HYDRODYNAMIC STABILITY OF THE BREAKWATER EXTENSION AT ÅRVIKSAND, NORWAY

Alfonse M. Dubi
University of Dar es Salaam
Institute of Marine Sciences
P.O. Box 668
Zanzibar
TANZANIA

1. ABSTRACT

The fishing port of Årviksand in Norway, which is situated on an open coast, is artificially harboured by breakwaters. The northern and southern breakwaters which were built in 1930-ies and 1950-ies respectively have been on several occasions damaged by heavy storms. This resulted in the need for further protection by extending the northern breakwater. This paper describes one of the hydrodynamic tests on the breakwater extension. The breakwater, with a berm breakwater concept design, is subjected to the joint probability of sea level elevation and wave heights (in other words, "fatigue" test). It is found that cover blocks of only 6 tons of weight each would be needed for the design of a beam breakwater, whereas cover blocks of 25 tons of weight each would be required for the design of a conventional (rubble mound), breakwater.

2. INTRODUCTION

2.1. Historical Background

Årviksand is a fishing port located on the Arnöy Island in Northern Norway (fig. 1). The layout of the port is shown in figure 2. This open-coast port is located close to fishing grounds and historically fishermen have landed their boats on the beach. 240 meters of the existing northern breakwater was built in the 1930-ies. After a heavy storm in December, 1936 the newest part of the breakwater was damaged and had to be rebuilt. During the year 1939 it was finished. In the 1950-ies this breakwater was extended from 240 meters to 480 meters and a new southern breakwater of 330 meters was built. The breakwaters were completed in 1962.

Figure 1. Location map

In January 1964 there was another heavy storm at Årviksand which damaged severely both parts of the breakwater. The breakwater was repaired in 1969 and by 1974 the harbour had been dredged to the necessary depth.

The tidal range at Årviksand is approximately 3.0 meters. The astronomical components are the most significant, but storm surges contribute up to 0.6 meters. The maximum observed water elevation at Tromsö, the nearest location with a permanent tide gauge, is 3.6 meters above the chart datum.

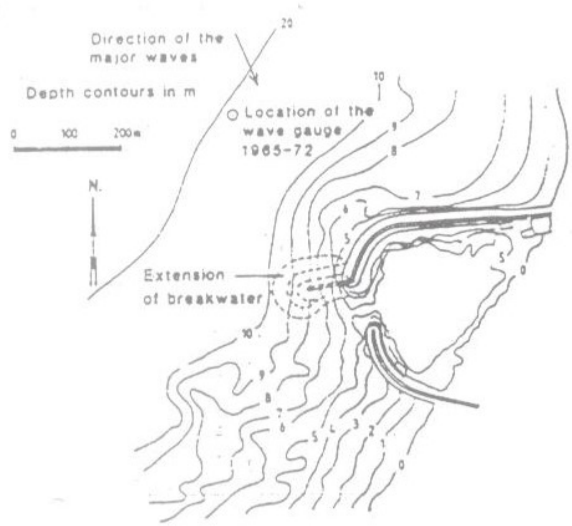

Figure 2. Årviksand fishing port

2.2. Wave Climate

The Norwegian Coast Directorate started in the 1960-ies long term wave measurement programs at four locations on the Norwegian coast, one of which was Årviksand. A pressure type wave gauge was operated in the period 1965-72. The location of the gauge is in approximately 19 meters of water depth as shown in figure 2. The waves were recorded on paper and the daily maxima were used for analysis. This set of daily maxima data has been fitted to a Weibull statistical distribution by the method of moments procedure. The Weibull parameters were obtained as γ = 0.777, location = 0.017 meters and scaling = 0.66 meters.
The estimated wave heights based on measurements (1965 - 72), hindcast data (1955 - 85) and refraction analysis are shown in the following table:

Table 1. Significant wave height at Årviksand

Return period (Years)	Hs based on measurements (1965 - 72) (m)	Hs based on hindcast (1955-85) and refraction analysis (m)
100	7.2	6.4
50	6.4	6.0
25	6.0	5.7
5	4.7	-
1	3.4	-

By a Monte Carlo procedure the daily maxima for seven years of observation have been simulated hundred times. Figure 3 shows the results of the spread of the 50 year significant wave height obtained in this way.

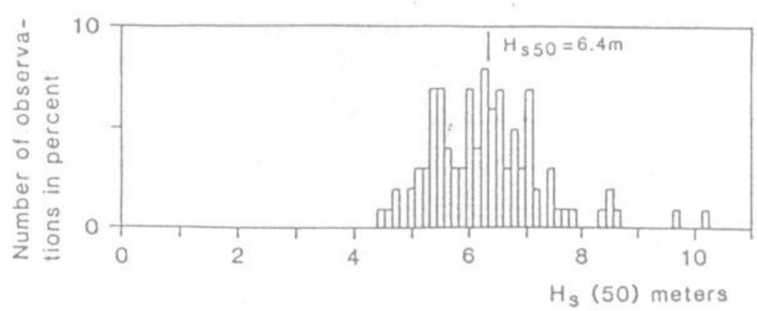

Figure 3. Monte Carlo simulation Årviksand

A similar study was made for the hindcast data from Tromsöflaket. Figure 4 shows the spread in the 100 year significant wave height for 30 years of data collection.

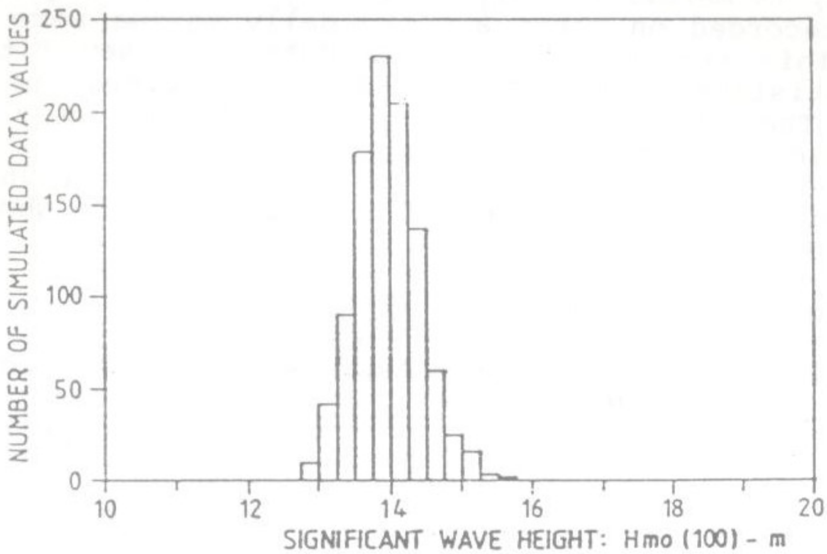

Figure 4. Monte Carlo simulation Tromsöflaket

There are statistical uncertainties related to hindcasting and measurement methods. In addition, there are uncertainties as to whether the period of measurements or hindcasting is climatologically a representative period. There are also uncertainties in the refraction analysis. These uncertainties should therefore be reflected in the design of breakwaters. In this paper, a test program designed to take into account of almost all of the uncertainties for the hydrodynamic stability of the breakwater extension is described. The results are presented for discussion.

3. MATERIALS AND METHOD

3.1. Previous test programs

The first step [4] in the investigation of the wave disturbance was to run a numerical model for 30,60 and 120m extensions of the northern breakwater. Then a physical model (scale 1:60) test was run to check the numerical calculations and to carry out stability tests on the breakwater in the same model. The tests were carried out with peak periods of 12, 14 and 16 seconds and with significant wave heights of approximately 2.0 meters and 3.0 meters. All these tests were done for a conventional rubble mound breakwater design with one layer of cover stones. Later it was decided to investigate a berm breakwater design and some additional wave disturbance tests were done. Some introductory two-dimensional flume tests on a berm breakwater showed that the berm breakwater concept with berm block weights of W_{50} equal to 3-4 tons was technically feasible. Tests on a three-dimensional model were carried out with irregular waves. The models layout is shown in figure 5.

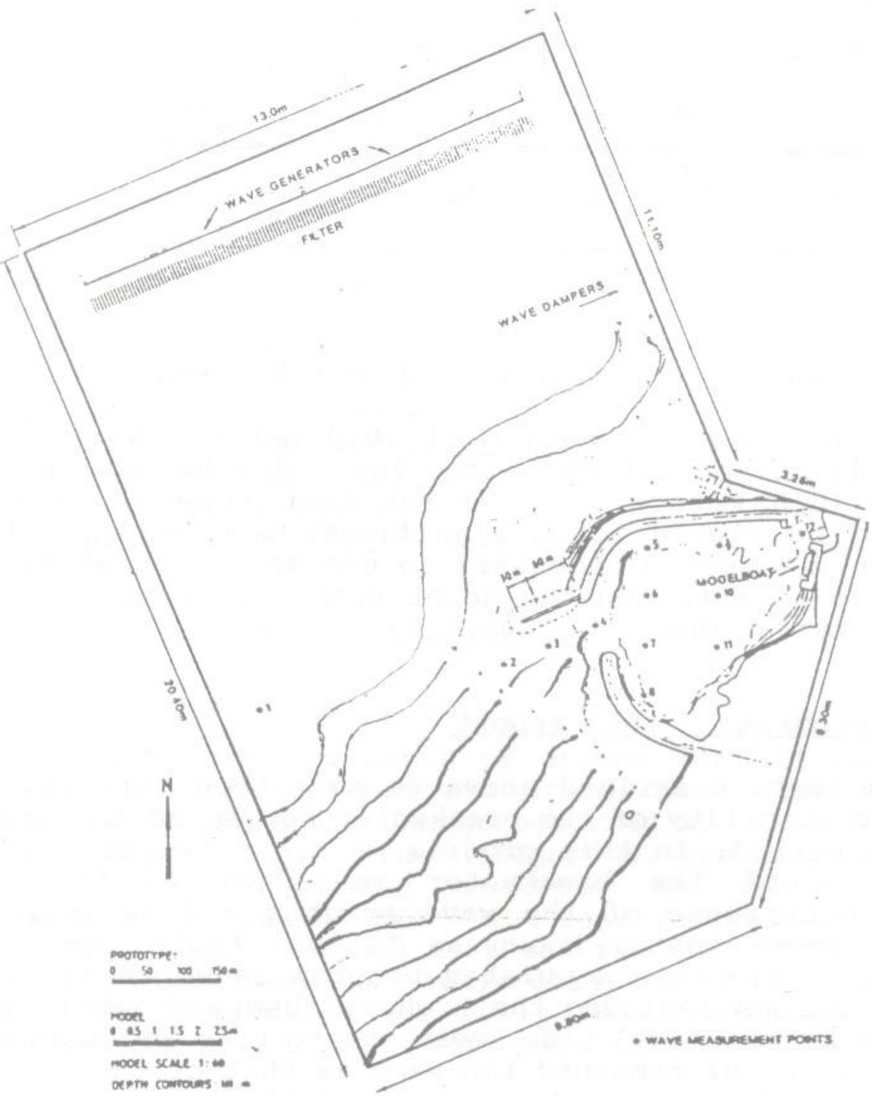

Figure 5. Model layout

Waves were measured at several locations numbered 1 to 12. Location 1 corresponds to the location of the wave gauge the period 1965-72. The breakwater design for the first tests is shown in figure 6.

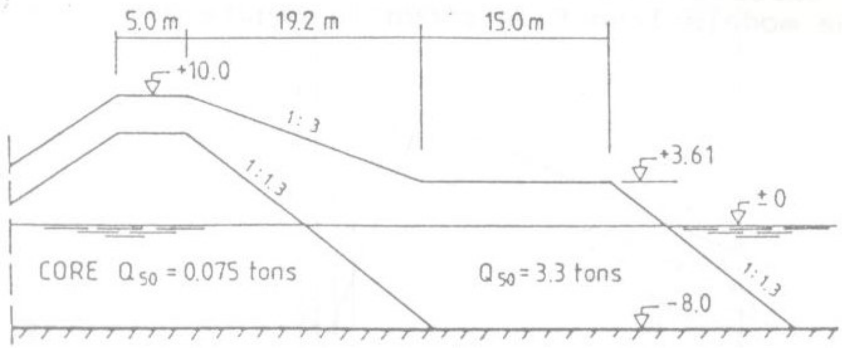

Figure 6. **Cross section of first tested berm breakwater**

This breakwater had a crest height of +9.0 meters which in due course of testing turned out to be too low. The height was then increased to + 10.0 meters. The first test showed that the breakwater would be stable for a significant wave height of 6.5 meters. However, it seemed necessary to expose the breakwater to the expected wave climate and the joint probability of the waves and water level variations.

3.2. <u>The test program in this report</u>

In the previous tests described above we have been interested in the hydrodynamic stability of the breakwater subjected to variable wave height and period. In this program we are interested in the dynamic stability of the breakwater subjected to the joint probability of occurrence of the wave heights and variation of water levels. These tests, known as fatigue tests, have been carried out with a computer simulated program shown in figure 7. Water level variations included the highest observed water level, highest observed astronomical tide, mean spring tide and mean water level. The model layout remained the same as that of the previous tests. The breakwater was subjected to variable water levels ranging between 3.6 meters and 0.5meters; and significant wave heights ranging between 4.5 meters and 7 meters for a duration of approximately 120 hours real time.

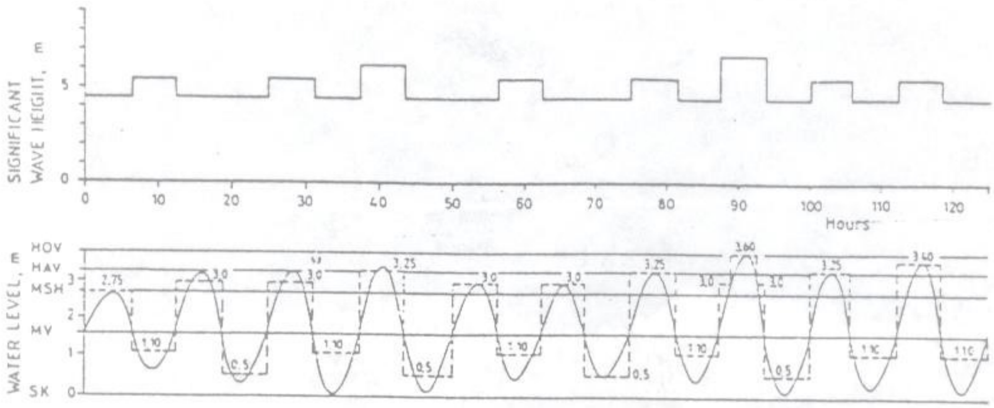

Figure 7. Test program for fatigue test

Water Levels Legend for figure 7

HOV = Highest Observed Water Level
HAV = Highest observed Water Astronomical Tide
MSN = Mean Spring Tide
MV = Mean Water Level
SK = Hydrographic Dept. O-point

4. RESULTS AND DISCUSSION

4.1 Test Results.

There have been movements of stones for all wave conditions.The breakwater was reshaped and stones were also moving over crest. A catastrophic damage occurred at the 117th hour, prototype time. A portion of trunk (figure 8) was washed away down to the still water level. The breakwater was rebuilt this time using average stone weight W_{50} = 4.4 tons instead of the previous W_{50} = 3.3 tons and the second test was run using the same test program. Unacceptable damage was again observed at the 98th hour, prototype time. The breakwater was built again using W_{50} = 6 tons and the test rerun. At the end there was no noticeable damage.

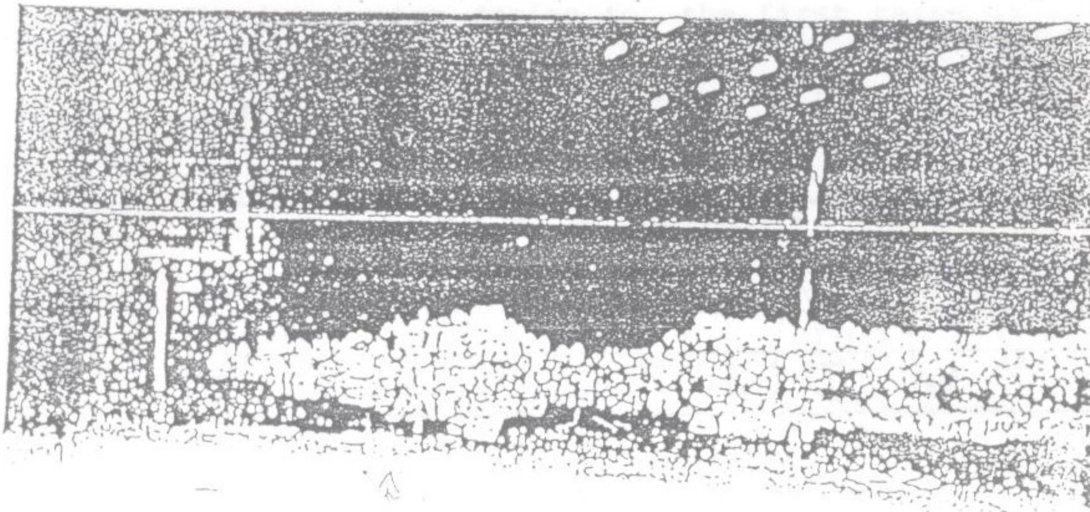

Figure 8. Damaged breakwater. First fatigue test

4.2. Discussion

As previously mentioned the measured wave data were fitted to a Weibull distribution. The return periods of waves of given heights are given by

$$R = \gamma/(1-F(H) \qquad (1)$$

where $F(H) = 1-\exp[-((H - H_0)/H_c)^{\gamma}]$ is the cumulative Weibull distribution function in which H_0, H_c and γ are Weibull parameters dependent on location, scaling and form factor. γ is the time interval between observations which was 1440 minutes (1day) in our case. For Årviksand [4] the parameters were found to be H_0 = 0.0017m, H_c = 0.660m and γ= 0.777. Since H_0 is too small to have any meaningful impact on the cumulative distribution, a two parameter Weibull distribution has been applied i.e.

$$F(H) = 1 - \exp[-(H/H_c)^{\gamma}] \qquad (2)$$

There have been no statistics available for Årviksand. However, the duration statistics have been given for Tromsöflaket (the nearest station with a permanent gauge) where the parametric persistence statistical model developed by Kuvashina and Hogben [3] has been used. This information has been applied for Årviksand.

For a given significant wave height H_s, the average duration of waves with a height H_s or larger is given by

$$\gamma_1 = [1 - F(H_s)].\ T/N \qquad (3)$$

and the average duration of non-exceedance is

$$\gamma_2 = F(H_s).\ T/N \tag{4}$$

where T is the time span considered and N is the average number of occurrences which exceed Hs during the time span T. For the waves at Tromsöflaket [2, 4] an expression for the duration of exceedance γ_1 and non-exceedance, γ_2 is given by

$$\gamma_1 = A/\ln\ [1 - F(H_s)]^{\beta} \tag{5}$$

$$\gamma_2 = \gamma_1\ F(H_s)/[1-F(H_s)] \tag{6}$$

where A and ß are empirical coefficients depending on the location which were found to have the following relationship:

$$A = 40\gamma^{-0.9} \tag{7}$$

$$\beta = 0.16\gamma^{-1.7} + 0.69 \tag{8}$$

where $\gamma = 1.29$, $H_0 = 0.70$m and $H_c = 1.87$. This gives then the average duration of storms at Tromsöflaket thus:

$$\gamma_1 = 31.8\ [(H_s - 0.7)/1.87]^{1.02} \quad \approx 60.2/(H_s - 0.7) \tag{9}$$

Further studies revealed that the largest waves arrive at Årviksand from NW - N direction and the refraction coefficients from this direction are approximately 0.5. Assuming this coefficient and using equation (9) we can estimate the average duration of the waves shoaling towards Årviksand as shown in table 2.

Table 2. Average duration of waves at Årviksand

Tromsöflaket H_s (m)	Årviksand H_s (m)	Average duration (hours)
8	4	8.2
10	5	6.4
12	6	5.3
14	7	4.5

Basing on Weibull cumulative distribution function for the daily maxima at Årviksand and using equation (1) we obtain the following values of return periods:

Table 3. Return periods at Årviksand

H_{max} (m)	H_s (m)	$1-F(H_s)$	Return periods (years)	number of events (50 years)	number of hours (50 years)
7.6	4	0.00127	2.63	19.0	114
9.5	5	0.00040	6.85	7.3	43.8
11.4	6	0.00011	25.5	2	12
12.35	6.5	0.000597	45.93	1	6.0
13.3	7	–	82.64	–	–

The average durations of the design wave heights at Årviksand have been approximated to be 6 hours when computing the number of hours through a 50 year lifetime of the breakwater. Hence the following durations were suggested for the test program.

Table 4. Average durations for the test program

H_s (m)	Duration (Prototype) (hours)	Duration(model) (hours)
4.6	82	10.6
5.0	33	4.4
6.0	6	0.7
6.5	6	0.7
Total	127	16.4

The joint probability of the wave heights and water levels can be found using one theoretical/mathematical framework described by Alcock and Carter [1]. However, available data for Årviksand were not analyzed in detail and hence the combinations were based mostly on subjective judgement.

5. CONCLUSION

The " fatigue" tests reported in this paper were among the last hydrodynamic stability tests on the breakwater extension at Årviksand. Emphasis has been put on the uncertainties in evaluating the design waves for the breakwater. Results of the study show that there is virtually no benefit in extending the breakwater beyond 90 meters. If the ruble mound breakwater is extended 90 meters, it is necessary to have cover blocks of approximately 25 tons of weight each; whereas the berm breakwater design gives W_{50} = 6 tons only. The berm breakwater concept has proven to be technically and economically feasible.

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NOMENCLATURE

A, β = empirical coefficients depending on location

$F(H)$ = cumulative Weibull distribution function

H = wave height

H_0, H_c and γ are Weibull parameters which depend on location, scaling and form factor

H_s = Significant wave height

N = average number of occurrences which exceed H_s

R = return period

T = time span

W_{50} = Average stone weight

Υ_1 = average duration of exceedance

Υ_2 = average duration of non-exceedance

REFERENCES

1. Alcock, G.A. and Carter. D.J.T., "Extreme events in wave and water levels." Proceedings of the Institution of Civil Engineering Conference on Development in Breakwaters, 2 - 4 October, 1985. Thomas Telford Publishers, London, 1986.

2. Anonymous. "Environment Conditions on the Norwegian Continental shelf- Barents Sea". NHL-report, STF 60 A 89052, 1989.

3. Kuvashina, S. and Hogben, N., "The estimation of wave height and wind speed persistence from cumulative probability distribution." Coastal Engineering Journal, vol. 9 pp 563 - 590,1986.

4. Törum, A., Næss, S., Instanes, A. and Vold, S., "On berm breakwater proceedings of the 21st International Conference on Coastal Engineering, ASCE, Spain, June, 1988.