and horizontal dis-
tances respectively from the intersection of the SWL and the equilibrium pro-
file to the conspicuous berm crest formed by the wave runup. Erosion depth
h_e and erosion length l_e are the depth and horizontal distance, respec-
tively, of the revetment toe from the intersection of the SWL and the
equilibrium profile.

17.   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 inter-
mediate condition. For these tests, a failure was defined as exposure of the
bulkhead, whereas a safe condition indicated that neither sand nor water
overtopped the bulkhead. These RRC's are described in more detail in
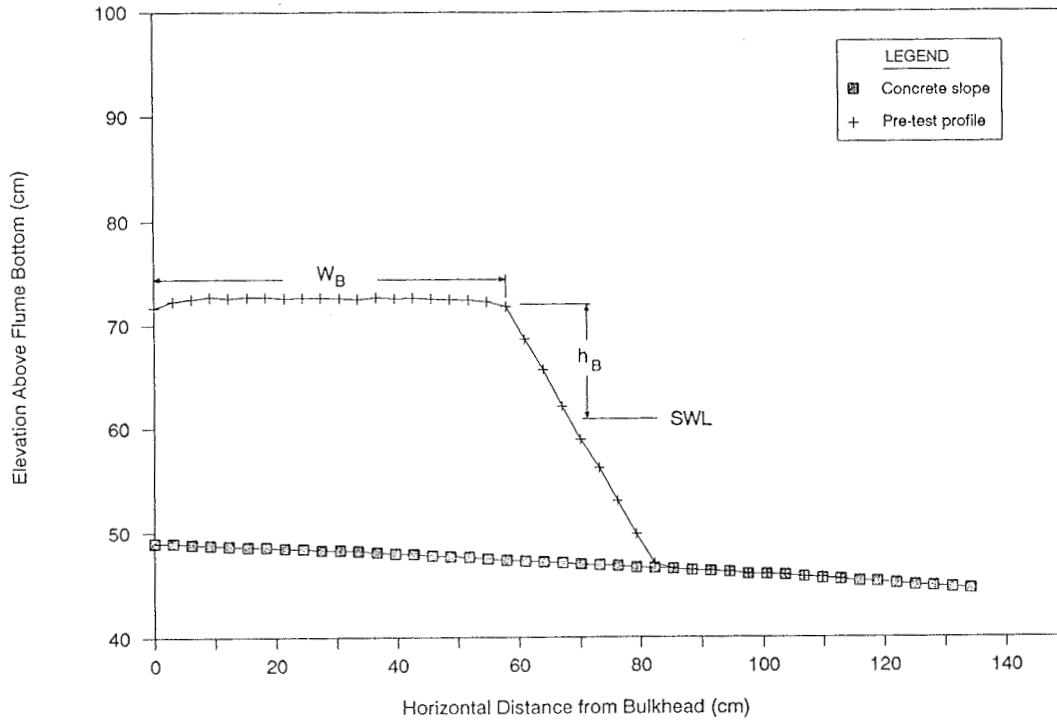paragraph 30.

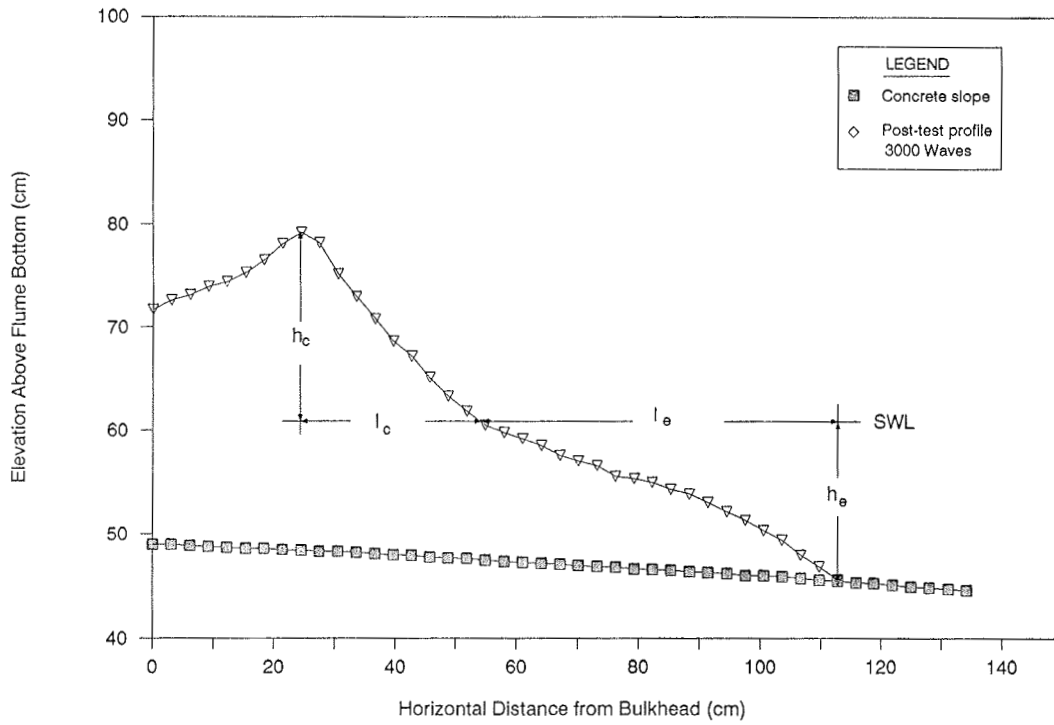Figure 1. Berm parameters from the pretest profile

Figure 2. Berm parameters from the posttest profile

PART III: FLUME SETUP, TEST CONDITIONS, AND RESULTS

Flume Setup

18. Model tests were conducted in the USACE Coastal Engineering
Research Center's (CERC's) 0.46-m-wide by 0.91-m-high by 45.73-m-long glass-
walled wave flume (Figure 3) using an undistorted Froude scale (Stevens et al.
1942) of 1:16 (model:prototype). Irregular waves representing Joint North Sea
Wave Project (JONSWAP) spectra (Hasselmann et al. 1973) were generated by a
hydraulically actuated piston-type wave maker. Wave data were collected for
each run using two arrays, each consisting of three electronically driven
resistance-type wave gages. Wave signal generation and data acquisition were
controlled using a DEC MicroVAX I computer, and data analysis was performed on
a DEC VAX 750 and 3600.
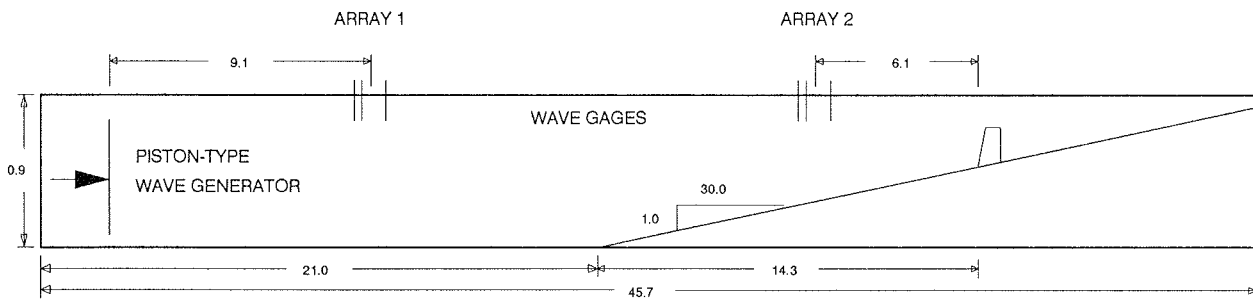
Note: Distorted scale: 1V = 5H

All measurements in meters.

Figure 3. General layout of wave flume

19. The test sections were placed approximately 35.4 m from the wave
board. Figure 4 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 1:1 (vertical: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 (see paragraph
25). A 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.

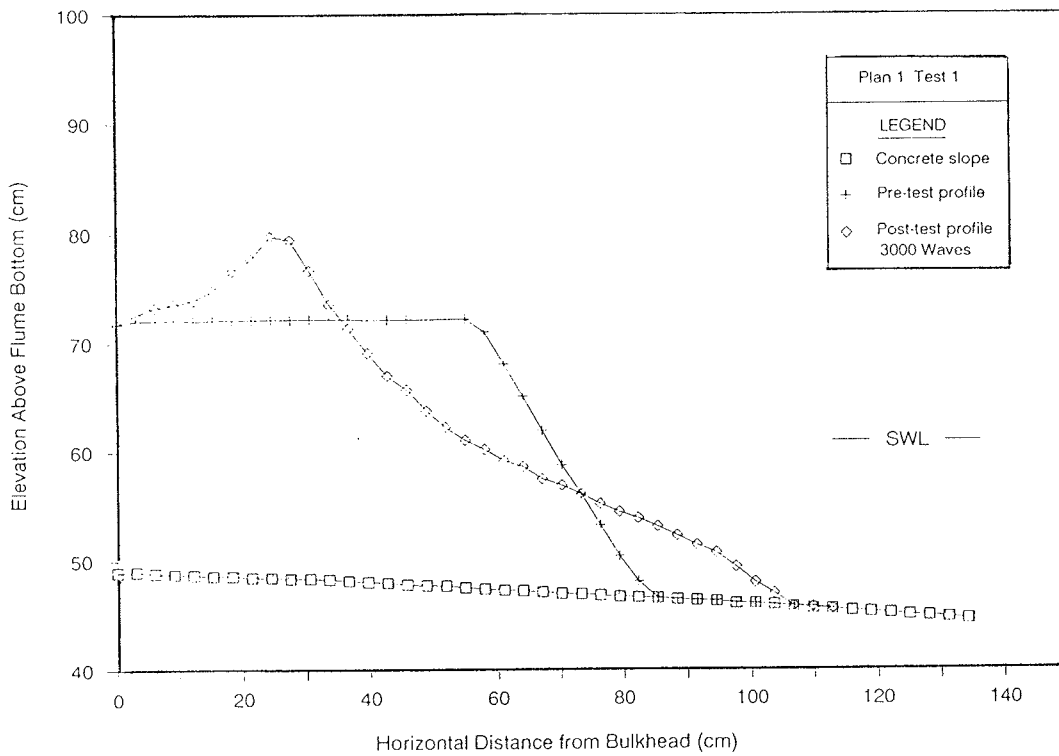20. Profiles shown in the figures are the average of five profile

Figure 4. Typical pre- and posttest profiles

surveys taken along the length of the test section. Surveys were made by
taking soundings with a rod attached to a 15-mm-diam disc by a ball and socket
connection. Soundings were taken every 3.05 cm along the length of the test
section. Very little across-tank variation in the profile was observed during
these tests.

21. A dense limestone was used for the rubble in this study. Because
of its small size, the stone was graded by sieve size. Table 1 summarizes the
gradations and specific gravity of the stone used.

Table 1

Characteristics of Stone Used in This Study

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:	2.68	2.72

10

Test Conditions

22.   Initial test conditions were generated to simulate wave action similar to that found on Lake Michigan near a site in Chicago at Devon Avenue. The initial berm width approximated that obtained by substituting these initial test conditions into 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 that may better represent the conditions for a structure on a body of water smaller than Lake Michigan.

Results

23.   Water depths and wave data collected during the wave flume tests are given in Table 2, and data for the dynamic revetments are listed in Table 3.   Soundings from the tests are given in Appendix A, and initial and equilibrium profiles are illustrated in Appendix B.

24.   It was found that the initial profile adjusts rapidly to incident wave conditions.   For tests with $T_p = 2.5$, there was little change in the profile between 3,000 and 5,000 waves, and for tests where $T_p = 1.75$, there was little change between 3,000 and 4,000 waves.

25.   The Dutch found the shape of dynamic profiles at equilibrium were not very sensitive to initial configuration (van der Meer and Pilarczyk 1986). Verification of this finding was made in a pair of early tests.   Figure 5a shows the initial profiles of Tests 4 and 5, which can be seen to be quite different.   The initial profile for Test 4 is the equilibrium profile at the completion of Test 3, and the initial profile for Test 5 is the standard starting profile used in this study with a horizontal berm and a 1:1 seaward face.   Wave conditions for Tests 4 and 5 were nearly identical, and the equilibrium profiles that were produced were also almost identical, as shown in Figure 5b.   A preliminary conclusion is that the final profile is independent of the initial profile as long as the volume of stone remains constant. This is a very important finding because it would reduce the cost of construction by allowing rough placement of the stone berm.   This conclusion parallels findings from studies of sand beach profile development.   The conservation of

Table 2

Water Depths and Wave Data for Physical Model Tests

| Test No.	No. Waves N	Water Depth at Wave Generator cm	Array 2 Water Depth cm	Water Depth at Toe cm	Array 2 Wave Height Hmo cm	Array 2 Wave Period Tp sec	Array 2 Wave Height Hs cm	Array 2 Wave Period Tz sec	Wave Height at toe Hmo cm	Reflection Coefficient Kr	Wave Height at toe Hmo cm	Deep Water Wave Length Lo cm
1	3000	60.96	37.19	14.81	6.73	2.505	6.64	2.030	4.85	0.4541	297	979
1	5000	60.96	37.19	14.81	6.83	2.526	6.85	2.028	4.93	0.4539	299	995
2	3000	60.96	37.19	14.81	13.31	2.516	13.78	1.724	9.59	0.3550	298	988
3	3000	57.91	34.14	11.77	6.81	2.597	6.75	1.973	4.65	0.4750	276	1052
3	5000	57.91	34.14	11.71	6.92	2.570	6.80	1.971	4.71	0.4670	272	1030
4	3000	57.91	34.14	11.71	13.25	2.516	13.62	1.683	9.03	0.3602	266	988
5	3000	57.91	34.14	11.71	13.03	2.516	13.48	1.689	8.88	0.3540	266	988
6	3000	60.96	37.19	15.30	13.62	2.649	13.72	1.737	9.92	0.3876	320	1095
7	3000	60.96	37.19	15.30	6.86	2.505	6.81	2.028	5.00	0.4590	302	979
7	5000	60.96	37.19	15.30	7.02	2.500	7.01	2.011	5.12	0.4598	301	975
8	3000	60.96	37.19	15.58	14.07	2.643	14.29	1.734	10.31	0.3855	322	1090
8	5000	60.96	37.19	15.58	13.61	2.500	13.74	1.774	10.00	0.3799	304	975
9	3000	60.96	37.19	13.66	7.07	2.542	6.86	2.011	4.95	0.4909	290	1008
10	3000	57.91	34.14	10.61	6.90	2.474	6.80	1.950	4.54	0.4442	249	955
11	4286	57.91	34.14	11.51	11.62	1.813	11.47	1.404	8.01	0.2998	188	513
12	4286	60.96	37.19	14.56	11.49	1.849	11.37	1.413	8.37	0.2755	215	533
13	4286	57.91	34.14	10.61	11.43	1.762	11.34	1.392	7.67	0.2820	176	484
13	7143	57.91	34.14	10.61	11.52	1.737	11.43	1.393	7.75	0.2749	173	471
14	3000	67.06	43.28	15.23	13.46	2.535	13.91	1.790	9.30	0.3768	305	1002
15	3000	67.06	43.28	15.23	11.44	1.752	11.41	1.421	8.12	0.2845	207	479
16	3000	67.06	37.80	15.91	13.51	2.557	14.04	1.753	9.93	0.3773	314	1020
17	3000	60.96	34.14	13.62	12.07	1.745	11.70	1.401	8.86	0.2966	196	475
17	4576	60.96	34.14	13.62	11.45	1.749	11.30	1.414	8.40	0.3217	196	477
18	3000	57.91	34.14	10.92	13.44	2.494	14.01	1.682	8.93	0.3658	255	970
19	3000	57.91	34.14	10.47	5.56	1.774	5.45	1.456	3.71	0.3470	176	491
19	4576	57.91	34.14	10.47	5.61	1.774	5.56	1.455	3.75	0.3393	176	491
20	3000	60.96	37.19	13.52	5.71	1.73	5.53	1.456	4.07	0.3318	193	467
20	4576	60.96	37.19	13.52	5.64	1.741	5.50	1.465	4.02	0.3327	194	473
21	3000	60.96	37.19	13.52	13.38	2.488	13.83	1.688	9.34	0.4222	282	966
22	3000	60.96	37.19	14.60	5.75	1.792	5.69	1.449	4.20	0.3271	208	501
22	4576	60.96	37.19	14.60	5.65	1.73	5.61	1.455	4.15	0.3223	200	467
23	2160	60.96	37.19	14.60	12.38	2.505	12.99	1.741	8.88	0.4271	295	979
24	3000	60.96	37.19	14.60	6.42	2.516	6.41	2.002	4.60	0.4980	296	988
25	3000	60.96	37.19	14.60	10.97	1.709	11.00	1.408	8.05	0.2982	198	456
25	4576	60.96	37.19	14.60	10.78	1.849	10.72	1.417	7.86	0.2990	215	533
26	3000	60.96	37.19	14.60	5.45	1.722	5.39	1.453	4.00	0.3728	199	463
26	4576	60.96	37.19	14.60	5.37	1.786	5.31	1.455	3.93	0.3698	207	498

Table 3

Revetment Data for Physical Model Tests

Test No.	Berm Width cm	Berm Height Above SWL cm	Cross-section Area Revetment At cm^2	Stability Number Ns	Revetment Response Category RRC *	Berm Crest Height hc cm	Berm Crest Length lc cm	Erosion Depth he cm	Erosion Length le cm	Beach Slope Below SWL he/le	Beach Slope Above SWL hc/lc	Percent Energy Dissipation by Revetment Percent D
1	54.86	10.77	1623	3.56	I	18.44	29.79	15.58	52.50	0.297	0.619	79.4
1	54.86	10.77	1623	3.62	I	19.04	26.71	15.36	52.54	0.292	0.713	79.4
2	54.86	10.77	1623	7.05	F	NA	NA	NA	NA	NA	NA	87.4
3	54.86	13.82	1624	3.41	S	17.56	26.75	12.53	40.30	0.311	0.656	77.4
3	54.86	13.82	1619	3.46	S	17.07	26.23	12.66	45.40	0.279	0.651	78.2
4	54.86	13.82	1619	6.64	I	21.12	37.14	12.66	49.73	0.255	0.569	87.0
5	54.86	13.82	1619	6.53	I	21.61	37.78	12.29	46.05	0.267	0.572	87.5
6	70.10	11.28	2061	7.29	I	21.43	36.28	15.88	62.78	0.253	0.591	85.0
7	70.10	11.28	2061	3.68	S	18.50	28.29	15.76	52.48	0.300	0.654	78.9
7	70.10	11.28	2061	3.76	S	19.51	31.31	15.61	52.51	0.297	0.623	78.9
8	77.72	11.15	2253	7.58	I	23.26	39.87	15.88	59.19	0.268	0.583	85.1
8	77.72	11.15	2253	7.35	I	24.20	42.51	16.09	62.65	0.257	0.569	85.6
9	24.38	10.93	862	3.64	F	NA	NA	NA	NA	NA	NA	75.9
10	24.38	13.98	862	3.34	I	16.73	24.08	11.37	36.88	0.308	0.695	80.3
11	54.86	14.25	1637	5.89	S	17.43	26.87	12.71	50.86	0.250	0.649	91.0
12	54.86	11.20	1637	6.15	I	17.89	28.75	15.85	61.12	0.259	0.622	92.4
13	24.38	13.98	862	5.64	I	16.52	25.64	11.35	45.99	0.247	0.644	92.0
13	24.38	13.98	862	5.69	I	16.73	28.60	11.64	46.08	0.253	0.585	92.4
14	77.72	11.27	2230	6.84	I	24.05	41.73	15.68	54.28	0.289	0.576	85.8
15	77.72	11.27	2230	5.97	S	17.74	30.70	16.00	57.69	0.277	0.578	91.9
16	92.96	11.24	2652	7.30	I	24.51	40.48	16.28	57.05	0.285	0.605	85.8
17	24.38	11.04	865	6.51	F	NA	NA	NA	NA	NA	NA	91.2
17	24.38	11.04	865	6.17	F	NA	NA	NA	NA	NA	NA	89.7
18	24.38	14.09	882	6.56	F	NA	NA	NA	NA	NA	NA	86.6
19	24.38	14.09	861	2.73	S	NA	19.18	11.32	40.26	0.281	NA	88.0
19	24.38	14.09	861	2.76	S	NA	19.17	11.32	40.27	0.281	NA	88.5
20	24.38	11.04	861	2.99	I	13.69	19.68	14.38	48.90	0.294	0.696	89.0
20	24.38	11.04	861	2.96	I	13.62	19.54	14.38	49.05	0.293	0.697	88.9
21	24.38	11.04	861	6.86	F	NA	NA	NA	NA	NA	NA	82.2
22	54.86	10.99	1624	3.09	S	14.20	22.11	15.45	49.52	0.312	0.642	89.3
22	54.86	10.99	1624	3.05	S	14.48	21.67	15.45	49.96	0.309	0.668	89.6
23	54.86	10.63	1596	9.22	F	NA	NA	NA	NA	NA	NA	81.8
24	54.86	10.63	1596	4.78	I	20.33	30.56	15.35	50.22	0.306	0.665	75.2
25	54.86	10.63	1596	8.36	I	18.99	26.91	15.50	56.00	0.277	0.706	91.1
25	54.86	10.63	1596	8.16	I	19.11	27.87	15.45	54.50	0.283	0.686	91.1
26	54.86	10.63	1596	4.15	S	14.72	18.69	15.42	48.36	0.319	0.788	86.1
26	54.86	10.63	1596	4.08	S	15.79	22.49	15.45	48.00	0.322	0.702	86.3

* F = failure; S = safe; I = intermediate.

13
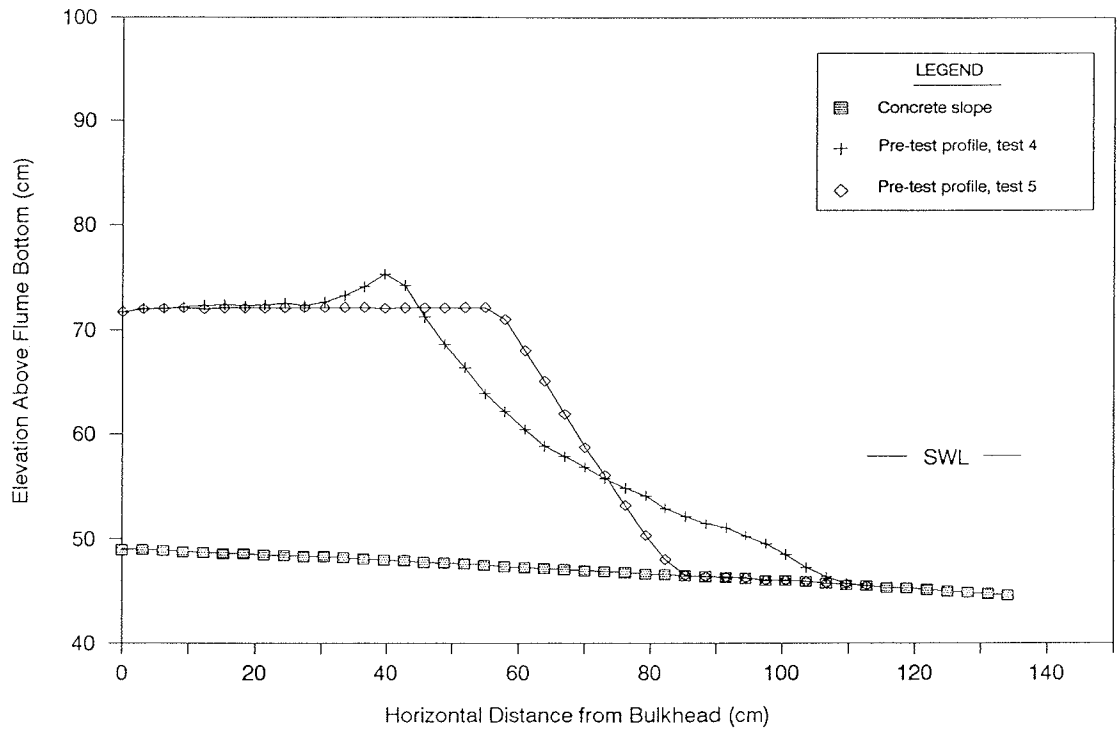
a. Pretest profiles
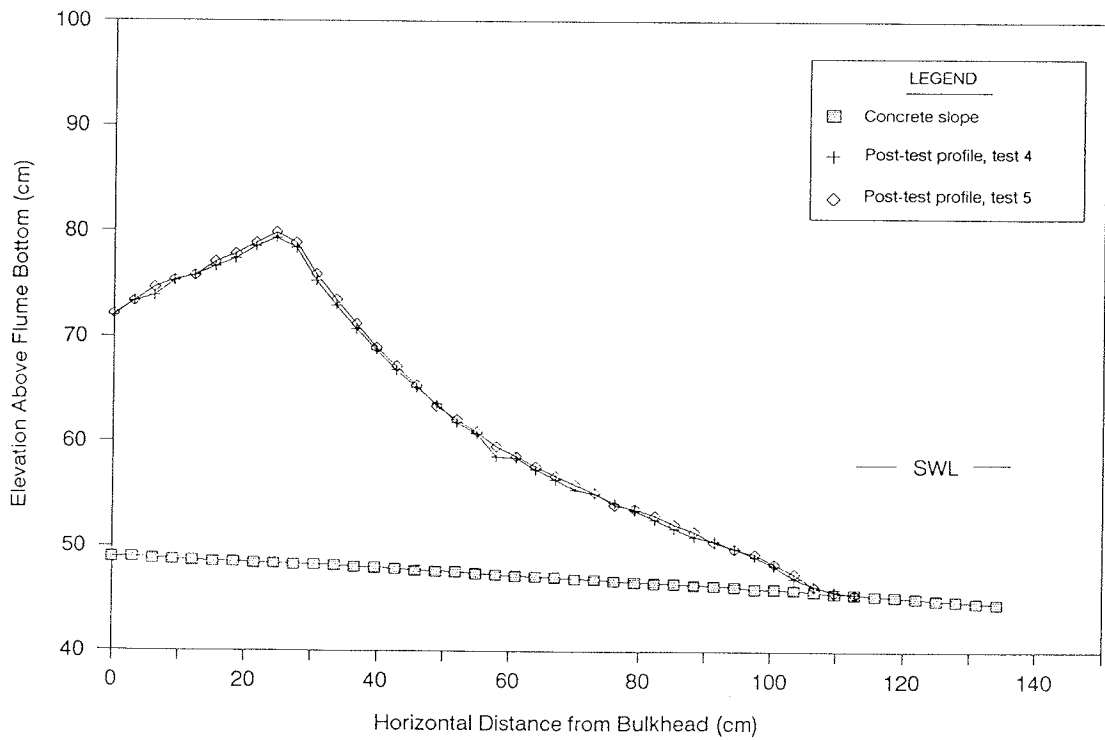
b. Posttest profiles

Figure 5. Pre- and posttest profiles for Tests 4 and 5

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stone places an important constraint on this generality since for severe wave conditions attacking a small revetment, a large portion of the stone can be thrown landward beyond the bulkhead and lost to the system. Loss of stone can cause a nonreversible deterioration of the revetment and ultimately failure.

26. Other interesting features of both the Dutch and CERC dynamic profiles are a pronounced beach crest and a very steep subaerial beach face. During the CERC study, the dynamic profile would typically reach a slope of about 45 deg seaward of the beach crest. The steepest beach face segment observed during this study was 52 deg. The angle of repose for sharp-sided stone or gravel is approximately 45 deg, and this value is assumed to be about the limiting value for the beach face slope.

PART IV: DISCUSSION

27. One development from van der Meer's (1988) research is a method to categorize "structures" from breakwaters to sand beaches (Table 4). The method is based on a stability number similar to the one used extensively by Hudson and Davidson (1975) in their study of breakwater stability. For irregular waves, the stability number is defined as

$$N_s = \frac{H_s}{d_{n50} * \left(\frac{W_r}{W_w} - 1 \right)} \quad or \quad \frac{W_r^{1/3} H_s}{W^{1/3} \left(\frac{W_r}{W_w} - 1 \right)} \tag{7}$$

where w_w is the unit weight of water, with $w_w = 1.000$ g/cm^3 for fresh water and $w_w = 1.025$ g/cm^3 for seawater. When stone sizes are relatively small, as in this study, $d_{n(50)}$ is determined by sieve analysis. Energy-based wave parameters are used in this study so the zeroth moment wave height H_{mo} (measured at Array 2) is used in Equation 7 rather than H_s . For this study, the range of stability numbers is from 2.7 to 9.2. Since CERC tests are run in shallow water where H_s is typically greater H_{mo} , the stability numbers from this study will be somewhat lower than van der Meer's. Regardless of differences, tests from this study fall into van der Meer's "berm breakwater and S-shaped profiles" and "dynamically stable rock slopes" categories (see Table 4 taken from van der Meer and Pilarczyk (1987).

Table 4

Structure Classification Based on van der Meer and Pilarczyk (1987)

Structure	Range of Stability Number
Statically stable breakwaters	N_s = 1-4
Berm breakwater and S-shaped profiles	N_s = 3-6
Dynamically stable rock slopes	N_s = 6-20
Gravel beaches	N_s = 15-500
Sand beaches	N_s > 300

28. Johnson (1987) remarks about the impressive aspect of waves building high, steep beach faces on Lake Michigan, and this feeling was shared by laboratory observers watching the rapid development of the beach face during small-scale tests. The reason for these beach features relates to the high porosity of the rubble, estimated to be about 45 percent. If the waves are large enough to mobilize the rubble, then runup will carry some of it upslope, but the return flow will not carry all of it downslope until the profile has reached equilibrium. Therefore, the subaerial beach face is about equal to the angle of repose largely because the runup flow drains away so quickly. The extent of particle mobilization is measured by the stability number N_s, defined by Equation 7. Height of the berm crest is a conspicuous feature that can be easily identified and accurately measured. The berm crest height is at the approximate upper limit of wave runup (Powell 1988). Visual observations of the tests indicate that the "mature" crest is only occasionally overtopped. Therefore, berm crest height is a good measure of extreme wave runup height.

Critical Mass Analysis

29. To evaluate economic feasibility of a rubble structure, it is clearly necessary to determine the minimum amount of stone that will provide the desired protection. This minimum quantity (volume per unit length of revetment) is referred to in this study as the critical mass.

30. All of the test results were classified into one of three revetment response categories. When wave conditions were severe in relation to the quantity of stone in the revetment, wave action eroded the rubble, usually by carrying it over the bulkhead, until waves impacted directly against the bulkhead. This category was designated failure, denoted by "F" in Table 3. When the amount of stone in a revetment was large in relation to wave conditions, development of the berm crest had enough room so that neither stone nor water was carried over the bulkhead. This category was designated safe, denoted by "S" in Table 3. The third category fell between safe and failure and occurred when the berm crest buildup extended far enough landward to reach the bulkhead and there was at least some overtopping of the bulkhead by both water and stone. This category was designated intermediate, denoted by "I" in Table 3. The three RRC's are illustrated in Figure 6.
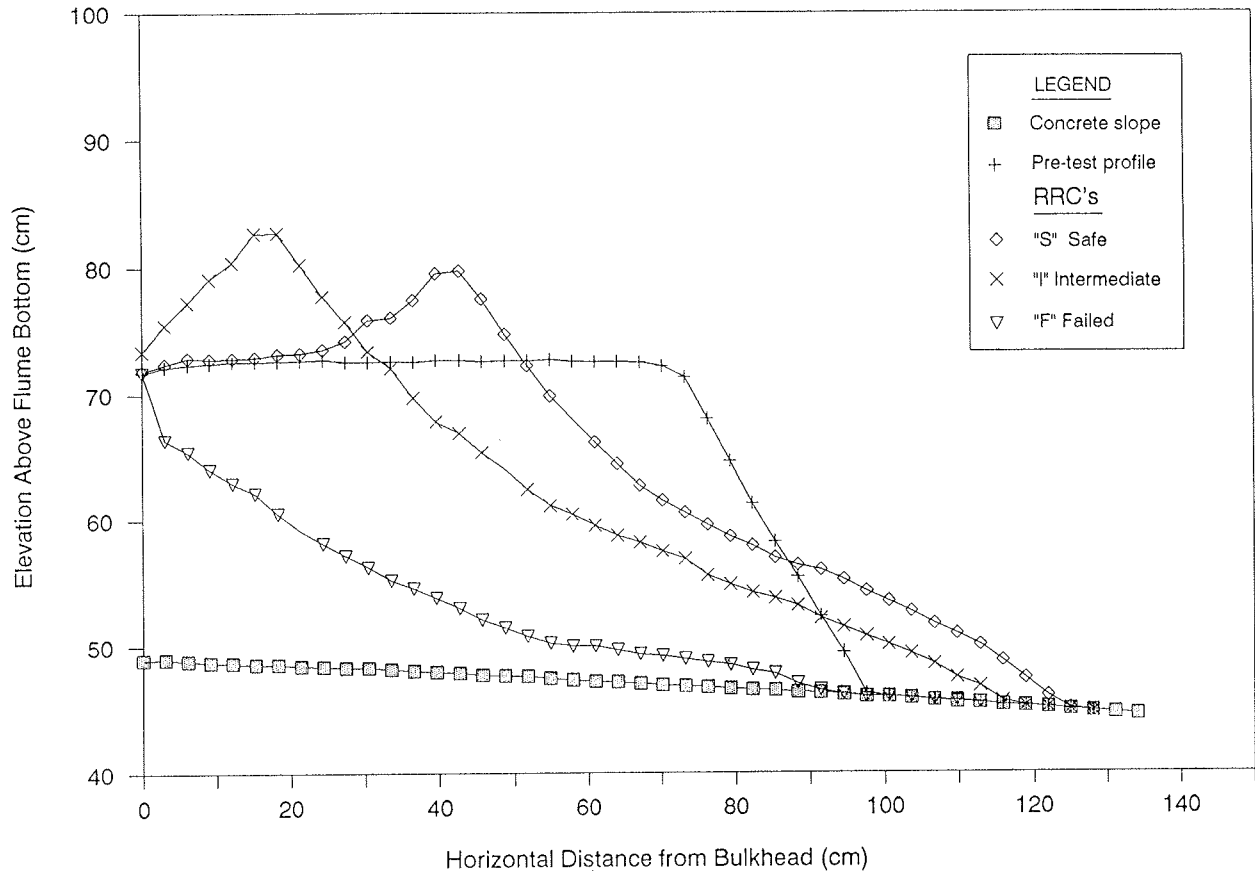
Figure 6. Typical equilibrium profiles illustrating the safe,
intermediate, and failed revetment response categories

31. To calculate 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 define these characteristic dimensions as functions of local wave steepness H_{mo}/L_p, where L_p is the wavelength determined by linear wave theory for the depth at the toe and the peak period.

$$\frac{h_c}{H_{mo}} = 0.270 * \left(\frac{H_{mo}}{L_p}\right)^{-0.645} , \ R^2 = 0.96 \tag{8}$$

$$\frac{l_c}{H_{mo}} = 0.677 * \left(\frac{H_{mo}}{L_p}\right)^{-0.521} , \ R^2 = 0.92 \tag{9}$$

$$\frac{l_e}{d_s} = \exp\left[2.24 * \left(\frac{H_{mo}}{L_p}\right)^{+0.143}\right] , \quad R^2 = 0.64 \tag{10}$$

Equations 8, 9, and 10 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 7, 8, and 9 show observed data with regression trends for Equations 8, 9, and 10, respectively. Although the smaller stone was not included in the analysis, it has been included in Figures 7, 8, and 9 to illustrate the applicability of the equations to other stone sizes. Stone sizes are denoted by symbols in these figures, with "L" indicating the larger stone and "S" indicating the smaller stone.

32. Characteristic dimensions determined by Equations 8, 9, and 10 may be used to determine a pseudo-cross-sectional area of the mature revetment A_s where

$$A_s = (d_s + h_c) * (l_e + l_c) \tag{11}$$

This equation is essentially just length times height. 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 then determined from A_s for the desired degree of protection. Total design volume is denoted A_t (cm^3/cm), which includes void space. Figure 10 shows the revetment response category versus the ratio of A_t to A_s. The two solid lines illustrate values of A_t/A_s of 0.67 and 0.46, and indicate that if

$$A_t > 0.67 A_s \tag{12}$$

the revetment is safe, and if

$$A_t < 0.46 A_s \tag{13}$$

the revetment will fail. Values of A_t/A_s between 0.46 and 0.67 are in the intermediate revetment response category. This guidance is based on laboratory tests with a range of stability numbers (Equation 7) from 2.7 to 9.2.

33. Johnson's (1987) criteria for protecting eroding portions of the Lake Michigan shoreline was 36 metric tons/metre. Assuming a porosity of 35 percent and a unit weight of 2.64 g/cm^3, this guidance is equal to 21 m^3/m. Figure 11 compares this guidance with the design volumes determined by
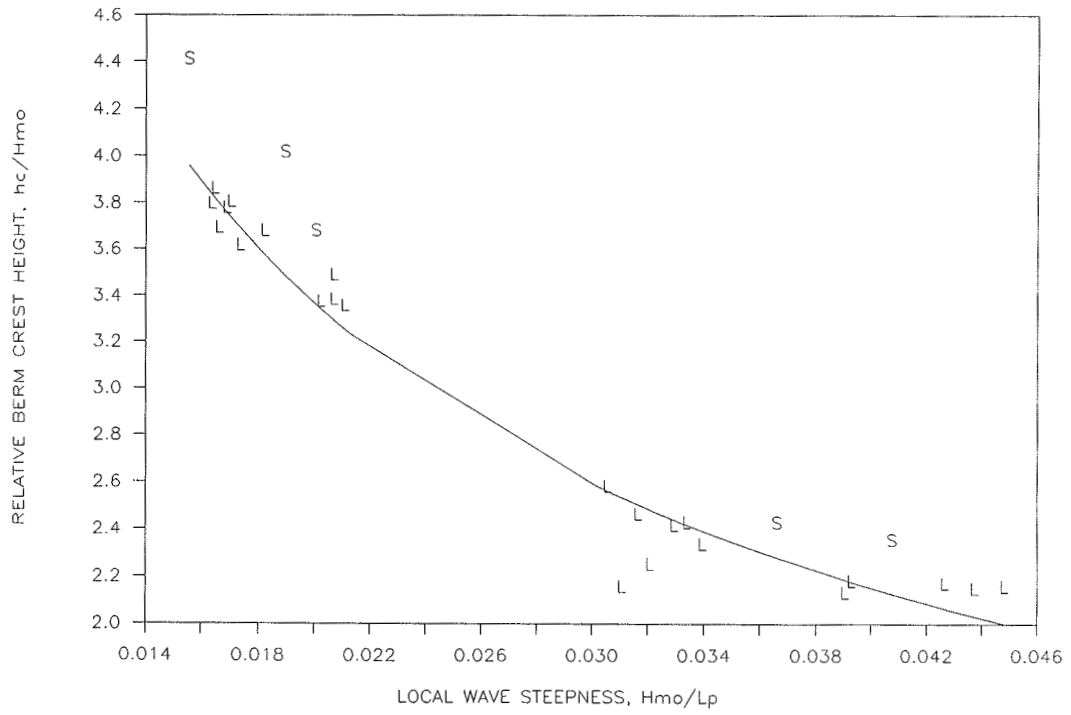
19

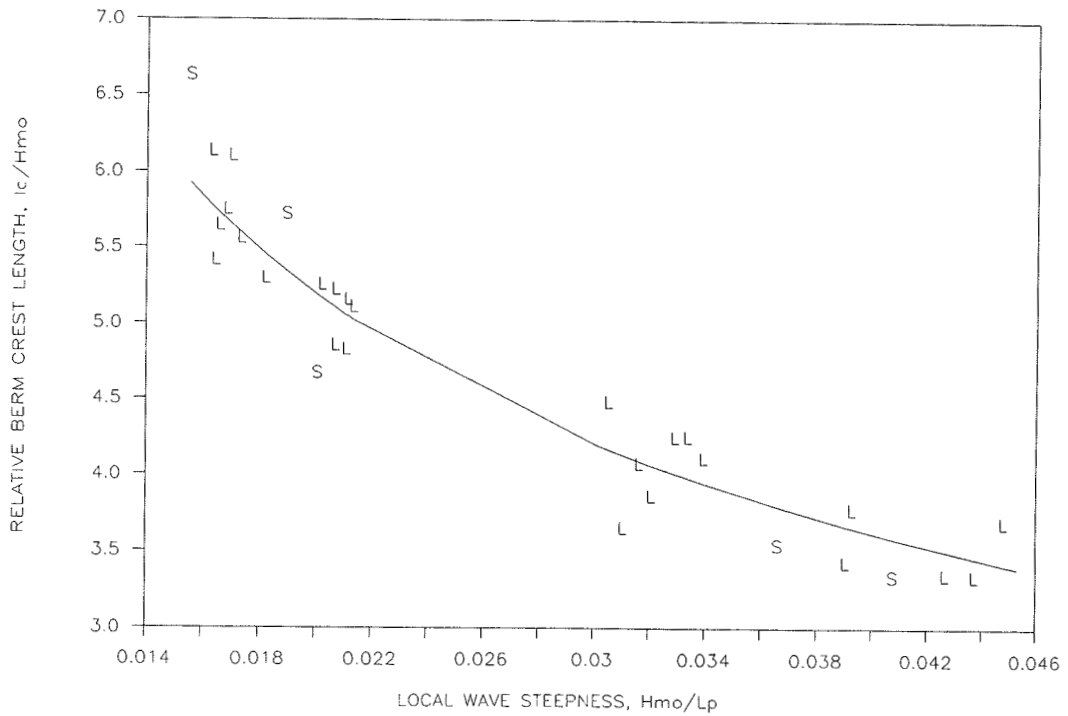Figure 7. Calculated and observed relative berm crest heights as a function of local wave steepness

Figure 8. Calculated and observed relative berm crest lengths as a function of local wave steepness
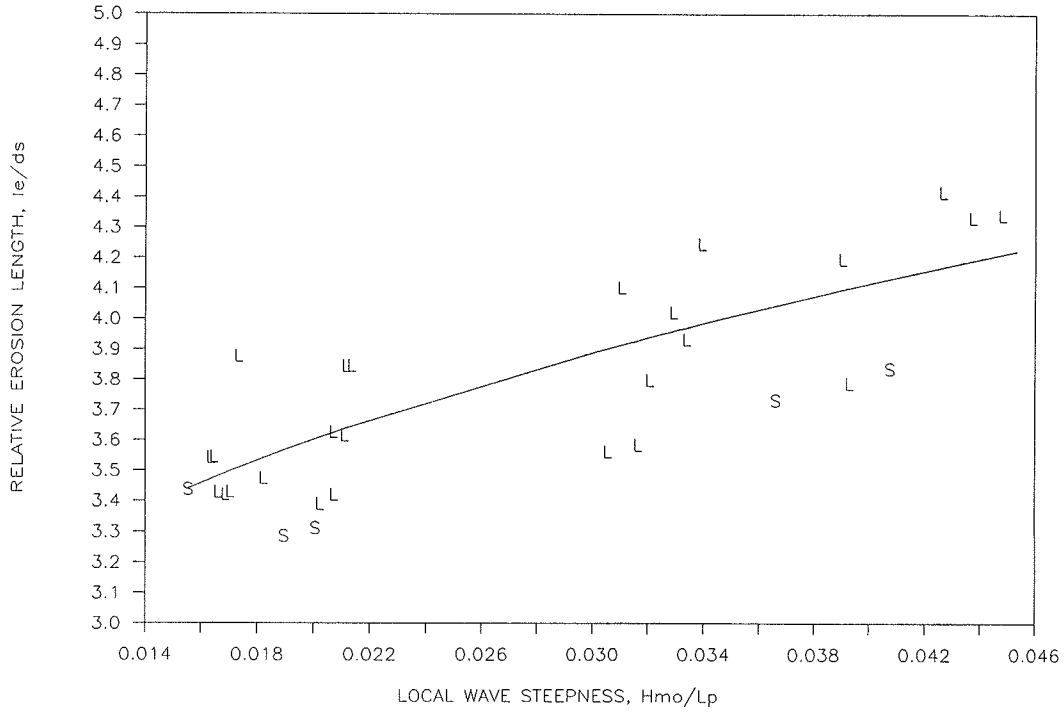
20

Figure 9. Calculated and observed relative erosion lengths
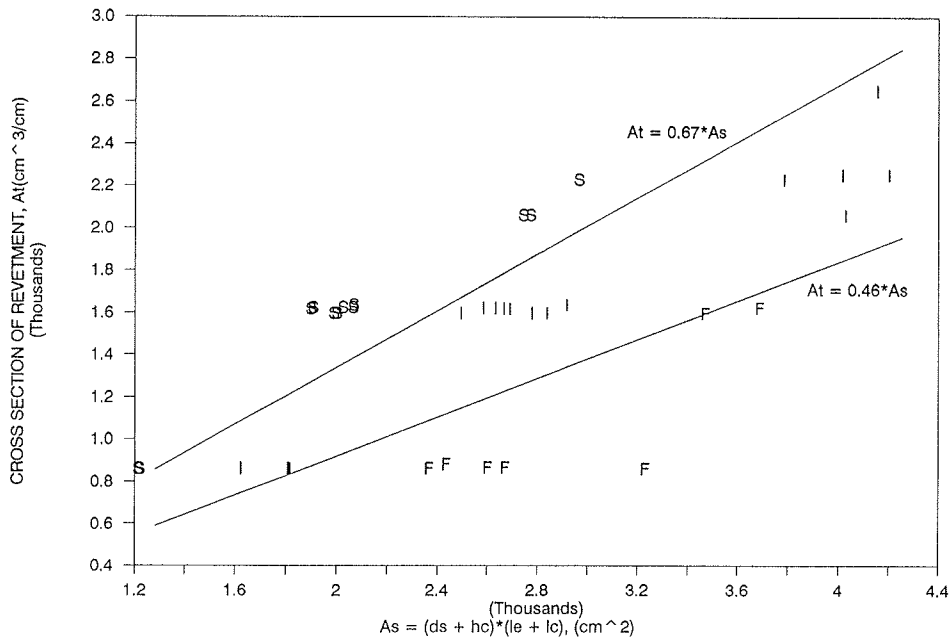as a function of local wave steepness

Figure 10. Total area of berm, A_2 , versus calculated A_s , with
with observed RRC. Solid lines illustrate that for $A_t < 0.46$ as
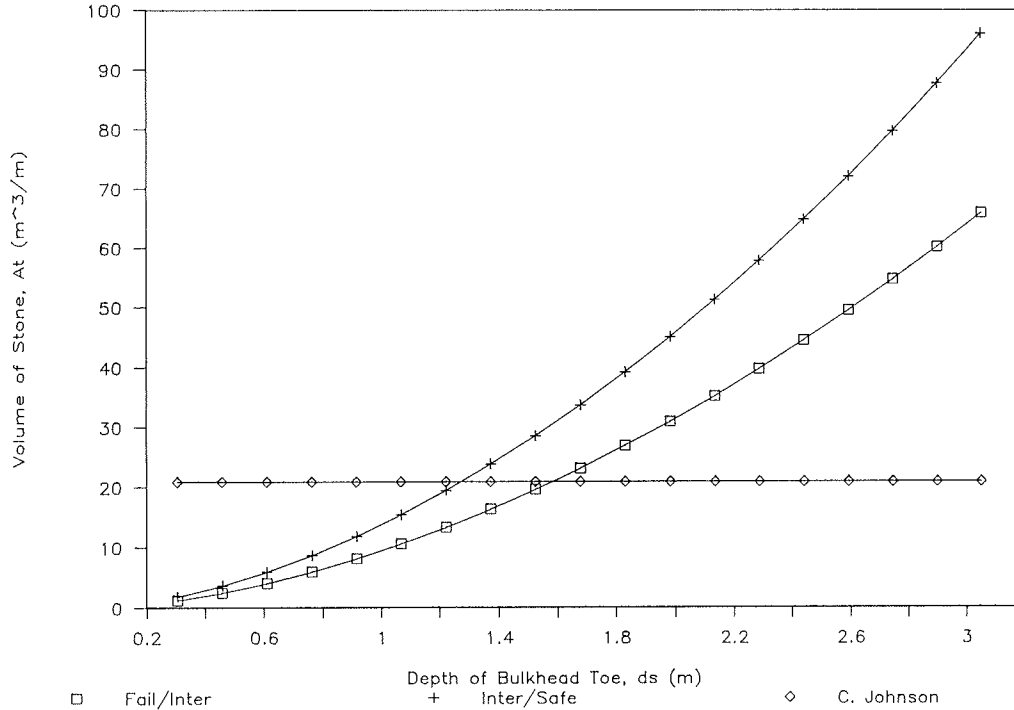the revetment will fail

Figure 11. Comparison of stone volume calculated in this report versus guidance in Johnson (1987) for Great Lakes storm with $T_p = 10$ sec and $H_{mo} = 0.6 * d_s$

Equations 8 through 13. Figure 11 assumes design conditions of $T_p = 10$ sec and $H_{mo} = 0.6 * d_s$.

Wave Reflection and Energy Dissipation

34. The reflection coefficient is defined as the square root of the ratio of reflected wave energy to incident wave energy (Goda and Suzuki 1976). 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:

$$K_r = \frac{1.0}{1.0 + CO * \left[\dfrac{d_{n(50)}}{L_o}\right]^{C1} * \exp\left(\dfrac{C2}{H_{mo}/L_o}\right)} \tag{14}$$

where L_o is the deepwater wavelength. Dimensionless regression coefficients are given by,

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$$CO = 23.4$$
$$C1 = 0.312$$
$$C2 = -0.00374$$

Equation 14 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 this analysis since at failure a substantial part of the reflection is from the vertical bulkhead. Percentage of incident wave energy dissipated by a dynamic revetment can be estimated by using Equation 14 and the relation,

$$\%D = (1.0 - K_r^2) * 100\% \tag{15}$$

where $\%D$ is the percent energy dissipation. Observed data give 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 make good wave absorbers.

PART V: SUMMARY AND CONCLUSIONS

35.   A series of laboratory tests were conducted to investigate the response of dynamic revetments to shallow-water wave conditions.   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.

36.   It was found that the equilibrium dynamic revetment profile was not sensitive to the initial profile.   This finding 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 provides a good indication of the extreme wave runup.

37.   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, zeroth moment wave height, and period of peak energy density.

38.   The influence of the initial berm width and berm height above the still-water level were two of the major variables investigated in this study. It was found that these parameters play a major role in determining how much stone is required to protect a vertical bulkhead from direct wave attack.

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APPENDIX A: TABLE OF PROFILE SOUNDINGS

Average Elevation Above Tank Floor (cm)

Horiz. Dist. (cm)	Concrete Slope Avg. (cm)	Plan 1 Test 1 Before Testing Avg. (cm)	Plan 1 Test 1 After 3000 Waves Avg. (cm)	Plan 1 Test 1 After 5000 Waves Avg. (cm)	Plan 1 Test 2 After 3000 Waves Avg. (cm)	Plan 1 Test 3 After 3000 Waves Avg. (cm)	Plan 1 Test 3 After 5000 Waves Avg. (cm)	Plan 1 Test 4 After 3000 Waves Avg. (cm)	Plan 1 Test 5 After 5000 Waves Avg. (cm)
0.00	48.96	71.73	71.73	71.73	71.73	71.73	71.73	71.97	72.16
3.05	48.99	71.97	72.58	72.70	66.37	72.19	72.07	73.35	73.38
6.10	48.87	72.07	73.22	73.01	65.39	72.13	72.07	73.83	74.66
9.14	48.75	72.10	73.50	73.88	64.02	72.22	72.19	75.27	75.39
12.19	48.69	72.00	73.92	74.55	62.92	72.28	72.31	75.81	75.78
15.24	48.60	72.10	74.78	75.27	62.10	72.46	72.40	76.67	77.16
18.29	48.60	72.10	76.58	76.94	60.51	72.34	72.28	77.46	77.89
21.34	48.47	72.10	77.67	78.97	59.17	72.46	72.40	78.53	78.92
24.38	48.41	72.13	79.75	80.34	58.17	72.28	72.52	79.38	79.87
27.43	48.32	72.13	79.44	79.62	57.19	72.55	72.34	78.41	78.89
30.48	48.32	72.13	76.64	76.66	56.28	73.10	72.67	75.33	75.94
33.53	48.23	72.13	73.47	73.67	55.24	73.68	73.28	72.98	73.53
36.58	48.08	72.13	71.27	71.72	54.63	74.72	74.08	70.69	71.24
39.62	47.99	72.07	69.05	69.05	53.87	75.81	75.33	68.71	68.93
42.67	47.93	72.10	66.98	67.48	53.05	74.63	74.20	66.79	67.19
45.72	47.74	72.10	65.70	63.65	52.13	72.37	71.21	65.15	65.39
48.77	47.68	72.10	63.74	62.25	51.49	68.41	68.65	63.59	63.38
51.82	47.65	72.13	62.25	61.01	50.82	65.85	66.40	61.82	62.10
54.86	47.47	72.13	61.03	60.11	50.27	63.96	63.90	60.76	60.97
57.91	47.35	71.00	60.18	59.09	50.03	62.04	62.19	58.56	59.51
60.96	47.25	68.04	59.14	58.34	50.03	60.39	60.48	58.47	58.65
64.01	47.19	65.12	58.59	57.51	49.75	59.11	58.87	57.31	57.65
67.06	47.07	61.95	57.47	56.72	49.42	58.01	57.86	56.40	56.73
70.10	46.95	58.75	56.89	56.21	49.27	56.95	56.86	55.42	55.94
73.15	46.89	56.06	56.09	55.51	48.99	55.88	55.79	55.06	55.18
76.20	46.77	53.20	55.18	54.51	48.78	54.87	54.87	54.20	53.96
79.25	46.65	50.36	54.42	53.83	48.53	54.11	54.11	53.47	53.66
82.30	46.58	48.02	53.87	53.09	48.14	53.32	52.95	52.62	53.02
85.34	46.49	46.49	53.08	52.27	47.83	52.47	52.16	51.74	52.16
88.39	46.37	46.37	52.22	51.34	47.04	51.67	51.46	50.97	51.49
91.44	46.31	46.31	51.37	50.61	46.49	51.19	51.06	50.58	50.46
94.49	46.22	46.25	50.67	49.84	46.25	50.21	50.24	49.88	49.81
97.54	46.00	46.04	49.33	48.41	46.04	49.14	49.54	49.05	49.33
100.58	46.00	46.00	47.93	47.38	46.00	48.05	48.50	48.20	48.38
103.63	45.91	45.91	46.95	45.94	45.91	47.19	47.22	47.01	47.50
106.68	45.73	45.73	45.73	45.60	45.73	45.73	46.34	46.28	46.25
109.73	45.58	45.70	45.70	45.47	45.70	45.70	45.70	45.79	45.70
112.78	45.49	45.49	45.49	45.35	45.49	45.49	45.49	45.49	45.49

A2

Average Elevation Above Tank Floor (cm)

Horiz. Dist. (cm)	Concrete Slope Avg. (cm)	Plan 2 Test 6 Before Testing Avg. (cm)	Plan 2 Test 6 After 3000 Waves Avg. (cm)	Plan 2 Test 7 After 3000 Waves Avg. (cm)	Plan 2 Test 7 After 5000 Waves Avg. (cm)
0.00	48.96	71.73	73.41	71.85	71.73
3.05	48.99	72.19	75.54	72.43	72.40
6.10	48.87	72.37	77.31	72.89	72.71
9.14	48.75	72.49	79.14	72.83	72.83
12.19	48.69	72.58	80.45	72.86	72.89
15.24	48.60	72.65	82.67	72.98	72.95
18.29	48.60	72.68	82.73	73.19	73.19
21.34	48.47	72.71	80.27	73.26	73.26
24.38	48.41	72.80	77.83	73.56	73.83
27.43	48.32	72.65	75.78	74.23	74.44
40.48	48.32	72.65	73.44	75.94	74.93
33.53	48.23	72.68	72.10	76.15	76.52
36.58	48.08	72.65	69.72	77.55	78.44
39.62	47.99	72.74	67.83	79.59	80.81
42.67	47.93	72.74	66.95	79.81	80.30
45.72	47.74	72.68	65.39	77.61	76.94
48.77	47.68	72.71	64.08	74.81	74.38
51.82	47.65	72.71	62.43	72.31	71.91
54.86	47.47	72.80	61.18	69.90	70.02
57.91	47.35	72.68	60.45	68.07	67.92
60.96	47.26	72.65	59.60	66.18	66.00
64.01	47.19	72.65	58.78	64.48	64.20
67.06	47.07	72.55	58.23	62.74	62.50
70.10	46.95	72.28	57.53	61.58	61.62
73.15	46.89	71.37	56.92	60.61	60.45
76.20	46.77	68.04	55.64	59.63	59.63
79.25	46.65	64.69	54.91	58.72	58.62
82.30	46.59	61.31	54.30	58.02	57.83
85.34	46.49	58.26	53.81	57.01	57.16
88.39	46.37	55.52	53.23	56.46	56.28
91.44	46.31	52.41	52.22	56.03	55.70
94.49	46.22	49.54	51.52	55.30	54.72
97.54	46.01	46.28	50.82	54.36	53.99
100.58	46.01	46.01	50.15	53.60	53.26
103.63	45.91	45.91	49.45	52.77	52.68
106.68	45.73	45.73	48.60	51.80	51.92
109.73	45.70	45.70	47.53	51.00	51.00
112.78	45.49	45.49	46.80	50.12	49.94
115.82	45.34	45.34	45.64	48.84	48.90
118.87	45.24	45.24	45.24	47.44	47.74
121.92	45.12	45.12	45.12	46.07	46.34
124.97	44.97	44.97	44.97	45.06	45.06
128.02	44.85	44.85	44.85	44.85	44.85

Horiz. Dist. (cm)	Concrete Slope Avg. (cm)	Plan 3 Test 8 Before Testing Avg. (cm)	Plan 3 Test 8 After 3000 Waves Avg. (cm)	Plan 3 Test 8 After 5000 Waves Avg. (cm)
0.00	48.96	71.73	71.73	73.41
3.05	48.99	71.91	74.90	75.63
6.10	48.87	72.19	76.70	77.64
9.14	48.75	72.16	78.44	80.30
12.19	48.69	72.31	80.33	82.03
15.24	48.60	72.37	82.06	83.74
18.29	48.60	72.40	83.47	85.51
21.34	48.47	72.43	84.32	85.05
24.38	48.41	72.55	84.56	82.70
27.43	48.32	72.49	82.25	80.02
30.48	48.32	72.55	79.81	78.41
33.53	48.23	72.55	77.80	76.03
36.58	48.08	72.65	75.36	74.11
39.62	47.99	72.52	73.56	72.40
42.67	47.93	72.77	72.04	70.05
45.72	47.74	72.68	69.84	68.35
48.77	47.68	72.68	67.89	66.67
51.82	47.65	72.71	66.09	64.84
54.86	47.47	72.77	65.03	63.65
57.91	47.35	72.77	63.68	62.40
60.96	47.26	72.55	62.28	61.25
64.01	47.19	72.55	61.40	60.42
67.06	47.07	72.65	60.27	59.60
70.10	46.95	72.58	59.39	59.11
73.15	46.89	72.34	58.59	58.32
76.20	46.77	72.37	57.92	57.77
79.25	46.65	72.22	57.16	57.01
82.30	46.59	69.66	56.43	56.37
85.34	46.49	66.28	55.94	55.64
88.39	46.37	62.98	55.18	55.00
91.44	46.31	59.81	54.21	54.51
94.49	46.22	56.28	53.50	53.87
97.54	46.01	53.41	52.99	53.14
100.58	46.01	50.46	51.98	52.22
103.63	45.91	47.53	51.74	51.13
106.68	45.73	45.73	50.76	50.06
109.73	45.70	45.70	49.94	49.18
112.78	45.49	45.49	48.99	48.32
115.82	45.34	45.34	48.47	47.23
118.87	45.24	45.24	47.07	45.98
121.92	45.12	45.12	45.91	45.49
124.97	44.97	44.97	44.97	44.97
128.02	44.85	44.85	44.85	44.85
131.06	44.73	44.73	44.73	44.73
134.11	44.57	44.57	44.57	44.57

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