

CHAPTER 91

Two Dimensional Effects in Modelling Berm or Reshaping Breakwaters

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Introduction

In recent years, the failure of many large conventional type breakwater structures has led to a careful examination of the physical processes of wave-structure interaction. Although naturally armouring structures, which gain their stability as a consequence of profile readjustment due to wave action, have been around in various forms for hundreds of years, the high incidence of failure of conventional structures has led to their increased use in the past decade.

The interaction of an incident wave with a rubblemound breakwater results in complex flow patterns involving unsteady non-uniform flow (Hall (1987)). In most cases, it is desirable to construct a breakwater which works in harmony with the flow field; that is to construct a structure with a geometry and armour stone weight gradation which results in natural profile readjustment and subsequent minimization of the applied hydrodynamic loadings.

A reshaping or berm breakwater can be described as a mound of rock, often comprised of a wide range of stone sizes, which undergoes reshaping as a result of wave-structure interaction. As a consequence of this wave action, a stable profile is developed. Two major processes occur in the development of the stable profile. First, the overall geometry of the structure responds to the nature of the hydrodynamic loadings. Material is sorted and redistributed into a profile which acts to minimise the applied forces by altering the flow field kinematics. Secondly, this natural sorting leads to consolidation (densification) of the armour layer as stones that move eventually find voids into which the

nest.

This type of structure has been used extensively in the past decade and it has been found that these structures are significantly less expensive than more conventional breakwaters designed in accordance with guidelines given in design manuals such as the US Army Corps of Engineers Shore Protection Manual. The armour stones required are smaller than those required by conventional stability formulae and a much wider gradation can be used. This allows for the design to be based on the actual quarry output rather than some pre-conceived specification for stone for which a quarry must be found. Experimental studies have indicated that the reshaped breakwater profile can be closely predicted for the design wave conditions and the available material properties. This profile can be used as an initial design. Currently model studies are used to optimize the design and minimise the cost.

Limited research on the two dimensional stability of these breakwaters has been undertaken in the past decade; and although virtually no research has been undertaken with respect to the three dimensional stability of these structures, Hall et al. (1983), Hall (1987), and Burcarth and Frigaard (1987) have shown that significant three dimension effects can exist. These effects may be classified as purely three dimensional effects (due to attenuation of energy within the cross section) or effects resulting from the variation of the angle of wave attack, thus affecting the erosion prone areas (particularly the head of the breakwater). The later effects are typically what is thought of as being dominant; however, Hall et al. (1983) has shown that the difference between test results from narrow flumes and wider three dimensional test sections (for incident waves parallel to the trunk of the structure) can be substantial. The tests reported in this paper were designed to evaluate these effects, in particular to evaluate if the reshaping of these breakwaters is influenced by the flume width used in the tests.

Testing Programme

Experimental studies were undertaken using the facilities of the Coastal Engineering Research laboratory of Queen's University, Kingston, Canada. The studies were undertaken primarily to investigate the mechanism of reshaping of berm breakwaters and to evaluate the specific influence of the various parameters that may affect the reshaping process. These parameters include

wave height, wave period, wave groupiness, duration of each segment of the design storm, the gradation of the armour stones, and the percentage of rounded stones occurring in the gradation.

Tests were run in a 22 x 26 metre three dimensional basin where three separate "flumes" having widths of 1.2, 2.4 and 4.0 metres were constructed parallel to each other. Identical test structure were built in each of the flumes and were then subjected simultaneously to the same wave conditions. Basin layout is shown in Figure 1.

Profiles of the test breakwaters were measured at several locations along the structure following each segment of the design storm as shown in Figure 1. The profiler used to measure the three dimensional profiles was a trailing arm profiler which provided a continuous reading of the elevation of the breakwater with horizontal distance.

Tests undertaken in the three dimensional modelling programme utilized a core material shown in Figure 2. Two different gradations of armour were used having the following characteristics:

	D_{50} (mm)	D_{85}/D_{15}	D_{max} (mm)
Gradation 1	19	1.9	32
Gradation 2	14	2.0	27

Tests were undertaken on a berm breakwater whose general configuration is shown in figure 3. The basic geometry of the test structure was determined based on the large number of berm breakwaters tested at various institutions in Canada and abroad. This configuration has been found to be easy to construct in prototype. Five capacitance type water level transducers were placed just in front of the breakwater to measure the dynamic water level fluctuations. An additional water level transducer was placed immediately in front of the paddle to measure the deep water wave conditions.

Preliminary work was done before the breakwater was built to synthesize, generate and sample the waves and determine the appropriate span settings (settings that control the maximum excursion of the wave paddle) for the wave paddle. All the tests in subsequent experiments were then carried out using the same waves and same span settings. Additionally, the wave records measured during tests with the breakwater in place were separated into

incident and reflected components. In most cases, excellent agreement was obtained between results of the reflection analysis and those measured with no structure in the flume. The bounded long wave component resulting from the occurrence of wave groups was removed from the analysis using a software filter.

All tests were conducted with irregular waves synthesized using the GEDAP procedure of the National Research Council of Canada Hydraulic Laboratory. Jonswap spectra were generated having periods of peak energy density, ranging from 1 second to 2 seconds and significant wave heights from 6 cm to 14 cm.

The notation of a design storm consisting of several segments of specific wave height-period combinations, which altogether simulate the growth and decay of waves during a hypothetical design storm was utilized in the tests. A total of four design storms were utilized in the three dimensional tests and are detailed in Tables 1 to 4.

Table 4 Wave Climate # 1-3D

Segment	H_s (m)	T_p (s)	Duration (min)
1	.06	1.2	60
2	.08	1.4	70
3	.1	1.6	80
4	.12	1.8	90
5	.14	2.0	100
6	.12	1.8	90
7	.10	1.6	80
8	.08	1.4	70

Table 2 Wave Climate 2-3D

Segment	H_s (m)	T_p (s)	Duration (min)
1	.08	1.2	60
2	.10	1.4	70
3	.12	1.6	80
4	.14	1.8	90
5	.16	2.0	100
6	.14	1.8	90
7	.12	1.6	80
8	.10	1.4	70

Table 3 Wave Climate 3-3D

Segment	H_s (m)	T_p (s)	Duration (min)
1	.10	1.2	60
2	.12	1.4	70
3	.14	1.6	80
4	.16	1.8	90
5	.18	2.0	100
6	.16	1.8	90
7	.14	1.6	80
8	.12	1.4	70

Table 4 Wave Climate 4-3D

Segment	H_s (m)	T_p (s)	Duration (min)
1	.12	1.2	60
2	.14	1.4	70
3	.16	1.6	80
4	.18	1.8	90
5	.20	2.0	100
6	.18	1.8	90
7	.16	1.6	80
8	.14	1.4	70

Each segment of each storm was run for a time equivalent to 3000 waves attacking the structure. Profiles were measured before the test and after each segment of test. For certain tests, several intermediate profiles were measured after 500, 1000, 1500 and 2000 waves. To assess the long term reshaping process, some tests were carried out for a duration of 36000 waves with profiles measured every 3000 waves. These tests were used to assess the reshaping of breakwaters as a function of storm segment duration. The berm breakwater was considered to have failed when erosion of the berm progressed landward to the intersection of the horizontal berm with the upper 1:3 slope (see figure 3). A total of 12 series of tests were undertaken and are described in Table 5.

Table 5

Test Series	Wave Climate	Initial Berm Width (m)
1	1-3D	.40
2	1-3D	.45
3	2-3D	.50
4	2-3D	.50
5	2-3D	.60
6	3-3D	.60
7	3-3D	.70
8	4-3D	.70
9	4-3D	.60
10	2-3D *GF=0.2	.45
11	2-3D *GF=0.75	.45
12	2-3D *GF=1.2	.45

Test Procedures

The following general procedures were followed during each test:

- (1) A test was not commenced until the water surface in the flume was completely calm.
- (2) Predetermined settings on the hydraulic wave generator were used for all wave conditions.
- (3) The same number of samples of water level variations were taken. Sampling would start on the passage of the third wave.
- (4) The locations of the water level transducers did not vary for test to test.
- (5) Observations were made regarding stone movement resulting from wave attack, both initially and after the breakwater profile reached near equilibrium.

- (6) The breakwater was not rebuilt following each segment of wave attack. Rebuilding of the entire breakwater was done following the completion of a test storm and carried out under water, to simulate, as close as possible, prototype construction conditions.
- (7) The water level transducers and profiler were calibrated every 3 days and were found to remain stable over the duration of the testing programme.

Results

It was found that the largest portion of berm erosion occurred during the attack of the first few relatively big waves in the wave train. The zone in which movement of stones occurred was located at the seaward edge of the berm. The stones moved by rolling about the slope. On passing of the first few waves, there was little sign of motion of stones in the landward region of the berm; the principal mode of reshaping was by rounding-off of the exposed corner of the berm.

After approximately five to ten big waves, there was very little mass movement, and subsequently, stones moved individually rather than "en masse". With time, the initial berm receded and armour stones were individually removed by the wave forces and deposited at some other location on the slope. With each impinging wave breaking on the berm, stones were removed from the outer edge of the berm and carried by uprush and downrush moving to and fro on the profile formed as a result of erosion. A net migration of stones offshore was observed from the profiles measured followed each section of the design storm. The reshaping process was usually too slow to be noticeable to the human eye (apart from the initial "shakedown" of the mound).

Figure 4 gives an example of average berm erosion (measured from the seaward edge of the structure) as a function of significant wave height for a particular test. In general comparable recession rates were observed in each flume over the course of the design storm. The net recession in each flume at the end of the test is typically within a range of 5-10%. Considering that on average 450 to 500 mm of erosion is experienced and given that $D_{50} = 14\text{mm}$, then $\pm$ one stone would give a resolution of approximately 6%. Thus the range of 5-10% indicates no significant difference in the performance of each section.

Table 1 Example Wave Climate 4

Segment	H_s (m)	T_p (s)	Duration (min)
1	.12	1.2	60
2	.14	1.4	70
3	.16	1.6	80
4	.18	1.8	90
5	.20	2.0	100
6	.18	1.8	90
7	.16	1.6	80
8	.14	1.4	70

Correct interpretation of the data can only be carried out by analysis those data in which the berm structure were stable (in this case, a stable structure was one in which berm erosion did not progress landward of the upper 1:3 slope). Figure 5 provides an illustration of the relationship between flume width and B/H_{peak} , where B is the starting berm width (equal to the total amount of recession) and H_{peak} is the significant wave height at the peak of the design storm. In general, B/H_{peak} increases with increasing flume width (thus indicating more reshaping for all wave climates).

Figure 6 shows the influence of groupiness factor on relative shaping (B/H_{peak}). For low values of groupiness, no significant difference in reshaping exists (as a function of flume width). As the groupiness factor is increased then the flume width becomes more important. The most significant difference in reshaping occurs at the highest value of groupiness. Values of B/H_{peak} for flume widths of 1.2 and 2.4 m are relatively constant whereas B/H_{peak} shows an increase with increasing groupiness for the 4 m flume results. These findings are in agreement with Hall et al (1989) and leave questions regarding the validity of two dimensional tests to evaluate the stability of berm or reshaping breakwaters.

Conclusion

A series of tests was undertaken utilizing various width "flumes" containing identical breakwaters simultaneously subjected to identical wave conditions. It was found that the extent of reshaping increased with increasing flume width. This effect was more pronounced when subjecting the structure to wave trains having a high groupiness factor.

This trend sheds doubt with respect to the validity of undertaking narrow flume two-dimensional tests of berms or reshaping breakwaters. At present, until further research can be undertaken, it is recommended that only fully three dimensional tests be undertaken when designing berm breakwaters, (even when incident wave conditions are parallel to the breakwater crest).

References

Burcharth, H.F. and Frigaard, P. "Reshaping Breakwaters, On the Stability of Roundheads and Truck Erosion in Oblique Waves", Proceedings of workshop of Unconventional Rubble-mound Breakwaters, Ottawa, 1987, pp. 55-72.

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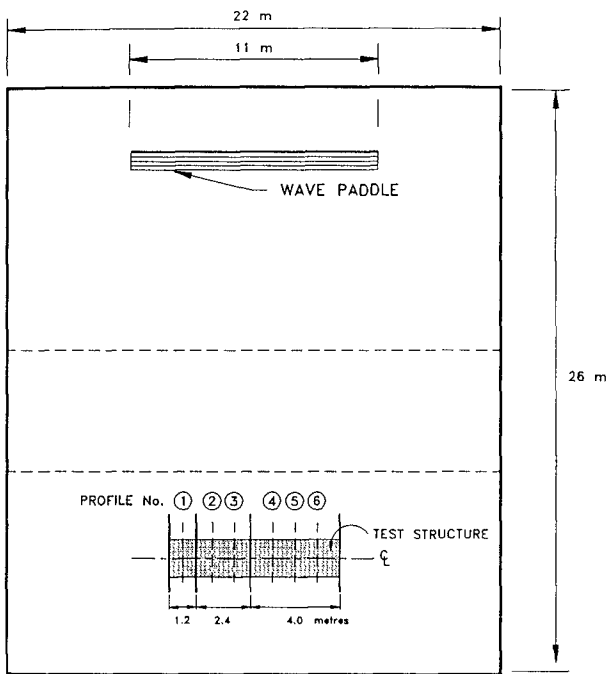


FIGURE 1 BASIN LAYOUT SHOWING FLUME LOCATIONS

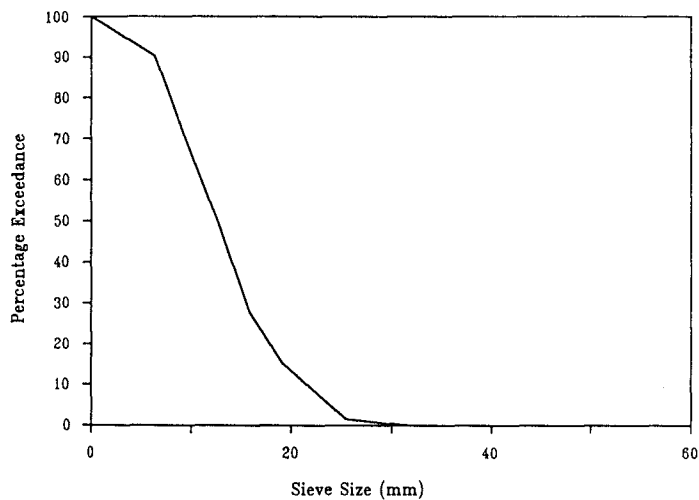


FIGURE 2 CORE MATERIAL GRADING

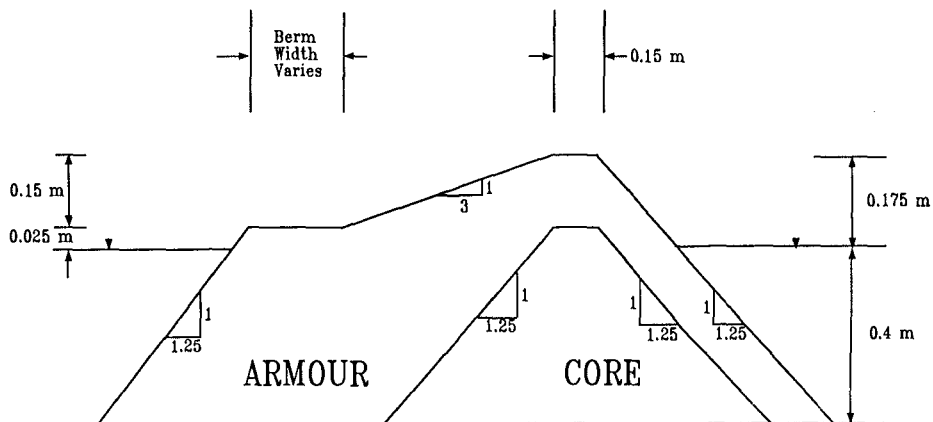


FIGURE 3 DIMENSIONS OF TEST STRUCTURE

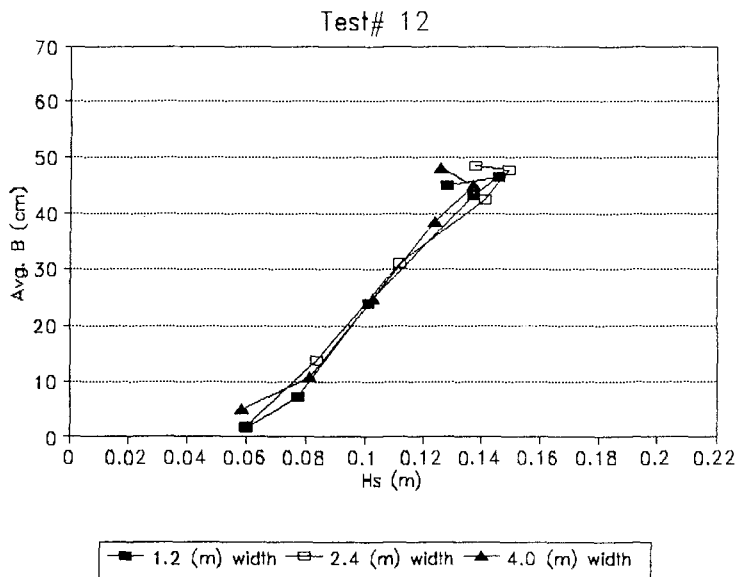


FIGURE 4 AVERAGE EROSION "B" RECORD DURING DESIGN STORM

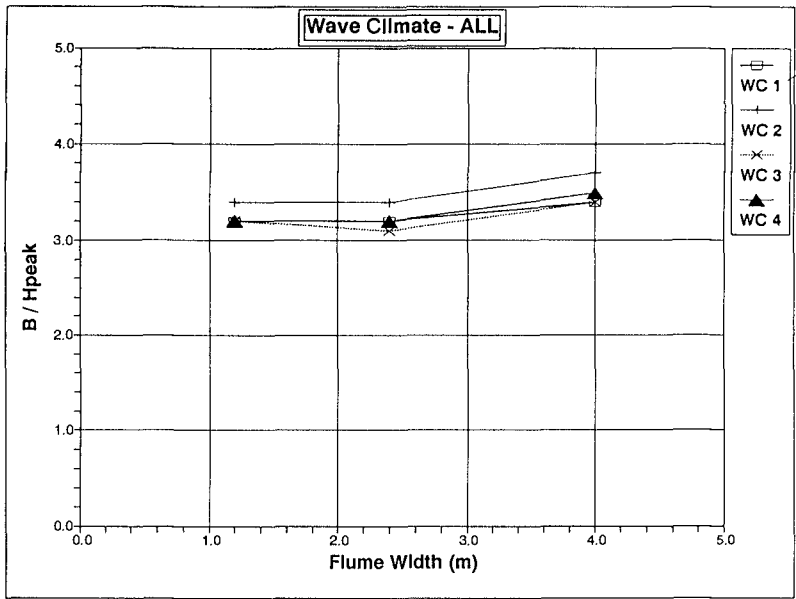


FIGURE 5 VARIATION OF TOTAL EROSION AS A FUNCTION OF WAVE CLIMATE

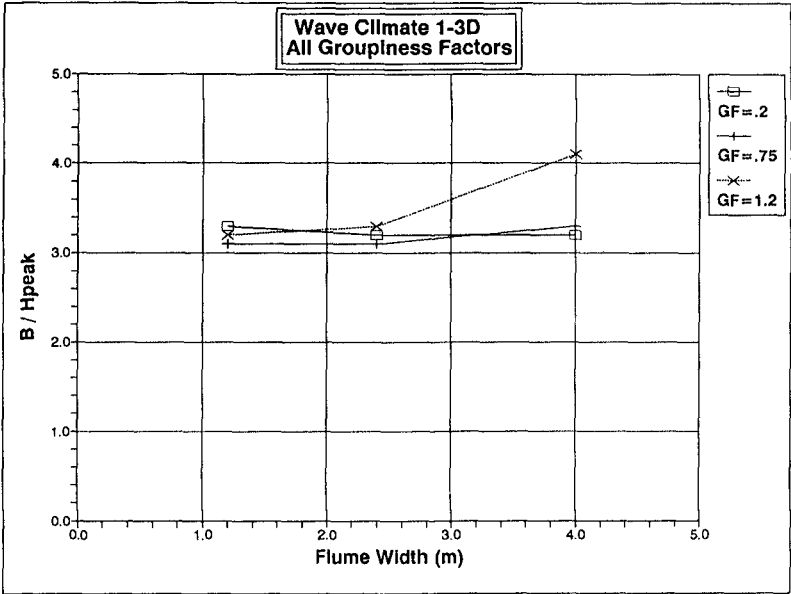


FIGURE 6 VARIATION OF TOTAL EROSION AS A FUNCTION OF GROUPINESS