The Design Concept of Dual Breakwaters and its Application to Townsville, Australia.

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1. Introduction

In Central and North Queensland the coastline is protected by the Great Barrier Reef over a length of some 1900km from Gladstone in the south to the tip of Cape York (Fig. 1).

The fetch distance from the reef to the coastline is very variable from about 15km to 140km. Hence the coastal areas north of Gladstone have moderate to low wave climates except during abnormal weather events such as cyclones or long time interval bands of strong winds. In these events larger storm waves of significant wave heights of 10m may be superimposed, in the case of cyclones, on storm surges several metres in height.

The design of breakwaters and shoreline structures for protection against all except cyclonic and strong wind band effects requires, in the main, readily available sizes of armour rock and relatively low crest elevations.

To offer similar protection against cyclonic weather events often requires the use of artificial armour units of concrete and a substantial increase in crest heights. The resulting increase in costs makes many of the small projects uneconomic.

In this paper the concept of using an offshore breakwater which is designed to fail under extreme wave conditions to protect an inner breakwater or revetment is examined and the results applied to Townsville Harbour where cost savings of the order of 40 percent were achieved over a conventional design.

2. Rosslyn Bay Breakwater

The design concept arose from studies and observations of the damage to the Rosslyn Bay breakwater during cyclone David (see Fig. 2) and the subsequent protection provided by the breakwater following damage (Foster, Bremner and McGrath, 1978). Features of this damage were:-

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(3) Principal, C.A.Miller & Associates, Associate Consultant to WRL
(4) Principal, B.C.Wallace & Associates, Associate Consultant to WRL

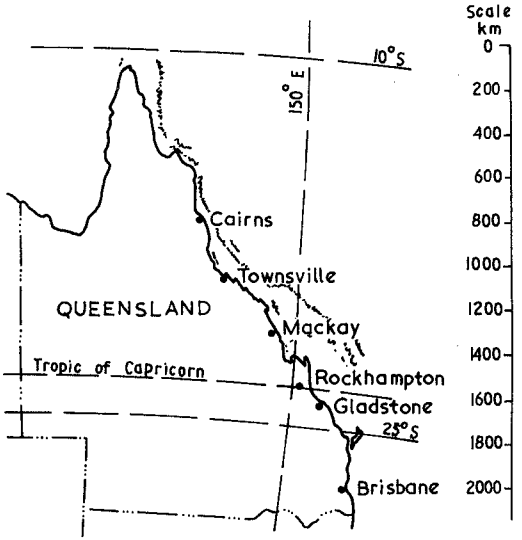


FIGURE 1: LOCALITY SKETCH

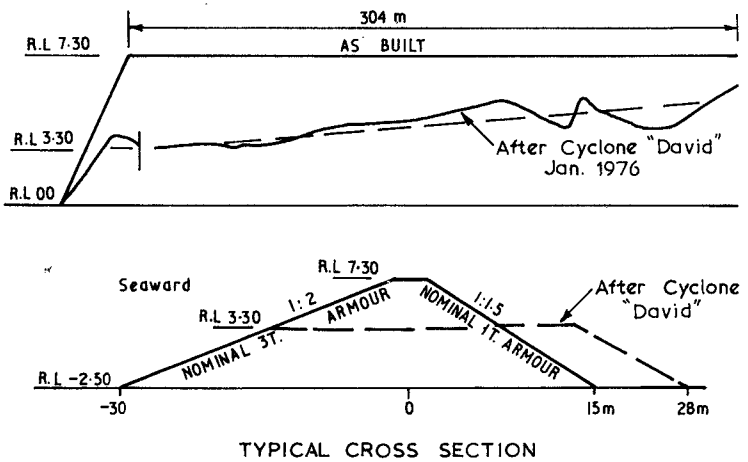


FIGURE 2: DAMAGE TO ROSSLYN BAY BREAKWATER BY CYCLONE DAVID

(i) Failure occurred in a controlled manner with material being displaced from the crest and deposited on the leeward slope, forming a widened and lowered profile. The breakwater was composed of 3t nominal armour rock on the seaward face, ½ to 1 tonne filter rock and less than 150mm core material.

(ii) Failure to the breakwater was closely simulated by model studies.

(iii) Reconstruction of the breakwater was not commenced until 2½ years after failure. During this period the breakwater acted as a submerged breakwater and continued to provide substantial wave protection within the harbour. Over this period waves of up to 1.8m height were experienced. All small boats continued to stay on their piled and bottom moorings and there was no damage to boats, moorings or shoreline revetments.

These observations triggered the idea that if a controlled submerged breakwater could successfully be built then a significant reduction in costs may well be possible. The interesting design problem was then posed of building a breakwater of sufficient initial stability to allow safe construction but when subjected to the designed forces, due to carefully chosen extreme and fairly rare weather events that result from cyclones, it would be reshaped in a controlled and predictable manner to become a stable and partially submerged structure and retain its wave attenuation ability.

(It should be noted that in Australia large floating plant is not available for breakwater construction and all breakwaters are constructed in the dry. This precludes the construction of an initially stable offshore submerged breakwater).

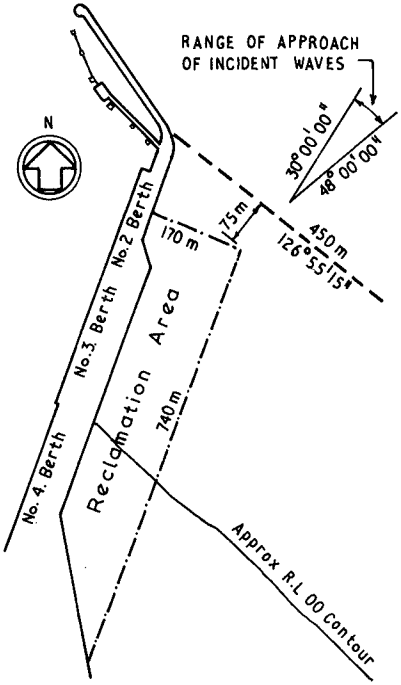
It is of interest to note that the repair to the Rosslyn Bay breakwater as reported by Bremner, McGrath and Foster (1978),which used a combination of rock and modified cubes,has been completed and in 1980 was subjected to waves of 3.2m in height during cyclone Simon. There was no damage to the breakwater.

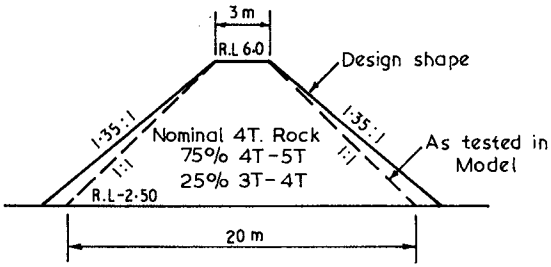
3. Townsville Harbour

3.1 General

To provide additional port facilities it is proposed to reclaim a significant area on the eastern side of the harbour as shown in Figure 3. Wave protection to this reclamation was to be provided by an offshore breakwater or by conventional armouring of the seaward face of the revetment. Initial design and economic assessment showed that the second proposal was approximately 60 percent more expensive and detailed designs were not undertaken. This cost saving resulted from:-

(i) use of locally available low cost rock in the offshore breakwater which was designed to "fail" under extreme wave and storm surge conditions and provide a partially submerged offshore breakwater;





TYPICAL CROSS SECTION
THROUGH OFFSHORE BREAKWATER

FIGURE 3: TOWNSVILLE HARBOUR

(ii) reduction in wave height at the revetment thereby reducing the height of reclamation required and the use of low cost readily available rock armour.

3.2 Design Conditions

Design wave and surge conditions have been extensively studied by Blain Bremner and Williams (1978), Harper and Stark (1977), Sobey, Rossow and McConagle (1978), and are summarised in Table 1 below:-

Table 1: Design Storm Conditions

Return Period (yrs)	Tide (MHW) (RLm)	Cyclonic Conditions			
		Storm Surge (m)	Design SWL (RLm)	H_s (m)	T_s (sec)
50	2.45	1.68	4.13	3.76	6.0
100	2.45	2.00	4.45	4.15	6.3
500	2.45	2.76	5.21	5.26	7.2

Tidal data based on Townsville Harbour Board datum of RL0.0 are as follows - HAT 3.6m, MHWS 2.9m, MHWN 2.0m, MSL 1.6m, MLWN 1.2m, MLWS 0.4m, LAT -0.2m.

3.3 Offshore Breakwater

A typical section through the proposed offshore breakwater is shown in Figure 3. It basically is composed of nominal 4t rock placed at its natural angle of repose with a range of rock sizes between 3 and 5t. Crest level at completion of construction is at RL 6.0 which will be overtopped only under extreme wave and surge conditions.

3.4 Model Tests

Extensive model tests of the scheme were undertaken by the Water Research Laboratory of the University of N.S.W. (Foster, Miller and Wallace 1980). A summary is given in this paper and the reader is referred to the above reference for more complete details.

Preliminary stability tests were run at constant water levels for the design peak water levels and wave heights shown in Table 1. The tests were run for two nominal rock sizes (M) of 2t and 4t, having approximate gradings of 75% between M and 1.25 M and 25% between 0.5M and M. Test duration was 7 hours. (Prototype).

Collapse of the breakwater was ordered with some stone being displaced seaward, flattening the seaward slope and the remainder being displaced landwards resulting in a broadening and reduction in elevation of the crest which is similar to that observed during the failure of the Rosslyn Bay breakwater (Foster, Bremner and McGrath 1978). The majority of damage occurred in the first two hours and basically stopped when a water cushion formed over the crest. Because of the higher water levels damage was less for the less frequent events, the highest damage being associated with the 1 in 50 year event. All further tests

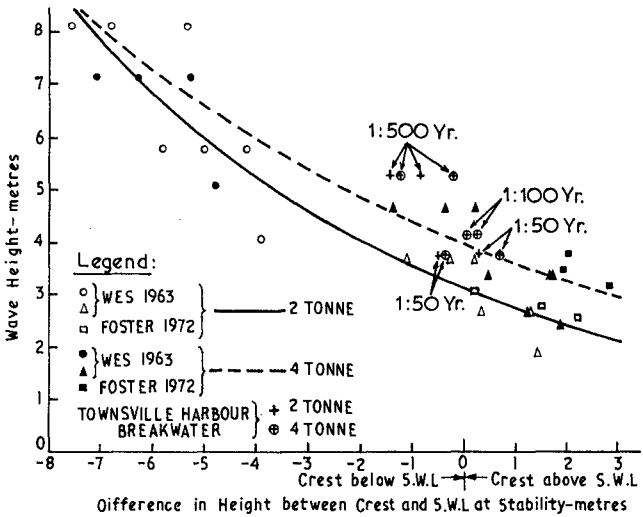


FIGURE 4: VARIATION OF CREST STABILITY WITH WAVE HEIGHT–CONSTANT WATER LEVEL

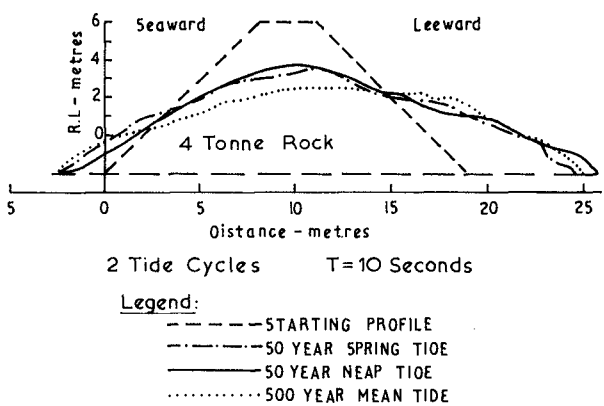


FIGURE 5: BREAKWATER DAMAGE

were undertaken with tide variation reproduced in the model so that the structure was subjected to the design wave under varying water levels.

The relation between crest level and S.W.L. at stability under constant water level is shown in Figure 4 together with results of similar studies undertaken by U.S. Army Corps of Engineers (1953) and Foster (1972) using Froudian scale factors to convert the results to equivalent 2 and 4t nominal rock. Figure 5 indicates that the results are within the range obtained in previous studies. However, the difference in damage for the 4t and 2t rock sizes is smaller than indicated by the lines of best fit (exponential). This may be due to the fact that the previous studies were carried out on conventional breakwaters having separate core, filter and primary armour layer.

In the second series of tests the breakwater was subjected to the design wave height and storm tide resulting from tide plus the design storm surge. Wave period was 10 seconds. Spring, neap and mean tidal cycles were tested. Typical results are shown in Figure 6. The mode of failure was similar to that for the constant water level tests but the degree of damage was substantially larger as the crest is exposed to higher wave forces as the tide changes.Damage was not strongly dependent upon the tidal cycle chosen.

During the tests maximum incident and transmitted waves were monitored and the results are shown in Figure 7. Wave breaking occurred over the crest and the transmitted wave was measured 65m landward of the structure after the wave had reformed.

Rather surprisingly these results indicate that the transmitted wave is not strongly dependent upon the amount of damage and implies a considerable degree of safety in the design. The reason for this appears to be, that the effect of increasing crest width as a result of failure tends to largely counterbalance the effect of increasing water depth relative to the crest. Similar results have been observed by Saville (1963) (see Appendix 1). The overtopping tests were undertaken in the laboratory's 1m wide monochromatic wave flume. The model scale was 1 to 27.4.

The three dimensional model tests to study the effects of diffraction around the head of the breakwater and wave heights along the revetment were undertaken at a scale of 1 to 57. In these tests the transmitted waves past the breakwater agreed closely with the larger scale tests which indicates that scale effects are small and that the major component of the transmitted wave results from wave overtopping.

The three dimensional tests undertaken to study diffraction indicated that during overtopping wave heights were not substantailly increased in the lee of the breakwater as a result of diffraction which enabled the initial design length of the breakwater to be substantially reduced in the final design.

4. Conclusion

Model tests have shown that the concept of constructing a break-

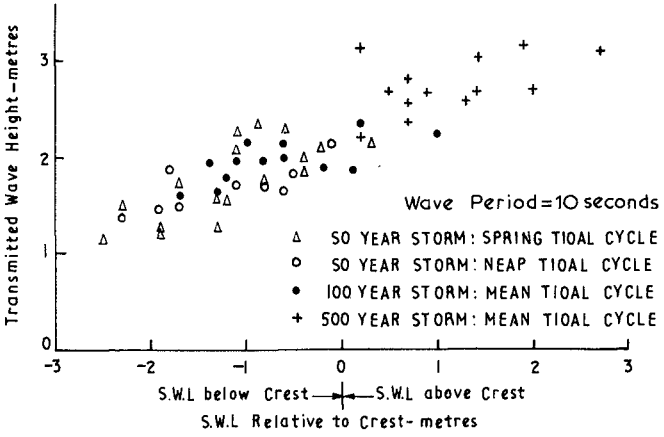


FIGURE 6: VARIATION OF TRANSMITTED WAVE HEIGHT WITH S.W.L RELATIVE TO CREST

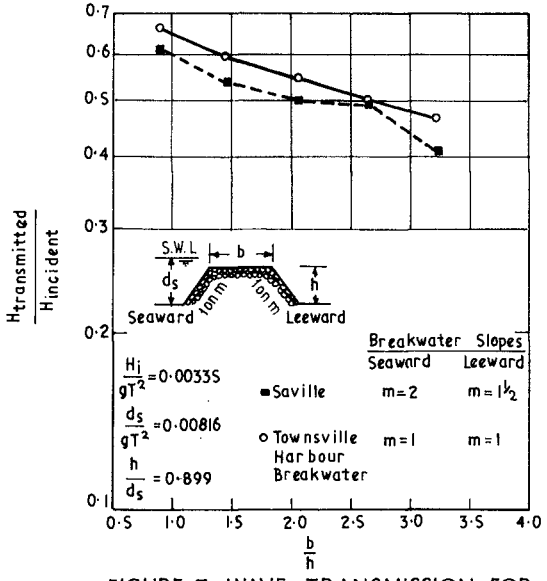


FIGURE 7: WAVE TRANSMISSION FOR VARYING CREST WIDTH

water to form a submerged structure after failure is an effective method of reducing wave heights at inshore structures. Tests indicate that the transmitted wave past the breakwater is relatively independent of incident wave conditions, rock size and depth over the crest. Although more damage occurs for the longer return period events and the smaller rock sizes the correspondingly wider crest tends to counterbalance the increased depth over the crest resulting in relatively small increases in the transmitted wave. Hence a considerable degree of safety is provided against the design conditions being exceeded.

In the view of the authors the important features of the design concept are:-

(i) The high degree of tolerance inherent in the design against the design conditions being exceeded.

(ii) The relative ease of construction which does not require large radius heavy cranes and leads to rapid construction and substantial cost benefits.

(iii) The size of rock is contained to economic winnable and handable natural rock sizes.

(iv) Wave run-up on the inshore structure is reduced, enabling lower crest levels to be adopted.

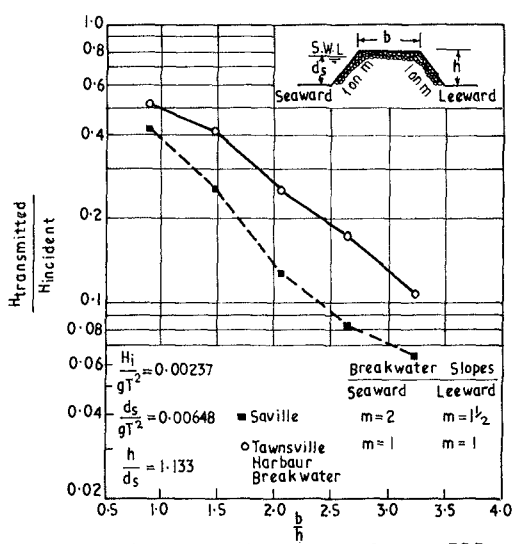


FIGURE 8: WAVE TRANSMISSION FOR VARYING CREST WIDTH

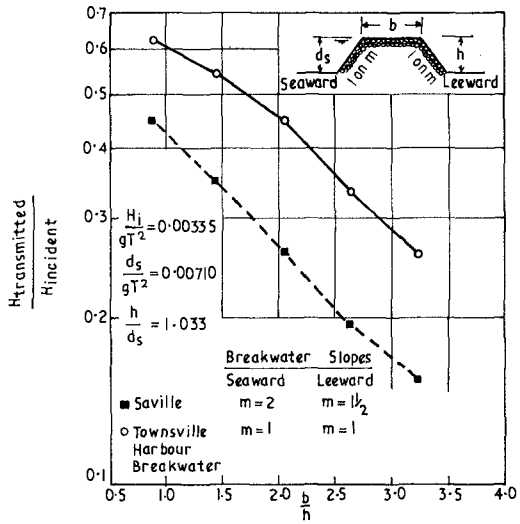


FIGURE 9: WAVE TRANSMISSION FOR VARYING CREST WIDTH

Appendix 1.

There are little data on wave transmission coefficients over permeable breakwaters of relatively narrow crest widths. Model tests for substantially broader crest widths are described by Saville (1973). These show that the transmitted wave is reduced with increasing width of crest. This fact has been used to explain why the present results indicate only a slow change in the transmission coefficient even though the breakwater crest is substantially lowered during failure.

In order to quantify the comparison with Saville's results supplementing tests were undertaken on the rock used in the model studies for the same dimensionless geometric and wave parameters. Results are shown in Figures 6 - 8. All tests follow similar trends to that observed by Saville. However, for S.W.L. below the crest transmission coefficients were found to be 20 to 40 percent higher. For S.W.L. above the crest the results agree closely. The differences are believed to result from the different rock sizes used in the tests. The results for SWLs below the crest tend to indicate that permeability and therefore wave energy transmission through the structure are higher for the submerged breakwater in the present studies. For SWLs above the crest, difference in permeability has only a marginal effect on transmitted wave since the majority of energy is transmitted by overtopping.

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