

## DESIGN OF AN OVERTOPPING BREAKWATER

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## 1. INTRODUCTION

The Maritime Services Board of New South Wales, Australia, is constructing a major new port facility on the northern foreshores of Botany Bay. A principal part of this project has been the construction of a large armoured revetment from the northern shores.

The entrance to Botany Bay faces south-east and it is from this direction that a large proportion of offshore wave energy arrives. Some of the wave energy which is directed onto the Bumborah Point revetment is reflected towards Yarra Bay on the northern shores of Botany Bay. Yarra Bay is largely undeveloped, but a sailing club has stood for many years on the beach at the southern end. As a consequence of this reflected wave energy being directed towards Yarra Bay, its wave climate has been changed considerably so that during the storms of May-June, 1974, Foster (6), damage was suffered by the club-house. Additionally the more severe wave climate and consequent steeper beach have made it much more difficult to launch sailing boats. The Maritime Services Board is charged with the responsibility to carry out remedial works where damage is caused by the port development. Figure 1 shows the revetment and sailing club site.

To assist in coastal engineering design aspects of the port development, a large fixed bed wave model of Botany Bay has been built to an undistorted scale of 1:120. This model, some aspects of which have been described by Lawson (4), has pneumatic wave generators which enable offshore wave directions between east-north-east and south to be generated with prototype periods in the range of 5 to 16 seconds. A pneumatic tide generator enables a sinusoidal tide to be generated.

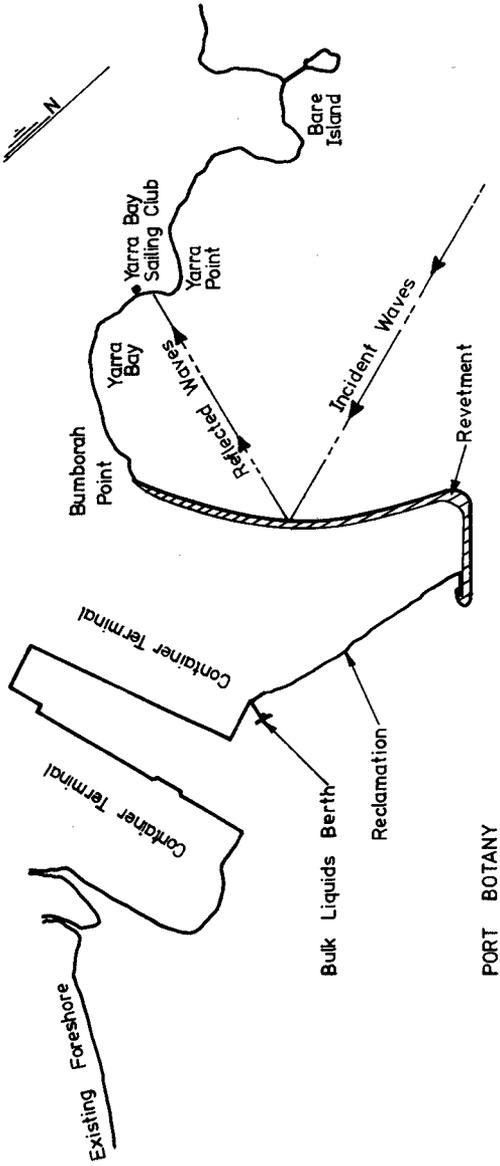


FIG. 1

PORT BOTANY

The effect of tide stage and velocity on wave propagation is important in Botany Bay because it is relatively shallow.

Wave-height exceedance statistics have been gathered for a number of years using Databell Waverider Buoys. The installation has been maintained for more than 8 years about 2 kilometres offshore of the entrance to Botany Bay and programmed gathering of data 4 times a day for 20 minutes carried out. Another buoy was installed in Yarra Bay for about 2 years.

The breakwater had to fulfill the following requirements :-

- (a) Return wave climate and run-up in front of the club house to pre-port-works condition;
- (b) Offer adequate rigging area on the beach;
- (c) Provide an adequate gap between its outer end and the shoreline to provide a 10 boat-length space, based on a 5 metre boat length;
- (d) Be no more visually obtrusive than necessary.

This last requirement meant that an over-topping breakwater would be desirable.

## 2. HYDRAULIC STUDIES

The hydraulic studies required for successful design of a shore structure must investigate the following aspects :-

- (a) optimisation of the performance of the breakwater taking due consideration of benefit-cost relationships.
- (b) provide design wave-height data based on probability levels.
- (c) investigate the changes in beach alignment which construction of the breakwater will cause.

## 2.1 Breakwater.

In order to determine a suitable breakwater length and alignment, various breakwaters were constructed in the Botany Bay Hydraulic Model. These were built so that no overtopping occurred. It was decided to calculate this aspect separately; the scale of 1:120 being considered too small for adequate run-up and overtopping determinations. Figure 2 shows the final design alignment. Breakwater lengths of 10m greater and smaller length were also tested in the model. The alignment shown in Figure 2 is the centre-line and fulfils the 10 boat length requirement taking into consideration the physical size of the structure.

Previous testing in the Botany Bay Hydraulic Model enable wave-height coefficients to be determined for the offshore directions east-south-east, south-east and south-south-east, combined with periods of 8, 10, 12 and 14 seconds at position A shown in Figure 2. These conditions contribute the major proportions to the wave energy reaching Yarra Bay as the entrance to Botany Bay faces south-east. The direction of reflected waves arriving in Yarra Bay varies very little with offshore direction-period combination and there is very little incident wave energy. It was therefore decided not to carry out this full range of 12 tests for the breakwater, but instead examine results available for position A and then, on the basis of partial probabilities of wave-height exceedance, carry out tests for the 5 most important conditions.

Two positions on the lee side of the breakwater were chosen and wave-height coefficients measured relative to position A, Figure 2. The tests were conducted at tide levels of 0.2m, 0.9m (M.W.) and 1.5m, these levels allowing probabilities of occurrence to be apportioned as 0.25, 0.5 and 0.25 respectively for tide level intervals spanning these values. Highest Astronomical Tide is 1.9m and the datum is Indian Springs Low Water, 0.0m. Depth at the breakwater head was 3.1m at I.S.L.W. Coefficients for the untested wave conditions were estimated by extrapolation.

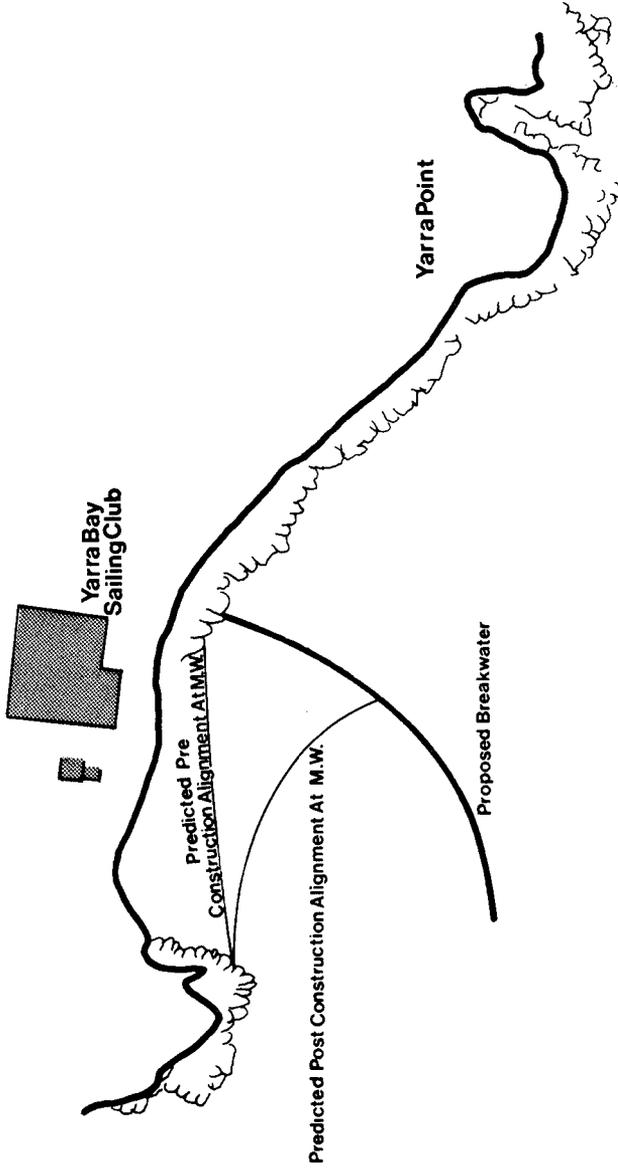


FIG. 2

● A

Overtopping coefficients,  $K_t$ , were determined following Cross and Sollitt (3). Their envelope equation is :-

$$K_t = 0.65 \left( 1.1 - \frac{H_b}{R} \right) \quad (1)$$

where  $H_b$  is breakwater crest elevation above Still Water Level.

$R$  is run-up.

Run-up was determined using data presented in the Shore Protection Manual (7) for run-up on permeable rubble slopes, a slope of 1:1.5 being used.  $K_t$  values were determined for a range of unrefracted deep water wave-heights at the same tide levels used in the model tests.

It was then assumed that wave coefficient in the lee of the breakwater,  $K$ , could be conservatively estimated by :-

$$K = K_A K_t + K_A K_{di} \quad (2)$$

where  $K_{di}$  was determined from model tests.

$K_A$  is the wave coefficient at position A, seaward of the breakwater.

Adoption of this method is considered to be conservative.  $K$  was evaluated for the different tide levels and for different breakwater crest elevations, 2.0m and 3.0m above I.S.L.W. Equations like (2) were used to determine plots of lee-side wave-height versus offshore wave-height; this not being a linear relationship. Then, following Lawson, (4), and applying weights of 1, 2 and 1 to the low, mean and high water level tests, partial probabilities of wave-height exceedance for the two lee positions were obtained. Summation of all these partial probabilities produced wave-height climate data which could then be used to calculate expected wave run-up on a statistical basis. The breakwater length and crest height were chosen to provide acceptable wave run-up conditions to the clubhouse.

However, wave run-up (i.e. in the horizontal plane) depends upon beach alignment and it was necessary to determine the expected ultimate mean alignment to be able to finalize the design. Additionally, in the final design, tribar armour units were used at the outer end of the breakwater. Run-up on these units is approximately 20% greater, Foster (5), than on rubble mound structures and so this change had to be included in the final calculations.

## 2.2 Beach Alignment.

Following the beginning of construction of the Bumborah Point revetment and the consequent change in wave climate in Yarra Bay, the beach was forced to realign, Treloar (8). In general this produced a wider beach in front of the sailing club, but the much higher waves and steeper beach lead to run-up being increased and dangerous boat launching conditions. A beach survey programme was begun in 1971 with surveys being at 3 monthly intervals. These surveys recorded beach levels at 12 cross-sections along the beach and extended from the frontal dune to a depth of about 4m below I.S.L.W. At the northern end the frontal dune soon disappeared and storm waves began eroding the bund protecting an old rubbish tip. Curve fitting to the survey data and calculations based on data measured in the Botany Bay Hydraulic Model were carried out to determine the expected ultimate mean beach alignment, Treloar (8). Statistical analysis of the survey data enabled the 95% confidence limits, based on a postulated normal distribution of beach width, to be obtained. This figure is approximately + 10m and results from onshore-offshore movement of sand, slope changes, alignment fluctuations and survey error. It is believed that this is a good estimate of the width band within which a beach contour (at Mean Water) will lie in Yarra Bay.

In 1974 about 30,000 cubic metres of sand were placed on the beach in Yarra Bay to overcome the problems caused by re-alignment. Beach surveys were continued and inclusion of this sand quantity in the beach alignment calculations enabled a predicted pre-construction alignment to be determined.

The pattern of diffracted waves behind the breakwater was used to determine an estimate of ultimate beach alignment. Circular wave fronts were drawn, centred at the breakwater seaward end and tangential to the predicted ultimate mean beach at the limiting orthogonal. This procedure assumes that the sand volume which will move in behind the breakwater will be sufficiently small to produce minimal narrowing of the beach in the remainder of the bay. The process of sand moving in behind the breakwater commenced soon after construction began. However, the plan shape of this "fillet" developed beyond the calculated circular arc alignment, this development being caused by the radiation stress resulting from the lee-side wave-height gradient. Sand will be transported by the current caused by this radiation stress gradient until the lee beach develops sufficiently for the gradient to become zero and a condition of stability established. Based on the circular arc beach and wave data on the lee side, run-up to the clubhouse was established for the various test conditions.

On the exposed side a complicated wave pattern was to be expected. An interference zone, formed essentially from waves reflected from the Bumborah Point revetment and waves reflecting again from the curved alignment of this breakwater, would cause further re-alignment of the small beach area between the breakwater and Yarra Point to the south. This re-alignment would take the form of a reduction in beach width near the breakwater. In order to avoid undermining of the breakwater toe 3,000 cubic metres of sand were placed on this beach area after construction was completed. It was considered that a larger quantity would not remain "entrapped" on the beach and might tend to move away around the rocky headland of Yarra Point.

## 2.2 Design Wave-Height.

The design wave-height information was obtained using wave-height coefficients at position A, Figure 2, and 5 years of Waverider buoy data obtained from the offshore deep water installation. This data has been collected with directional information and log-normal wave-height

exceedance distributions determined for direction - average zero crossing period combinations. The model mono-chromatic wave periods were related to the prototype zero crossing periods by considering proportions of energy in frequency bands spanning each monochromatic frequency in a Moskowitz spectrum defined for each zero crossing period. These data were used in the structural design of the breakwater. Table 1 shows wave-height exceedance data for significant wave-height as well as data obtained from 440 days of wave records measured by a Waverider buoy installation at position A. These latter data were not available at the design stage but confirm, with reasonable accuracy, the model results.

<u>Probability of Exceedance in Days/Year</u>	<u>Significant Wave-height (m)</u>	
	<u>Model</u>	<u>Prototype</u>
1	1.9	1.8
2	1.6	1.5
3	1.5	1.3

TABLE 1: Design and Measured Wave-Height Data

### 3. STRUCTURAL DESIGN

As indicated earlier the need to construct this breakwater became more urgent as the work on the major revetment in the port development progressed (Figure 1).

Increasing lengths of exposed core and completed armour at different construction stages of the revetment, and their various effects on reflection of waves to this site during the storms which occurred in the years 1974-75, produced damage to the club house resulting in the urgent need for ameliorative works to be undertaken.

In order that protection could be afforded prior to the principal storm period of May-June 1976 the design of the breakwater was completed without wave flume tests.

The structural design of the breakwater was thus based on a survey of information contained in the literature. Figure 3 indicates



site details for the final design.

### 3.1 Materials Available.

At the wave heights indicated, use of concrete armour tribars sized at 2½ tonnes and 6 tonnes was possible as a result of their use on the main revetment in the Port.

Rock armour sizes of 1 and 2 tonnes were similarly being used in that project. This material was being won from a quarry on the south coast of N.S.W., a road haul of approximately 120km.

Suitable core material was available up to 2 tonnes R.O.Q. within 60km from the site. Sizing of this material was 65% finer than 1 tonne.

The urgency of construction indicated the need for use of these available materials, if possible, in this breakwater.

### 3.2 Wave Height

In keeping with design wave heights for other port works, the significant wave height corresponding to a 0.01% probability of exceedance was considered appropriate for the structural design of the breakwater. At this probability, significant wave height is exceeded for a total of nine hours in 10 years.

As earlier indicated overtopping to some extent could be tolerated.

### 3.3 Crest Elevation.

A major requirement for this breakwater construction was to keep the crest elevation as low as possible. Additionally, so that the core material could be placed at normal stages of the tide, a crest elevation of approximately 3 metres above I.S.L.W. would achieve this aim. The depth at the breakwater's head is approximately 3.1 metres below I.S.L.W.

### 3.4 Head.

As reflected in the Shore Protection Manual's recommendations, (7), a structure head

is more conservatively treated for armour design.

The most commonly used formula for estimating armour sizes for non-overtopped breakwaters subject to breaking and non-breaking waves is the Hudson formula. This formula is based on a no damage stability criterion, defining permissible damage as displacement of up to 5% of the armour cover layer.

The largest available rock armour size was 3 tonnes for placing in two layers, but at the design stage this size of rock was not stockpiled at any possible source. Data available indicated that the cost increased by 50% for 3 tonne rock above 1 to 2 tonne size.

For minor overtopping, the Shore Protection Manual (7), recommends a  $K_d$  of 2.9 on a slope of  $1\frac{1}{2}$  to 1 with levels of damage from 0 to 5%.

Foster (5), indicated damage co-efficient  $K_d$  values of 4 and 8 for 3 tonnes rock at slopes of  $1\frac{1}{2}$  to 1 and 2 to 1 would lead to damage levels of 10% and 5% respectively for no overtopping. The data compares favourably with the Shore Protection Manual recommendations.

A conclusion reached by Lording (2), indicated that damage of a rock armour breakwater head should be restricted to 3%. Even though the testing in that report was limited, it was concluded that the most critical design for the tested structure was for S.W.L. just below crest level. For all tide stages this condition applies at the site of this breakwater for the design crest level.

Thus on the above data it appeared that damage levels of 3% for 3 tonne rock might occur at the 0.01% probability wave height.

Investigation of a concrete armour alternative was then pursued as flatter slopes than 2 to 1, and larger rock size may not have ensured the desirable lower damage levels of 3% at the head. Additionally, rubble deposited in the lee of the breakwater head may need to be removed to ensure adequate navigation.

Construction of the major port works, having included the use of single layers of tribars of 2½ and 6 tonne, enabled easy availability of these armour unit forms. For the 0.01% exceedance wave height the co-efficient of damage for 2½ tonnes tribars is 6.9, just below recommendations in the Shore Protection Manual (7) for minor overtopping. It is noted that in these recommendations the margin of uniform tribars (1 layer) over random placed tribars (2 layers) on the structure trunk is not maintained for the structure head. This presumably results because uniform placing makes greater use of frictional resistance than does random placing and this gain becomes less significant when the breakwater face is convex and the lateral reactions between blocks are somewhat reduced. For overtopping the damage co-efficient of 6.9 is not conservative for a breakwater head when looking at the Shore Protection Manual recommendations, (7) if the tribars are placed in one layer. However, the construction cost saving in placing tribars in one layer is significant.

To increase the size of tribar units for which forms were readily available, namely 6 tonnes, meant that the elevation of the core material would be below M.H.W.M. as armour unit and underlayer size is increased and this would be an unsatisfactory working level for a contractor.

In tests by Jackson (1), random placing of tribars in two layers on a convex surface of 1½ to 1 slope indicated a crest level to block dimension (C/D) ratio of 0.5 and still water level radius to block dimension ratio of 5. In this design these ratios are 0.8 and 3.4 indicating a tighter proposed head geometry when compared to the tests. Jackson recommended a damage co-efficient of 6 based on the Shore Protection Manual density of placement for two layers of random placed tribars.

In other work in the port tribars have been pattern placed ensuring leg to leg contact between adjacent tribars. This technique achieves lower placing densities and higher stability than placing uniformly to the Shore Protection Manual recommendations (7), Foster (5). However, pattern placing is difficult below water and a pattern could not be achieved in construction on the convex face at the head.

Foster's data (5), indicates for single layer tribar tests a damage co-efficient,  $K_d$  of 7 for a damage percentage level of 2%. That level of model damage is viewed by the writers to indicate a virtually maintenance free structure.

While Lording's data (2) applied to a rubble mound structure it indicated that critical damage occurred more to the crest and lee side for water levels just below the crest. Movement of tribars would be restricted in these areas by ensuring that :-

- (a) they were anchored in a substantial toe, achieved during excavation for trimming of the core; and
- (b) at the intersection of the sloping face and the crest adjacent tribars would be placed under close supervision to ensure leg to leg contact.

It was concluded that, provided tribars were placed to achieve the density stated in the Shore Protection Manual, with tight packing on the head, armouring of the crest, seaward and lee slopes in 2½ tonnes tribars would give an economical maintenance free head. The design adopted at the head is indicated in Figure 4.

### 3.5 Trunk.

The head design adopted continued for approximately 44 metres along the centreline of the breakwater where the bed is reasonably level. From that point, chainage 100m, water depth decreases rapidly.

For the design of the trunk it was proposed to use an armour stone of similar size for both crest and side slopes due to the short length to be armoured. Lording (2), concluded that a limit of 6% displacement of rock armour would ensure structural integrity of the trunk. At the design stage the Maritime Services Board had a commitment for over supply of 1 tonne rock in other port works.

Lording (2) indicated that crest armour size necessary for stability with 6% displacement

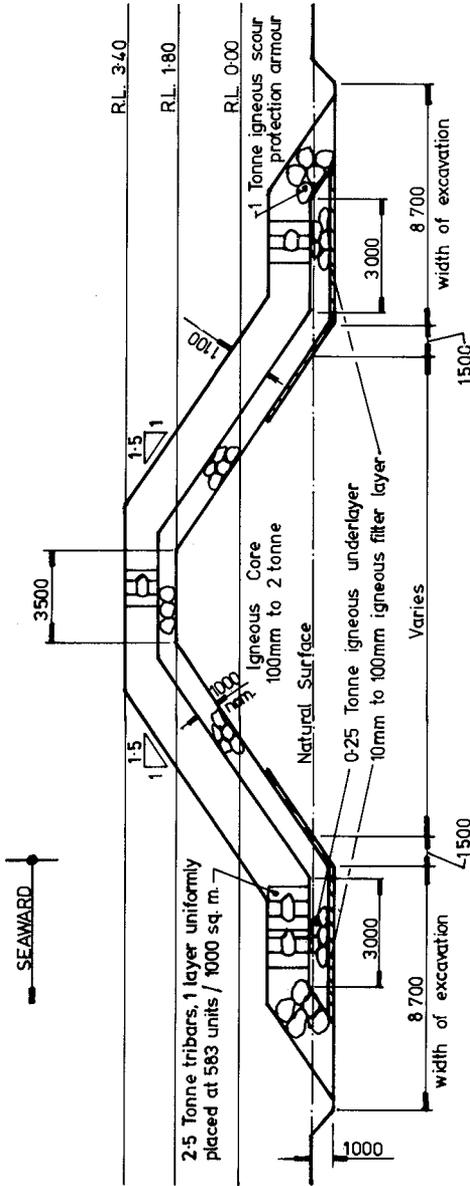


FIG. 4

appeared more critical at critical still water depths. Additionally identification of the critical still water depth for a given armour zone may permit estimation of armour sizes for that zone by an empirical formula (such as Hudson's) with a degree of accuracy not greatly inferior to that achieved for the non-overtopped criteria. For the  $1\frac{1}{2}$  to 1 seaward and leeward slopes tested, a  $K_d$  of 7 corresponds to a satisfactory displacement of 6%. The maximum armour size was less than that required for non-overtopped breakwater.

The 1 tonne armour size, readily available, corresponded to a 6% damage level, at a wave height of 2.5 metres on a  $1\frac{1}{2}$  to 1 slope, equivalent to a damage co-efficient,  $K_d$  of 7.

The structural integrity of the breakwater would not be lost at that damage level. In view of the low probability of higher wave heights mentioned herein, greater damage could be expected, but the structure would protect the club house to allow repair when necessary.

The trunk design adopted is shown in Figure 5.

### 3.6 Post Construction Performance.

Since completion of the breakwater, damage to the trunk section only has occurred following a storm of maximum significant wave height of 2.5 metres at position A (Figure 2). The principal reason for damage was undersize rock in this area. However, the breakwater was not breached and damage occurred to the seaward part of the crest and the seaward face between chainages 70 and 90 metres. At the area of damage, during construction, some slumping of the unprotected core occurred due to inferior quality of core rock. If all the unsatisfactory rock was not detected and replaced prior to armouring the breakwater, storm damage may have been triggered by local slumping of the core during the storm in question.

The quantity of rock placed to repair this section represented less than 3% of the total 1 tonne rock placed on the seaward and lee slopes and the crest of the trunk. The repair rock was sized 2 to 3 tonnes and the cross section (Figure

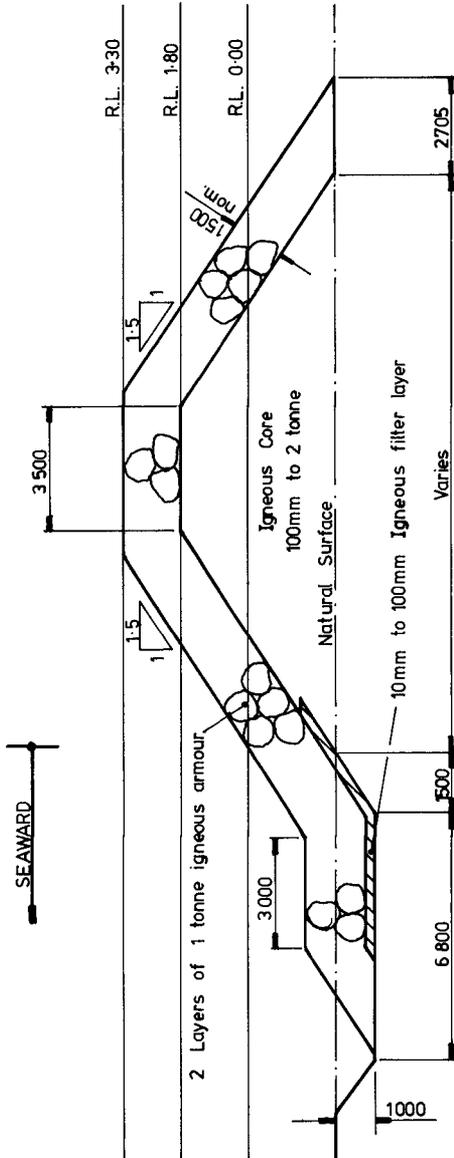


FIG. 5

5) was reinstated.

On the cost information available, the repair demonstrated the saving in construction cost if the trunk had been armoured in larger rock size, and/or by using a flatter slope. However this saving is demonstrated only for this short breakwater trunk.

For an overtopped breakwater it is not recommended that reduction of rock armour size below that for a non overtopped breakwater be adopted unless wave flume tests are undertaken. Additional protection would have certainly been given to this breakwater because of depth limitations at the site.

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