BREAKWATER WITH A SAND BITUMEN CORE

E. Loewy — B.Sc., A.C.G.I., C. Eng., F.I.C.E., M.A.S.C.E. Partner, Watermeyer Halcrow and Partners.

K.G. Witthaus — B.Sc., Pr. Eng., F.I.C.E., Partner, Watermeyer, Halcrow and Partners.

R.J. Maddrell — B.Sc., Ph.D., M.I.C.E., Senior Engineer, Sir William Halcrow and Partners.

J.P. Wood — B.Sc., M.I.C.E., Resident Engineer, Marine Works, Koeberg Nuclear Power Station.

SUMMARY

Sand bitumen was used in the core of a rubble mound breakwater to form an impermeable barrier, essentially to inhibit the recirculation of warm water exhausted from a power station outlet back into the intake pumps. The core had a secondary advantage in permitting the placing of bitumen grouting to the rock on the lee face of the breakwater to stabilise it against wave overtopping.

The paper describes the reasons which led to the adoption of this unique method on a very exposed coastline. It also outlines some of the measured properties of the sand bitumen and the methods used and experience gained in its mixing and placing.

The sand bitumen, which has not to our knowledge been used for this purpose and as a core material previously, is proving to be successful.

1. INTRODUCTION

In 1967, the Electricity Supply Commission of South Africa (ESCOM) decided upon the location of Africa's first nuclear power station at a coastal site adjacent to the Atlantic Ocean, some 35 kilometres to the north of Cape Town. The first phase of development of this project is for the construction of two reactors, each of 920 MW, which should be commissioned by 1984. Further development has been allowed for in the future immediately to the north of the station which could eventually increase its capacity.

The site was selected, inter alia, because of its proximity to unlimited supplies of cooling water coming from the Antartic in the cold Benguela current. However, this section of coastline is extremely exposed and devoid of any naturally deep and protected area from which to draw the water. The wave climate is severe with the deep water significant wave height exceeding 2 metres for about 38% of the time and maximum wave heights of 11 metres have been recorded. It is axiomatic that power station cooling water should not contain heavy concentrations of ssdiment and, on this coast, with its sevsre wave climate and 1 in 60 to 1 in 100 bed slopes, one of the problems relating to the abstraction of cooling water is that considerable quantities of sand are suspended in a very wide surf zone. The firm of Watermeyer, Halcrow and Partners, which is a joint partnership between Watermeyer, Legge, Piesold and Uhlmann and Sir William Halcrow and Partners was retained by ESCOM to advise on the best means of obtaining cooling water and, later, to undertake the detailed engineering design and supervision of construction of the scheme that was finally adopted.

Feasibility studies on a number of possible schemes, which included tunnels with offshore intakes, concluded that the most suitable and economic solution would be to situate the pump houses within a dredged basin, protected by rubble mound breakwaters.

The configuration and detailed design of this basin had to satisfy the parameters of:-

(a) stability of the breakwaters under wave action;

(b) limiting wave penetration and wave heights at the intakes;

(c) controlling sediment build-up around and, in particular, in the basin; and

(d) restricting recirculation of warm exhaust water.

Detailed site measurements of the local marine environment (1), were combined with physical model studies of the breakwaters and a mathematical modelling of sediment build-up (2) to examine items (a) to (c) above. These considerations essentially determined the final configuration.

As illustrated in Figure 1, the pump intakes are located along the original shoreline and are protected by the main (south) breakwater which is 933 m long and the north breakwater which is 578 m long. The basin entrance is in about 8 metres of water at high water.

In this first phase of station development the pump intakes will draw in a total of about 80 cumecs of seawater at ambient temperatures ranging between 8 and 16° C. This water will be heated through a range of about 10° C in its passage through the turbine condensers and will be discharged through a shallow outfall channel into the surf zone just to the south of the main breakwater (see Fig. 1).

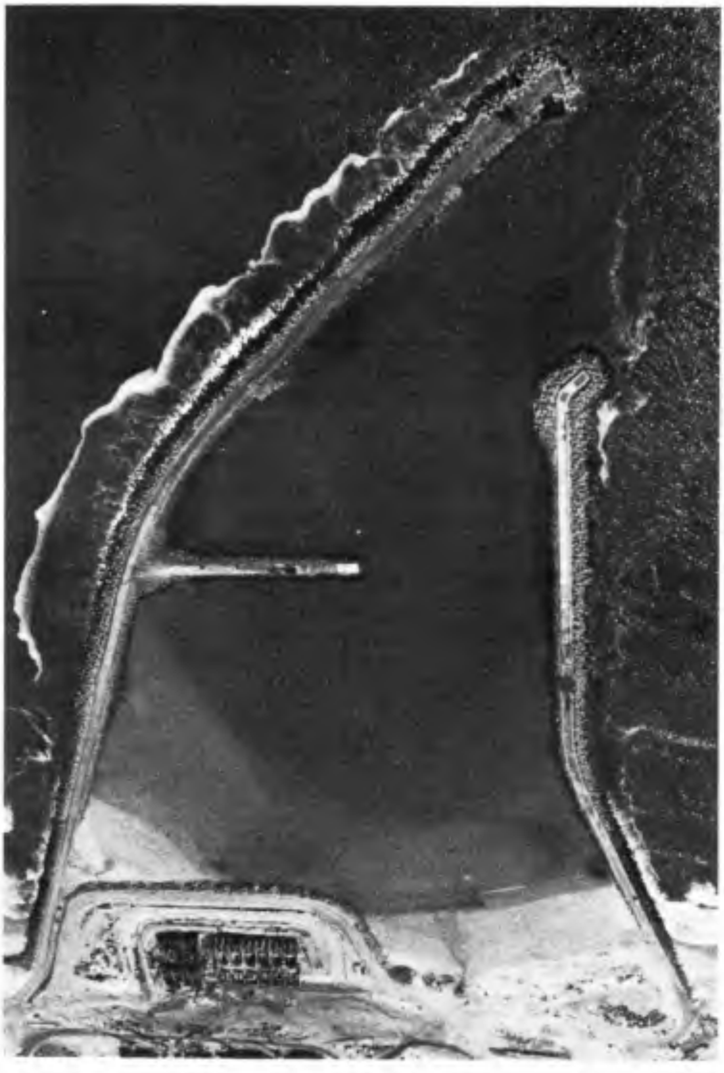

Figure 1. CONFIGURATION OF INTAKE BASIN

It was feared that, in connection with (d) above, the warm outlet water could pass through the rubble mound of the south breakwater in significant quantities and re-enter the pump intakes during certain times of the year. Field observations using dye were made of the transmission of water through the breakwaters at Richard's Bay which confirmed these fears.

The flow through a rubble mound breakwater without an impermeable core was also studied theoretically. Because of turbulence, Darcy's formula was inappropriate and Hazen's formula for permeability was adopted. For the purposes of this calculation, wave set-up outside the basin was assumed whilst the cooling water pumps were drawing sea water from within. This produced low flows in the order of about 3 cumecs over 300 m of breakwater length. However, when it was assumed that the breakwater contained a network of very twisted rough pipes, the flow became significant e.g. for 500 kg and 1 000 kg rock over a 300 m length of breakwater the calculated equivalent flows through the structure were 28 and 55 cumecs respectively.

Similar recirculation problems were solved, following construction, at Fukushima nuclear power station in Japan by driving steel sheet piles parallel with and close to the breakwater inside the basin. This arrangement was examined for Koeberg, as well as several other alternative solutions involving grouting, sand bitumen, slicework and blockwork. Eventually it was decided to construct an impermeable core to the breakwater with a mixture of sand and bitumen. This construction material had been used previously in the United States and, more notably, in the Netherlands but, as far as is known, never before in this manner as a breakwater core material.

The sand bitumen core offers a further benefit at the seaward, more exposed end of the south breakwater. Normally the crest of the structure in this area would have to be heightened and/or the lee face reinforced with additional and possibly heavier armour units as a safeguard against wave overtopping. Fortunately, however, the sand bitumen core acts as a barrier against the transmission of air/water pressure through the rubble mound, and permits the use of relatively light rock on the lee face, grouted to act as a spillway for overtopping waves. Without the impermeable sand bitumen core, air and water pressure within the rubble mound caused by wave attack on the seaward face would almost certainly dislodge the grouted rock on the leeward face. The rock grouting was also carried out with a sand bitumen grout, but since the characteristics required were different, the constituents and mixing proportions differed from the sand bitumen core.

Breakwater construction commenced in October, 1977, and placing of the rubble mound and sand bitumen was completed in December, 1979. The construction has proved successful and the resistance to erosion of the sand bitumen core when exposed to the force of the waves at the scar end during construction was found to be surprisingly good - better in fact than the filter layers of rock on either side of it.

2. DESIGN CONSIDERATIONS

Two possible methods of constructing a barrier within the heart of the south breakwater to impede the passage of water through the rubble were examined in detail, namely:

(i) the installation of large precast hollow concrete rectangular boxes with compatible curved front and rear faces placed in contact with each other on a bed of prepared rubble (i.e. a type of slicework construction).

(ii) a sand bitumen core.

Solution (i) would have provided a relatively impermeable boundary only between the levels of - 1.0 m and + 2.0 m G.M.S.L. However, the rock above it would have been grouted and it was considered that this form of construction would provide an adequate barrier to the passage of heated exhaust water because the heated water will largely be confined to the surface layer.

Solution (ii), because the column of sand bitumen would be continuous, provided an impermeable barrier throughout the depth of the breakwater. It did, however, require the provision of filter layers to protect the core during the life of the breakwater.

The feasibility of construction of the breakwater utilising the hollow concrete box solution was tested in a 3D model, as it was suspected that the boxes might be unstable during the construction stage. It was envisaged that the core area would be partly protected from wave action, along the most exposed length of breakwater, by rock mounds on either side. Although tests showed the boxes to be vulnerable during periods of severe wave action, it was concluded that construction was feasible.

The comparative study showed that the sand bitumen core was clearly the better of the two solutions. No special method of placing was required (i.e. it could be placed in a similar manner to the rock). Its use, unprotected in temporary works in Europe, had shown it to be a reasonably tough, resilient, material. It would provide a complete barrier between sea bed level and the underside of the concrete cap. And it was significantly cheaper than the precast concrete alternative when all factors were taken into account.

It was envisaged that the sand bitumen would be placed, from trays, in the form of a Christmas tree, the Contractor using the same plant to transport both the sand bitumen and rock. However, only minimum thicknesses for the sand bitumen core and its adjacent filter layers were specified, giving the Contractor freedom to choose his own method of placing (Figure 3). In the event, the Contractor chose to adopt a Christmas tree similar to that originally envisaged (Figure 4).

It was appreciated that, during storms, some large rocks might be washed into the core area. This could pose long-term stability problems because, if the bitumen on placing were to span a large void, this could gradually "pipe" through the sand bitumen, its size only marginally decreasing. Consequently, it was thought that, if necessary, after storms a sand bitumen mix with a higher bitumen content could be placed, at higher temperature, in order to provide a material of higher plasticity and minimise the likelihood of large voids in the core area. In the event, however, no increase in bitumen content for this purpose was required.

The presence of an impermeable core offers the advantage of permitting the lee face of the breakwater to be grouted. However, it also means that the transient pressure which would normally be transmitted through a rubble structure must be contained or dissipated within the seaward half of the structure. As may be seen on Figure 3, blow holes some 6 m apart, were provided through the cap on the seaward side of the core to dissipate these pressures.

Consideration was also given to the effect that the impermeable core might have in decreasing the wave energy absorption capacity when compared with a normal rubble mound breakwater. This might result in a need for heavier dolosse armour units than would otherwise be the case. Consequently, the stability of the breakwater with the impermeable core was extensively model tested both in a flume and in a 1:80 scale 3D model.

In making these tests, it was realised that the 3D model would tend to give conservative answers (i.e. less energy would be destroyed in the model than in the prototype) because of Reynolds scale effects. Another conservative factor was that, for site safety reasons, extreme environmental conditions were considered in the breakwater design. However, even when applying an estimated "one-in-a-million" storm condition (including surge plus wave set-up plus run-up) the design wave remains depth limited, and the stability of the outer armour units was found to be acceptable.

The sizes of the dolosse armour units adopted on the south breakwater were 6 t, 15 t and 20 t and, on the north breakwater, 6 t and 15 t. In fixing dolos sizes, consideration was also given to available crane capacities, overall cost, and the requirement that the need for future maintenance of the breakwater should be minimized.

3. SAND BITUMEN PROPERTIES

3.1 Contract Requirements and Trial Mixes

The Contract Specification required sand, won locally from sand dunes on the site, to be mixed with 80/100 pen. straight run bitumen. The bitumen content could be fixed, subject to trial mixes, at between 3 and 6 percent by weight. The grading limits were specified to lie anywhere between 2 mm and 0.063 mm sieve sizes.

At the commencement of the Contract suitable dunes were located. Table 1 below shows a typical grading curve which indicates that the sand was fine and essentially single-sized.

Sieve Sizes (mm)	Percentage Passing
1,18	100
0,600	99
0,300	92
0,150	6
0,075	0

TABLE 1. Typical Grading of Local Dune Sand

Before construction commenced, trial mixes were undertaken with bitumen contents ranging from 3 to 6% in half percent increments as specified in the Contract Document. In the abeence of documented methods of judging their suitability, each of the mixes was appraised by both the Engineer and the Contractor on a somewhat subjective basis. It was observed that the mixes containing 3 and 3½% bitumen had little inherent strength or cohesion and looked unstable. The 4% mix showed a remarkable improvement whilst the 4½% mix was marginally better still. Further increases in bitumen content did not appear to improve the properties of the mix. In view of likely batching inaccuracies (see ASTM D1663-74) of ± ½% the bitumen content was set at 4½% in the knowledge that if it dropped to 4% the resulting mix would be acceptable.

In practice, it was found that the tolerances on the batches provided by the Contractor were very much better than ± ½% and in a sample of 119 bitumen content tests (of many more taken) the standard deviation on the bitumen content wae 0.128%.

3.2 Laboratory Teeting

With the exception of routine bitumen content and sand grading teste, no other laboratory testing was required under the Contract.

Apart from the data published by van Asbeck (4), Visser (5) and Kerkhoven (6), very little data could be traced on the behaviour and teeting of eand bitumen. It eeemed important to carry out some objective teste on thie unique material in order that:

- a record could be made of the physical state of the material during oonstruction for future reference should deterioration ever oocur;
- to give an indication of the propertiee of the materials that are actually in place in the breakwaters;
- to provide at least some data for others in the future.

Therefore the ensuing tests were undertaken, although it should be noted that the results are not necessarily definitive. They should be interpreted with care, as the material actually used in the breakwater was placed under a wide variety of physical conditions and was not compacted. Its properties in-situ may therefore be variable and the laboratory test results may not be truly representative of the actual sand bitumen core.

Finally, it should be pointed out that the characteristics of any bituminous mix are dependent upon numerous variables such as sand grading, temperature, bitumen content, bitumen grade, etc., and that these results are therefore not necessarily universally applicable.

3.2.1 Marshall Testing

Two samples with 4½% bitumen were tested by the Standard Marshall Method (ASTM D1559).

The results are given below.

	Sample No.	
	1	2
Briquette compacted by	10 blows each side	5 blows each side
Density of Mix tonnes/m^3	1,8	1,8
S.G. of Sand	2,6	2,6
Voids in Mineral Aggregate %	30,8	30,8
Stability kN	0,61	0,24
Flow mm	0,46	9,0

TABLE 2. MARSHALL DATA

3.2.2 Durability Testing

This test was made in an attempt to obtain some quantitative index of the durability of the sand bitumen in service in the breakwater. The mix contained 4½% bitumen and the test was the "Wet-Dry Durability Test" adapted for sand bitumen. The procedure for testing is given in the South African Standard Methods of Testing Materials Method A19 which is almost identical to ASTM D558.57 and D559-57. Samples, compacted in a standard fashion, were subjected to repeated cycles of brushing with a wire brush and the loss of weight recorded. Tests were carried out for five cycles of brushing on four sets of four samples. Figure 2 summaries the results obtained.

3.2.3 Permeability Tests

Three specimens of the 4½% content mix were tested by filling a 76 mm dia. ring 18,75 mm thick with sand bitumen at 120° C and allowing it to cool. Standard falling head permeameter tests were performed with water at 12° C.

The results are given in Table 3 below.

Test No.	Dry Density (kg/m3)	Coefficient of Permeability (m/s)
1	1685	3.7×10^{-5}
2	1681	3.7×10^{-5}
3	1751	2.3×10^{-5}

TABLE 3. RESULTS OF PERMEABILITY TESTS

3.2.4 Shear Strength Tests

Three undrained, unconsolidated triaxial tests were carried out on the 4½% mix. Samples were prepared by filling a 75 mm dia. x 150 mm deep mould with sand bitumen heated to 120° C and allowing it to cool to testing temperature. Tests were conducted at temperatures considered representative of the sea adjacent to the breakwaters.

A shear box test was also carried out in an attempt to assess the strength of the cold joint between adjoining layers. Here the lower layer was permitted to cool before the overlying layer was added.

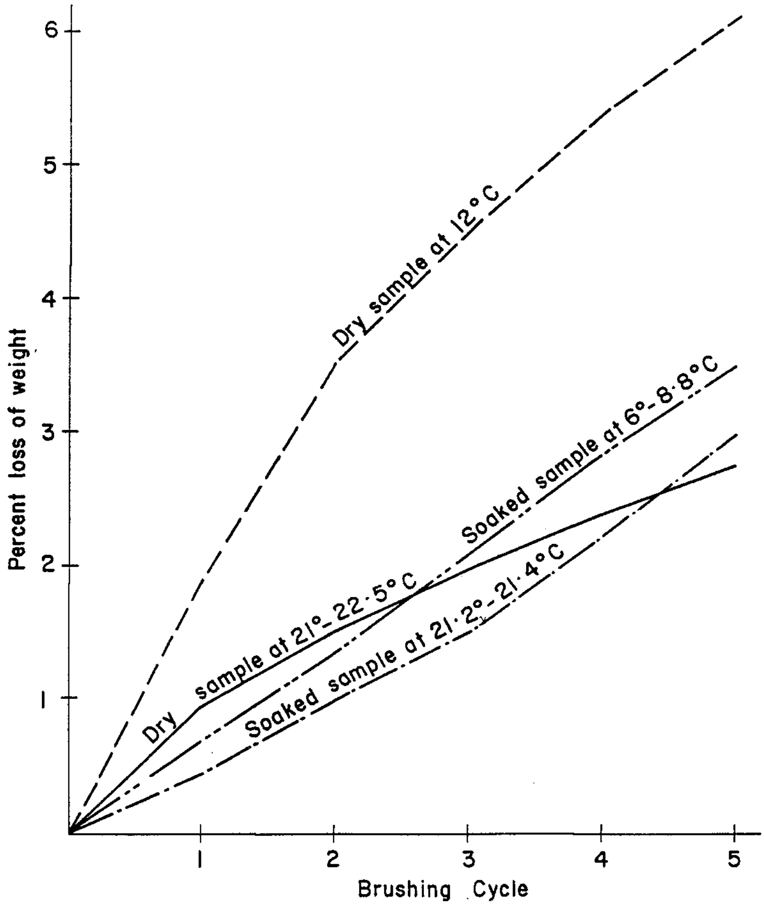

Figure 2. RESULTS OF BRUSHING TESTS

Test Type	Test Temp.	Apparent Cohesion	Friction Angle
	^{o} C	KPa	ϕ
Triaxial UU	8	175	53^{o}
"	12	90	46^{o}
"	16	60	45.5^{o}
Shear box	12	74	45^{o}

TABLE 4. RESULTS OF SHEAR STRENGTH TESTS

4. APPLICATION

The Main Contractor for the construction of the breakwaters sensibly employed a specialist asphalting firm as sub-contractor to produce the sand bitumen and bitumen grout mixes. This sub-contractor established a continuous drum mixer on the site which could reach an output of about 40 tonnes of material per hour.

The bitumen was added by nozzle at the entrance to the drum and the flow rate was regulated by a load cell on the sand supply conveyor. This proved extremely satisfactory.

Batching proceeded on a daily basis and the materials were generally mixed at about 115^{o} to 130^{o} C. The material was placed in storage bins to cool down to placing temperatures that could range between 40^{o} and 95^{o} C. In practice, it was found that the optimum temperature for placing material in the sea was about 60^{o} C. At higher temperatures the mix held together poorly and the washout losses were high whilst at lower temperatures the transport lorries had difficulty in getting the material to tip out. It should be noted that the material retains its temperature for many days thus it is both possible and reasonable to specify the placing temperature. In the height of summer, it still had ample heat to be used after a week. In winter it had to be used within 4 days. Heating elements were built into the bin walls and these were occasionally used to heat up mixes which had fallen below the minimum specified temperature. This procedure was sometimes supplemented by mixing cold and freshly mixed hot batches with a front-end loader.

Sand bitumen was loaded from the bins into tipper trucks and transported to the breakwater. There it was tipped into the rock tray of the breakwater crane which then positioned the load according to a predetermined pattern and tipped it into the sea. Some 37 000 cu. metres of sand bitumen were placed in this manner in individual loads of about 7 tonnes.

Figure 3 shows a typical idealised cross-section through the breakwater with its sand bitumen core. The core itself is protected on either side by graded rock filters with the seaward face armoured with dolosse units. In practice, the Contractor could not, of course, construct this idealised cross-section and dumped the rock and sand bitumen in a succession of layers, resulting in the "Christmas Tree" cross-section. This is shown on Figure 4.

Although the sand bitumen is protected by the rock filters in its permanent state in the breakwater, it was exposed to wave attack at the scar end during construction. Under these conditions it proved highly resistant to wave attack and this is well illustrated in Figure 5 which shows sand bitumen being placed at about water level. As can be seen, the mix stands up well to the erosive force of the waves. No doubt its long-term erosion resistance is poor and it requires the filter armouring but this property was extremely useful during the short construction phase.

Very soon after placing, the cold sea hardened the outer shell of the sand bitumen which must have greatly improved its early erosion resistance. Many instances of this property were observed during construction, where wave action removed the rock on either side of the sand bitumen, leaving the core standing proud.

Another helpful aspect of this material was found to be its ability to envelop discrete boulders. As previously mentioned, it was foreseen at the design stage that boulders could be washed by the sea or accidentally dropped by the Contractor into the core area. Observations during construction of the breakwater and core samples obtained by drilling indicated that such boulders become totally surrounded by sand bitumen thus preserving the integrity of the core. These boreholes also indicated that at lower sections, the core area sometimes tended to become filled with sand before the sand bitumen was placed on top. In one instance a pocket of sand over 1 metre thick was located in a borehole between the underside of the core and the top of the small rock foundation blanket.

Two sets of prototype tests, using dye placed on the south of the breakwater during strong south to south east winds have indicated that little or no transmission of water is taking place through the core.

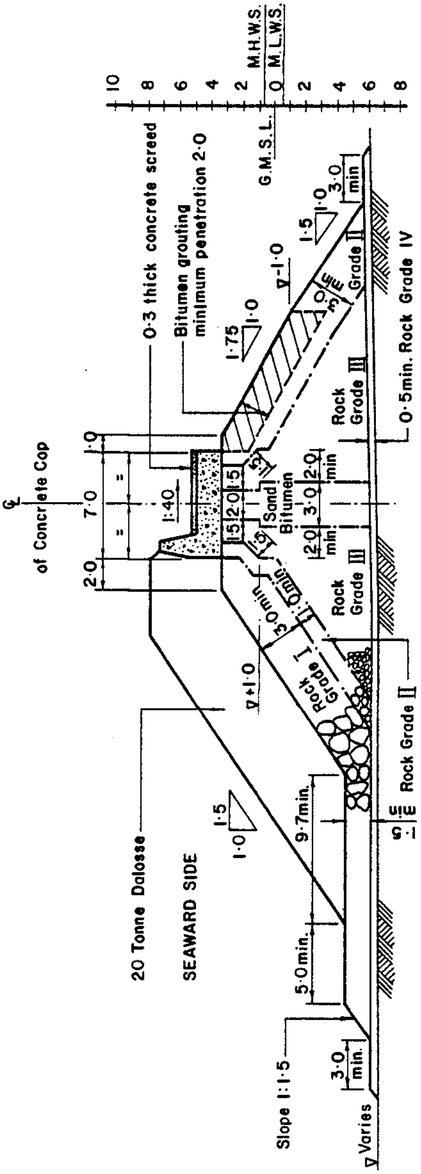

Figure 3. IDEALISED CROSS-SECTION THROUGH BREAKWATER

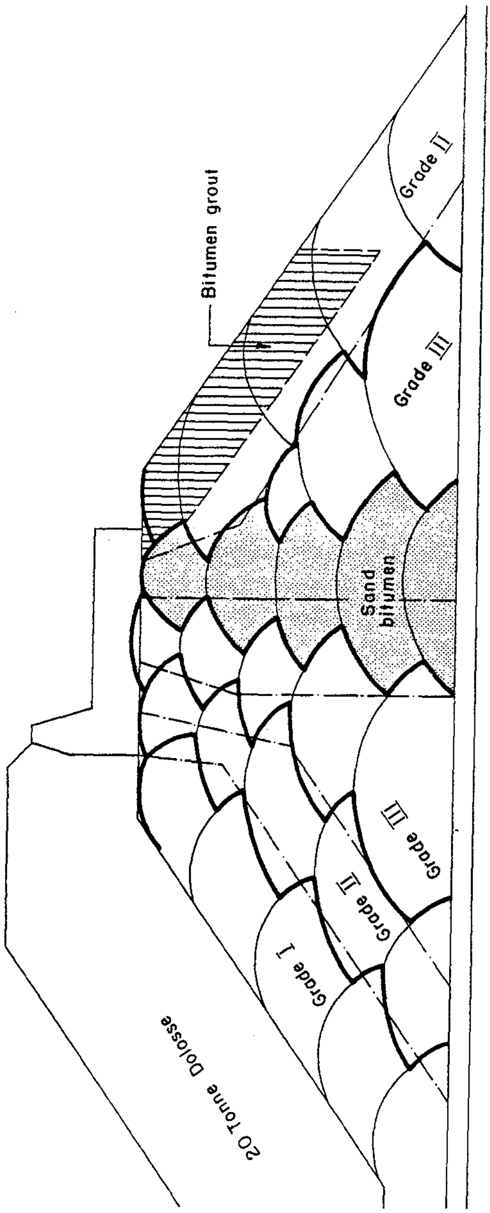

Figure 4. ACTUAL BREAKWATER BUILDING PROFILE

Figure 5. SAND BITUMEN BEING PLACED IN BREAKWATER

Figure 6. BOULDER SURROUNDED BY SAND BITUMEN

5. CONCLUSIONS

The use of sand bitumen in this instance as an acceptably impermeable core material has, so far, proved successful.

Its unit cost was higher on this occasion than the quarry run which would normally be used. However, it could find application as a general breakwater material in areas where rock is expensive. Because of high wave activity and sediment transport, the lower portion of the breakwater core became contaminated with sand before the bitumen could be placed.

On this project a net saving in the total cost of breakwater construction might be claimed as the presence of the sand bitumen core permitted grouting to the lee face, thus obviating the need for increasing its crest height and providing lee face armouring towards the seaward end. The extra cost of the sand bitumen and grout thus tends to be balanced by the savings in crest height armour units.

Care is necessary in the design of the armour layer as a breakwater with an impermeable core may not absorb as much wave energy as a conventional rubble mound and some of the wave energy within the breakwater may be reflected. The design should therefore be model tested.

6. ACKNOWLEDGEMENTS

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