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Model Simulation of Damage to Rosslyn Bay Breakwater during Cyclone "David"

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SUMMARY The breakwater at Rosslyn Bay, Queensland, was severely damaged by Cyclone "David". Model tests were undertaken to simulate the damage caused to the breakwater under the prevailing wave and surge conditions. Model results are compared with what happened in the prototype.

1 INTRODUCTION

Model studies have been used over many years to assist in the design of the armour of rubble mound breakwaters. Such models are invariably designed in accordance with the Froude model law, which implies constant lift and drag coefficients over the range of Reynolds number applicable to the model and the prototype.

As a result of extensive model testing several empirical equations have been developed relating armour weight to wave and breakwater characteristics. The equation in most common use by engineers is that commonly referred to as Hudson's equation

$$M = \frac{\rho H^3}{K_D(\Delta - 1)^3 \cot\alpha} \qquad (1)$$

where M is the mass of armour unit, ρ the density of armour, Δ the relative density of rock, α the face slope of the breakwater and K_D an empirical coefficient commonly referred to as the damage coefficient and is a function of armour shape, geometric arrangement, number of layers, amount of damage, storm duration and other factors.

For rock armour values of the damage coefficient as a function of per cent damage has been determined for non-breaking waves by CERC (1973) and for critical breaking waves by Foster and Gordon (1973) as shown in Table I.

TABLE I

K_D AS A FUNCTION OF COVER LAYER DAMAGE ROUGH QUARRY STONE, 2 LAYERS, RANDOM PLACED

Breaking Conditions	Percentage Damage			
	0 to 5	5 to 10	10 to 15	15 to 20
Non breaking	4.0	4.9	6.6	8.0
Breaking at Toe	2.0	3.3	4.3	5.0

Values of the damage coefficient given in Table I are based on model tests using monochromatic waves of constant height H and period T. Experience has demonstrated the benefit of model tests but there has always remained some doubt as to whether the damage produced in a laboratory flume using regular waves is truly representative of that produced by the wide spectrum of wave heights and periods to which the prototype is exposed. Even if this can be answered other questions remain as to what prototype wave characteristics are the regular waves related and what is the time scale of damage between the prototype and the model?

In recent years the development of the laboratory random wave generator has provided some answers to these questions. Tests in the laboratory comparing damage to a breakwater exposed to regular and irregular waves have been carried out at a number of research institutes (Refs. 1, 3, 6, 7). In general, these have indicated that the initiation of damage in a monochromatic wave flume is closely related to that which occurs with irregular waves when the height of the uniform waves is equated to the significant wave height. After damage has been initiated research would indicate that the rate at which damage occurs is faster and more irregular with monochromatic waves than with irregular waves because of the greater number of high energy waves to which the structure is exposed. These results are indeed fortunate as they give a good deal of confidence in the vast amount of empirical data on breakwater stability that has evolved from testing with monochromatic waves.

2 ROSSLYN BAY BREAKWATER

During the period 16th to 20th January 1976 the rubble mound rock breakwater at Rosslyn Bay, Queensland, was severely damaged by wave and surge action as a result of cyclone "David". The failure to the breakwater provides a unique opportunity to further compare model and prototype behaviour and in particular to look at the effect of overtopping and time scale of damage. The model tests were undertaken by the Water Research Laboratory of The University of New South Wales in association with the Department of Harbours and Marine, Queensland, and Blair, Bremner and Williams Pty. Ltd., Consulting Engineers, Queensland.

A typical design section through the Rosslyn Bay breakwater is shown in Figure 1.

MODEL MATERIAL USED

CLASS A	75%	120 to 140 grams
	20%	60 to 120 grams
	5%	20 to 60 grams
CLASS B	35%	20 to 40 grams
	35%	10 to 20 grams
	25%	2 to 10 grams
	5%	0·5 to 2 grams
FILTER	30%	20 to 60 grams
	55%	10 to 40 grams
	15	6 to 3mm
CLASS C	60%	6 to 3mm
	25%	3 to 1mm
	15%	< 1mm

PROTOTYPE MATERIAL

CLASS A	75%	3 ton
	20%	1½ to 3 ton
	5%	½ to 1½ ton
CLASS B	35%	½ to 1 ton
	35%	500 lbs to 1000 lbs.
	25%	100 lbs to 500 lbs.
	5%	15 lbs to 100 lbs.
FILTER	30%	½ to 1 ton
	55%	100 lbs to 1000 lbs.
	15%	Coarse Class C
CLASS C	60%	6" to 3"
	25%	3" to 1"
	15%	Minus 1"

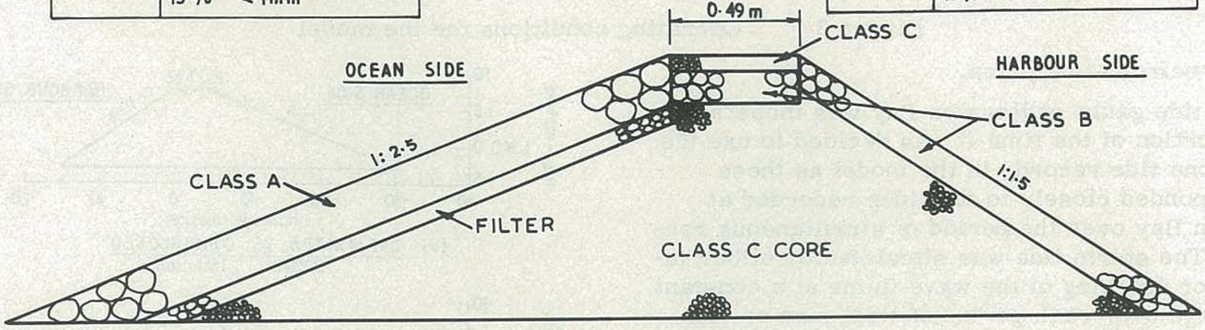

Figure 1 Typical design section

3 HISTORY OF FAILURE

The history of Rosslyn Bay breakwater, its damage during cyclone "David" and repair work carried out following the cyclone, has been reported by Bremner (1976). The following is an abstract from this paper.

'Cyclone "David" started to influence the area on 15.1.76. It moved steadily in a SSW direction for 5 days, building up heavy seas from Mackay to Byron Bay, (against a stationary high in the Tasman Sea).

Little or no damage occurred to breakwater until the high tide 0950 hours on 18.1.76 when there was evidence of overtopping of the breakwater. An inspection about 1130 hours on 19.1.76 revealed heavy overtopping of the breakwater, particularly in the pontoon area and from the fuel wharf to the end. The water level within the Harbour appeared to have risen to RL 6.0m at the time of high tide (1037). On this particular tide, waves were not actually breaking on the structure. Wave height was estimated at 2.5m with a maximum 3.5m, with a period of from 5 secs. to 6 secs.

The breakwater structure suffered minor damage up to midnight of 19th January. Major damage occurred from 11 pm on 19th to early hours of 20th January, coinciding with the predicted high tide.'

Excellent wave data during the cyclone were obtained by the Department of Harbours and Marine at Double Island Point, Yeppoon and Mackay using Datawell Wave Rider Buoys.

Tide records during the maximum storm surge were obtained at Rosslyn Bay and Gladstone approximately 110 km to the south. The maximum storm surge of 1.0 to 1.1m was recorded in the afternoon of 19th January. However, the maximum storm surge did not coincide with the higher tides. The maximum storm tide level of 5.1m (Gladstone gauge) was recorded during the morning of 19th January when unfortunately the Rosslyn Bay tide gauge was inoperative. High water levels of comparable magnitude were also recorded during the morning high tide on 18th January. Comparison of the high tide records at Gladstone and Rosslyn Bay over the simultaneous period of record indicate reasonably close correspondence with a maximum difference of +0.3m and a minimum difference of -0.1m. Based on these records it is probable that the maximum water level at Rosslyn Bay occurred during the morning high tide on 19th January with a level of between 5.3 and 5.4m.

The interrelationship between high waves and high water levels is shown in Figure 2. Assuming that damage was primarily caused by overtopping of the crest of the breakwater, Figure 2 would indicate that maximum damage would have been expected during the pm tides (approximately midday) on 19th January. However, the observations of Bremner (1976) indicate that whilst heavy overtopping was occurring at this time damage was not substantial. The major damage occurred during the pm high tides (approximately midnight) on 19th and continued during the high tides on 20th. It is probable that the high waves and tides during the morning of 19th considerably weakened the structure contributing to its eventual failure during the two following high tides.

4 THE MODEL

A linear scale for the model of 27.59 was selected from consideration of available model material. The model was constructed in accordance with the cross section shown in Figure 1. The unit weight of the material used in construction of the model was 2.65 and gradings are given in Figure 1.

Tests were undertaken in the Water Research Laboratory 1m wide wave flume. Waves were generated by a flap type paddle and water levels on either side of the breakwater were equalized by using a

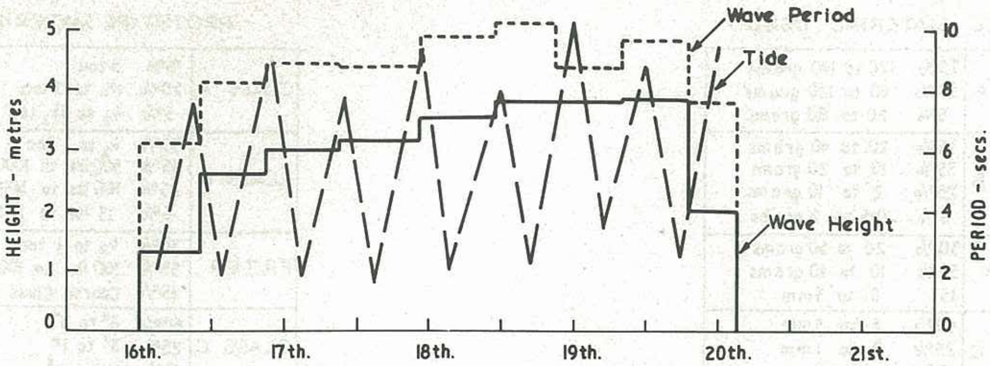

Figure 2 Operating conditions for the model

3 HP recirculating pump.

As the tide gauge at Rosslyn Bay was inoperative over portion of the time it was decided to use the Gladstone tide records in the model as these corresponded closely to the tides recorded at Rosslyn Bay over the period of simultaneous record. The storm tide was simulated by either infilling or draining of the wave flume at a constant discharge over each period of tidal rise or fall. This resulted in saw-tooth tides in which the tidal amplitude and tidal phase was reproduced in accordance with Froudian scaling but with some distortion of the tidal curve.

Wave conditions simulated in the model were the significant wave height and peak period of the spectrum. Wave data in the prototype were recorded at a sampling period of 12 hours and it was assumed in the model that these conditions remained stationary over a 12 hour period. The operating conditions for the model are shown in Figure 2.

These test conditions correspond to general practice in operation of monochromatic wave tests on rubble mound structures.

5 TEST RESULTS

The sequence of failure is shown in Figure 3.

The test began at 2230 hours on 16th (low tide) and was continued until 0700 hours on 20th (low tide), time being scaled according to the Froudian time scale. Initial damage to the breakwater occurred at 0900 hours on 17th when the crest was destroyed in a landward direction.

Damage continued during each period of high tide when the crest was overtopped. The main damage was the result of overtopping and only a small proportion of the armour was displaced seaward.

At the completion of the test series the structure was subjected to continuous wave attack under stationary conditions corresponding to a wave height of 3.8m and a storm tide level of 5.1m. Little additional damage occurred indicating that near equilibrium damage had been reached.

Repeating the test gave almost identical results both as to incidence of damage and the final profile.

6 DISCUSSION AND CONCLUSIONS

A comparison between the model and the prototype damage is given in Table II.

The mechanics of damage in the model and prototype was the same with the majority of damage occurring by rock being displaced from the crest and settling on the harbour side as a result of wave overtopping of the crest. However, damage in the model occurred much earlier than the prototype (Froudian scaling). This is not surprising as the prototype armour would be expected to be more strongly interlocked as a result of wave action and settlement over the period since it was constructed. Indeed as shown in Table II the breakwater suffered no major damage between 17th and 19th even though wave and surge conditions were comparable and at times more severe than when it did suffer major damage on 20th (see Figure 2).

The final profile reached on the model was in equilibrium with the wave and surge conditions and showed very close agreement with the prototype.

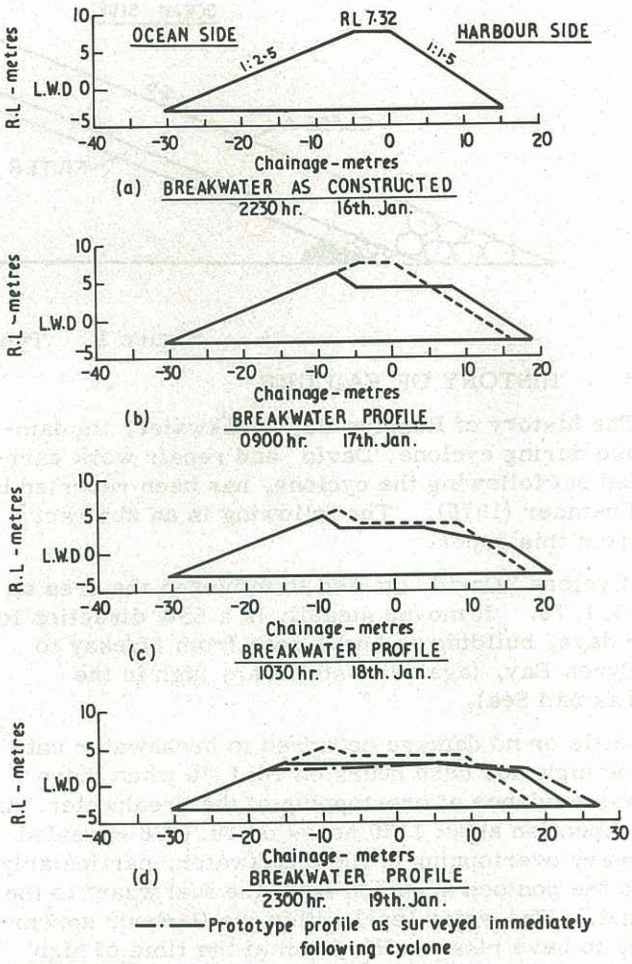

Figure 3 Sequence of Failure

TABLE II

COMPARISON OF MODEL AND PROTOTYPE DAMAGE

Date Jan. 1976	Ptoto-type Time (hrs.)	Prototype	Model
16th	2230	No damage	Fines removed from crest
17th	0900	" "	Major damage - crest lowered from RL 7.5 to RL 4m
17th	2100	" "	Continued damage
18th	0950	Slight "	Continued damage
18th	1630	" "	Continued damage, crest at RL +3.5m
19th	1130	Heavy overtopping, slight damage	Continued damage, crest at RL+2.5m
19th	2400	Minor damage	Stable profile
20th	Early hrs.	Major "	" "

However, it cannot be stated with certainty if the prototype was in equilibrium or whether further damage may have resulted if the storm had continued. Damage in the model continued over five periods of high tide before reaching a stable profile. The prototype damage occurred predominantly over one high tide period.

This would indicate that the rate at which damage occurred in the model was significantly lower than the prototype even though the final profiles showed close agreement. This is opposite to what has been experienced in model tests of non overtopped structures using random waves (Refs. 1, 3, 7). It is possible that the larger waves in the spectra play a much more significant role in the stability of overtopped structures. Further tests are planned to investigate this possibility.

Most of the damage was to the crest of the structure. Damage to the seaward face was slight (less than 5 percent). Using the recorded wave conditions it is of interest to compare this damage with that indicated in Table I. If the densities of rock and seawater are assumed to be 2650 and 1025kgm^{-3} respectively, then from equation(1)

$$K_D = 266 \frac{H^3}{M}$$

Assuming a maximum wave height at the structure of 3.84m (as measured offshore) then the damage coefficients for M_{75} = 3 tonne and M_{50} = 2.5 tonne are 5.0 and 6.0 respectively. Water depths at the toe varied during the tests from 4.0 to 8.0m whilst waves varied between non-breaking and breaking. For these damage coefficients Table I would tend to indicate somewhat higher levels of damage than were observed in either the model or the prototype. This requires further study. For the prototype increased strength with time is probable as a result of settlement and compaction under wave action. The differences in the model are not so readily explained. Two effects may be contributing to the apparent increased stability. Firstly, as a result of overtopping downrush and uplift forces may be reduced. Secondly, tests on which the results given in Table I are based were conducted without tide. It is possible that the continuing changing location of maximum wave forces as a result of tide variation reduces the level of damage on the face of the structure.

In conclusion, the tests clearly indicate that an overtopped breakwater is much more susceptible to damage than one which is not overtopped. In regions of high storm surge (which is of particular relevance to the northern part of Australia) to achieve this may in many cases be uneconomical. Further research is needed to provide design criteria for the crest of overtopped structures of similar standard to that available from many years of testing of non-overtopped structures.

7 ACKNOWLEDGEMENTS

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