CHAPTER 144

DESIGN OF MAIN BREAKWATER AT SINES HARBOUR

John Dorrington Mettam*

Introduction

In March 1972 the author's firm in association with two Portuguese firms of consulting engineers, Consulmar and Lusotecna, were appointed by the Portuguese Government agency Gabinete da Area de Sines to prepare designs for the construction of a new harbour at Sines on the west coast of Portugal. The location is shown in Figure 1.

The main breakwater, which is the subject of this paper, is probably the largest breakwater yet built, being 2 km long and in depths of water of up to 50 m. It is exposed to the North Atlantic and has been designed for a significant wave height of 11 m. Dolos units invented by Merrifield (ref. 1) form the main armour.

The project programme required that studies be first made of a wide range of alternative layouts for the harbour. After the client had decided on the layout to be adopted, documents were to be prepared to enable tenders for construction to be invited in January 1973. This allowed little time for the design to be developed and only one series of flume tests, using regular waves, was completed during this period. Further tests in the regular flume were completed during the tender period and a thorough programme of testing with irregular waves was commenced later in the year, continuing until August 1974 when the root of the breakwater was complete and the construction of the main cross-section was about to start.

The model tests, which were carried out at the Laboratorio Nacional de Engenharia Civil in Lisbon, were reported by Morais in a paper presented to the 14th International Coastal Engineering Conference in 1974. (ref. 2)

Layout

Sines was selected as the most suitable place on the whole Portuguese coastline for development of a maritime industrial complex served by a new deepwater harbour.

The first phase of the project was to be a new oil refinery with a harbour handling tankers up to 500,000 DWT. Later development would include provision for tankers up to 1,000,000 DWT as well as a wide range of other berths to serve other industries.

* Partner in the firm of Bertlin and Partners, Consulting Engineers of Redhill, Surrey, U.K.

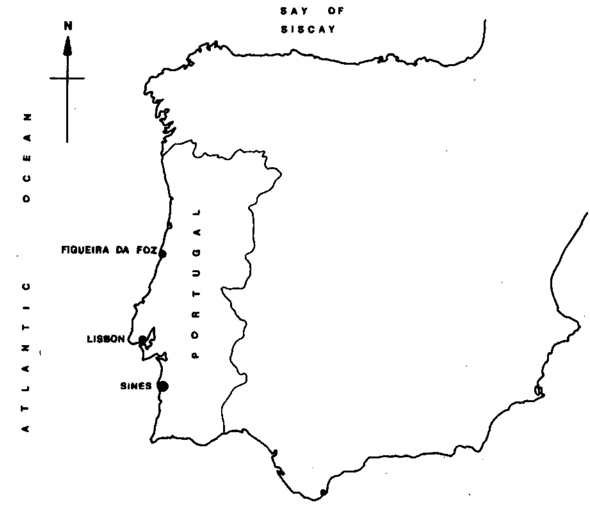


FIG. 1 SITE LOCATION

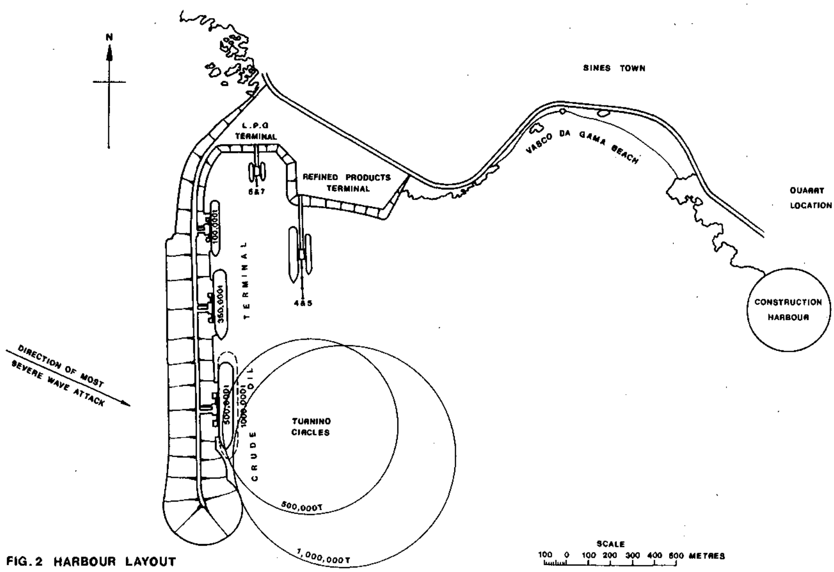


FIG. 2 HARBOUR LAYOUT

The layout of the first phase of the West Breakwater is shown in Fig. 2. Details of the adjacent berths are given in Table 1.

Upgrading of Berth 1 to handle ships of 1,000,000 DWT would require strengthening of the breasting dolphins, and some dredging to enlarge the turning circle.

Table 1 - Details of Berths 1 - 7

Berth No.	Commodity	Max. Ship Size (DWT)	Draft (m)	Depth at Berth (m. below chart datum)
1	Crude oil	500,000 (1,000,000)	28 (33)	39 (see note 1)
2	Crude oil	350,000	25	28
3	Crude/Products	100,000	15	18
4	Products)	45,000	12	19
5	Products)	(100,000)	(15)	(see note 2)
6	L.P.G.)	3,000	6	7
7	L.P.G.)			

Note

(1) It is envisaged that a second berth for 1,000,000 DWT could be provided by a southward extension of the breakwater.

(2) Berths upgraded to 100,000 DWT in Phase 2.

Other berths for dry bulk commodities, general cargo etcetera would form a separate complex adjacent to the South Breakwater, the precise layout of which has not yet been decided. Construction of this complex will be by stages to suit development of industries.

A construction harbour near the South Breakwater was included in the contract near the quarry site to assist the contractor to handle the enormous volume of rock required for breakwater construction. This will finally be used for harbour craft, fishing vessels and coastal shipping.

Design Wave Height

A wave recorder had been installed at Sines in September 1971, as soon as it had become clear that this site would be selected for the harbour. This however had only yielded records for one winter when design of the breakwater started, and for two winters when the final decision had to be made on the weight of armour unit. To supplement this record an analysis was made (ref. 3) to assess the probable wave climate at Sines from some earlier detailed observations made at Figueira da Foz (ref. 4) over a period of 7 years.

In view of the uncertainty regarding the best method of extrapolating extreme values from short period records four different methods were used. No method gave consistently better fit to the available data and no conclusion could be drawn regarding relative reliability. The method of calculation of waves at Sines from those at Figueira da Foz was considered likely to give slightly too high a value - because of the difficulty of allowing for the fact that Sines is further from the main storm areas. Equal weight was therefore given to the shorter period of records at Sines even though these gave a lower result. Table 2 gives the results of the various methods of extrapolation together with the value of significant wave height finally adopted.

Table 2 - Extrapolation of Extreme Waves (Significant Height)

Source	Method of Extrapolation	Return Period (years)			
		1	10	30	100
Figueira da Foz	Gauss	6.5	9.8		14.0
	Weibull	6.3	8.6		11.0
	Weibull Modified(ref. 5)	6.5	8.7		11.0
	Exponential	6.5	8.8		11.2
	Mean	6.5	8.9		11.8
Sines Waverider	Gauss	7.8	9.2		11.5
	Weibull	6.1	7.0		8.0
	Weibull Modified (ref. 5)	5.8	6.5		7.2
	Exponential	7.0	8.8		10.4
	Mean	6.7	7.9		9.3
Adopted Value for Design		6.5	8.5	9.5	11.0

For the irregular wave flume tests damage criteria were established on the lines proposed by Ouellet (ref. 6). These are shown in Table 3, which also includes overtopping criteria. The wave spectrum used was the Pierson - Moskowitz, based upon site storm records.

Table 3 - Breakwater Design Criteria

Storm Return Period (Years)	Significant Wave Height H_s(m)	Dolos Movement	Overtopping
1	6.5	Nil	Begins with H_{max} individual 10 - 11 m
10	8.5	Oscillation only	-
30	9.5	Beginning of displacement	Severe overtop with 15-16 m H_{max} individual
100	11.0	1% Damage	-

When testing with regular waves the same criteria were applied as far as practicable using wave periods corresponding to a fully risen sea. Wave heights and periods are given in Table 4.

Table 4 - Wave Periods for Regular Wave Flume

Height (m)	8	9	10	11	12	13	14	15
Period (sec.)	11.5	12.3	12.9	13.5	14.2	14.8	15.3	15.8

Development of Design for Main Cross-Section

In the initial design stages consideration was given to a range of possible solutions. The main choice was between rubble mound construction and composite construction with a vertical face structure founded on a rubble mound.

Some preliminary tests of the composite section showed that very large wave forces would be exerted on the vertical structure even if it was founded as deep as -20 m CD. Such forces could probably have been much reduced by the introduction of a perforated face construction. Even so, with the construction problems on such an exposed site it was decided that a rubble mound design would be more economical.

In this decision the availability of a suitable rock, in very large quantities, was of course a major consideration.

With the great depths at the site it was important to adopt a form of armouring which allowed the seaward face to be as steep as practicable, to reduce the volume of rubble core, and to restrict the reach required for the cranes placing heavy armour units.

Discussions with Mr. Merrifield convinced the designers that Dolos units would be the best solution currently available. Comparative studies of dolos and tetrapods during the first phase of regular wave flume tests confirmed this view. Attendance by the author at the 13th International Coastal Engineering Conference in Vancouver in 1972 was also most helpful and the author is indebted to the many people at that conference with experience in research on dolos, and in their use, who generously shared their knowledge.

In preparing breakwater designs it is the author's view that preliminary designs based upon past experience must always be fully tested by flume testing using the precise conditions of waves and topography for the site. This is particularly important when dolos are being used because classic design formulae such as Hudson's (refs. 7 & 8) developed initially for rock armouring, do not allow properly for the type of interlock displayed by these units which alters the importance of slope and specific gravity. Such formulae also omit the effect of wave period, which must be a major factor in determining the wave forces particularly at Sines where wave periods are longer than at many other sites.

For this reason the design was mainly developed by flume testing, first on the regular wave flume and later on the irregular wave flume. The initial design submitted to contractors for tendering, Fig. 3, was based on the first series of tests in a regular wave flume.

This design would in any case have been subject to change when further flume tests were completed. In addition the successful tenderer (Societa Italiana per Condotte d'Acqua S.p.A) submitted an alternative design which was accepted in principle by the Client subject to model testing, Fig. 4. The purpose of the change was twofold; to widen the breakwater crest to provide space for the oil pipelines which would otherwise have required a separate structure; and to provide more space for constructional equipment. This design was considerably modified after carrying out irregular flume tests, the final cross-section being shown in Fig. 5.

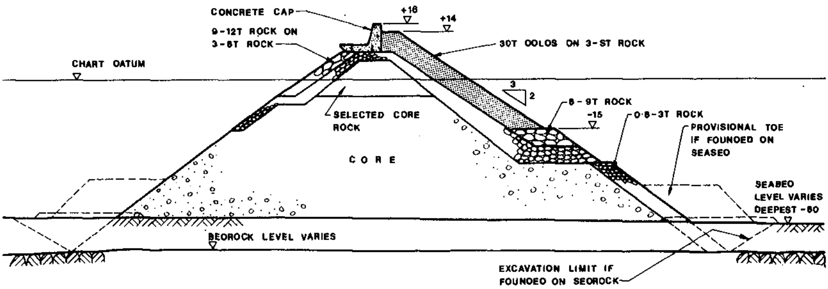


FIG. 3 TENDER DESIGN

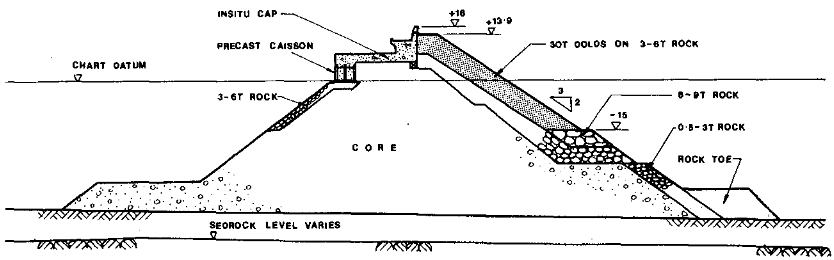


FIG. 4 CONTRACTOR'S ALTERNATIVE DESIGN

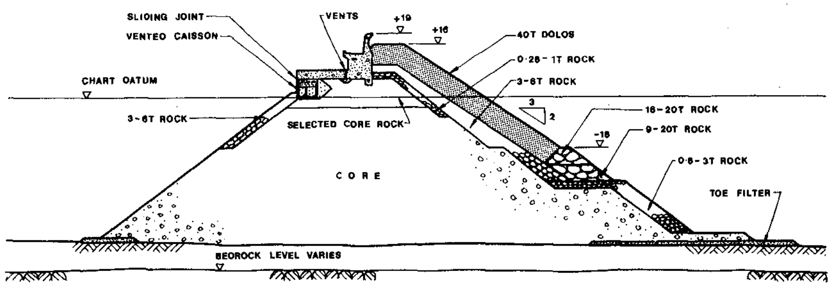
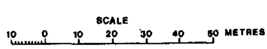


FIG. 5 FINAL DESIGN



The main changes resulting from the irregular wave tests were:-

1) Increased crest height on wave wall.

2) Adoption of curved profile wave wall.

3) Increased width of dolos armouring in front of wave wall.

4) Increased weight of capping and introduction of sliding joint at junction with caissons at back.

5) Introduction of heavier armour (16-20t) at toe of dolos slope.

The weight of dolos armouring, which had been changed from 30t to 40t nominal (actually 42t) as a result of the second series of regular wave tests was confirmed by the irregular flume tests.

The final design is discussed, section by section, below.

Foundation

The main length of the breakwater is founded, in depths varying from 30m to 50m, on granular deposits overlying shaley mudstone. The deposits consist of sand and gravel in thicknesses of up to 12m.

Site investigations completed after tenders were invited, showed that these deposits were sufficiently dense to support the breakwater. Calculations were made to check the stability under surcharge loadings of up to 60 m of rubble filling. Detailed consideration was also given to the possibility of liquefaction under earthquake conditions (ref. 9) and under the reversal of pressure resulting from severe wave attack. It was concluded that it would not be necessary to remove any bed material before placing the rubble mound.

To guard against scour a toe protection was provided consisting of a 1 m thick filter layer of 5 mm to 100 mm graded stone on which a 2 m thick layer of core material was placed. A narrower strip of filter material under the back slope of the breakwater guards against leaching of the foundation material through the toe of the rubble core.

Rubble Core

The main rubble core was obtained from a quarry sited close to the shore near the site of the construction harbour. Borings showed massive rock, with unbroken cores up to 3 m long. The rock, which has a specific gravity of 2.9, consists of gabbro and diorite.

Two grades of core material were specified. The principal grade complies with the following: "Core material shall not contain overburden or any clayey organic or other deleterious material. It shall consist of rock evenly graded from 1 kg to 3,000 kg. It may contain broken rock fines under 1 kg not exceeding 5% by weight. The quantity of rock under 10 kg in weight shall not exceed 15% of the total weight."

The 'selected' grade, used in the main cross section only above level -2.5 m CD and in the root of the breakwater is similar but the finer material under 10 kg is restricted to between 5% and 10% by weight.

Placing of most of the core material is by dumping from 1,000 t hopper barges. Up to -20 m CD there is no restriction on placing but above this level core construction is not allowed more than 50 m ahead of the secondary armour, which itself is not allowed more than 50 m ahead of the dolos. The purpose of this is to restrict the amount of work at risk during storms. (Fig. 6). The upper part of the core is tipped from 70 t lorries running on top of the core which is at level + 5.5 m CD (c.f. high water level +3.8 m CD)

Settlements are being recorded during construction and concreting of the cap is not allowed until 3 months after placing of the core by which time the main settlement has taken place.

The effect of earthquakes on the breakwater was checked using the analysis suggested by Newmark (ref. 10) and developed by Goodman and Seed (ref. 11). This analysis, which is a dynamic method of approach, is used to determine the displacement of the breakwater during the passage of the design earthquake.

Under an earthquake of intensity between 7 and 8 on the modified Mercalli scale, which is expected with a return period of 100 years, a settlement of about 0.5 m is expected.

Rock Armouring and Underlayers

Secondary armour of 3 t to 6 t rock is provided under the main armouring of dolos units. The size is determined partly by the size of the main armour- so that it cannot be drawn through the gaps in between the dolos - but also by the requirement that it should resist attack by 3m to 4m waves during construction.

This secondary armour should be able to retain the core material, after fines have been leached out of the surface of the core. However a tertiary layer of $\frac{1}{2}$ t to 1 t stone was added in the most vulnerable zone to reduce the amount of fines leached out of the core near the wavewall. This material can only withstand minor wave attack and must be placed just before the 3 t to 6 t stone.

Fig. 6 Breakwater under construction

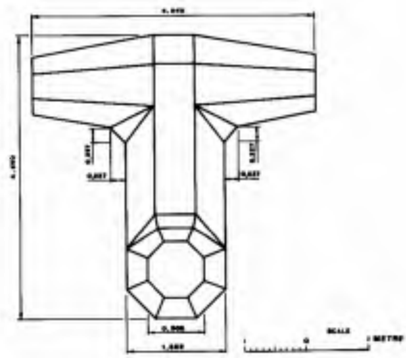


Fig. 7 40 tonne dolos

Fig. 8 Dolos moulds

At the toe of the dolos face irregular wave tests showed that draw down under severe wave attack caused damage with 6 t to 9 t rock as originally provided. Stone of 16 t to 20 t weight, supported on 9 t to 20 t rock proved sufficiently stable, but only with the revised shape shown in Fig. 5. This toe armour rests on 0.5 t to 3 t rock which also extends down the seaward face.

On the rear face armour is only provided to a depth of 15 m below low water. Rock of 3 t to 6 t weight proved adequate because overtopping water is thrown clear of the vertical back face of the cap and its energy is dissipated in water.

The 3 t to 6 t armour is also required to resist waves running along the back of the breakwater due to direct attack in southerly storms which are infrequent and diffracted waves from more westerly storms.

Main Dolos Armour

The main dolos armour units have a volume of 16.56 cu m and with a concrete specific gravity of 2.53 weigh approximately 42 t.

The dimensions, shown in Fig. 7, provide a waist: leg ratio of 0.35. Fillets at the junction of the legs have a dimension of 5% of the leg length. Tests by Lillevang (ref. 12) show that this size of fillet greatly reduces the stress concentration compared with a sharp corner but that a better stress pattern is obtained with a radius. When these results were made available to us the moulds had already been fabricated and units had been cast which showed little or no tendency for cracking at this corner, and it was decided not to alter the design.

Consideration was given to providing reinforcement but it was not done. The benefit from reinforcing this type of unit is very doubtful and the potential danger of damage due to corrosion of the steel is more serious. Instead it was decided to provide a high strength concrete - 400 kg/sq cm at 28 days - made with a low heat pozzolanic cement. The contractor's design for moulds, shown in Fig. 8, is considered a very good one. The arrangement of casting with the trunk vertical allowed concrete to be placed and vibrated carefully in the critical area at the corners between the legs and trunk. A compressible joint in the moulds reduced the tendency for shrinkage to put the trunk into tension.

Time did not permit photoelastic analysis of stresses in the dolos. Some comparisons were however made between concrete stresses (ignoring the notch effect of sharp corners) in these and earlier units. These are presented in Tables 5, 6, and 7 in terms of static and dynamic stresses under loads equal to the dolos unit weight.

Table 5 - Comparison of dimensions of Dolos units

Project	East London	Sines	Ratio $\frac{\text{Sines}}{\text{East London}}$
Weight	20 U.S. tons	41.9 metric tonnes	2.31
Volume (cu.m)	7.55	16.56	2.19
Overall Length (m)	3.693	4.540	1.23
Waist Thickness (m)	1.108	1.589	1.43
Waist Ratio	0.30	0.35	1.17

Table 6 - Comparison of maximum static tensile stress in Dolos units

Note (1) Applied load = self weight of 1 Dolos unit

(2) All stresses are in tonnes/sq.m.

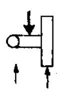
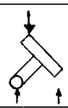
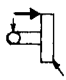
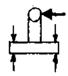
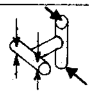
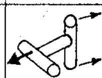
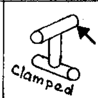
Load Case	1	2	3	4	5	6	7
Loading Condition							
East London Dolos	93 (100%)	80 (100%)	193 (100%)	243 (100%)	112 (100%)	205 (100%)	287 (100%)
Sines Dolos	84 (90%)	63 (80%)	186 (97%)	173 (71%)	104 (92%)	131 (64%)	200 (70%)

Table 7 - Relative dynamic stresses in Dolos units

Load Case	1	2	3	4	5	6	7
East London Dolos	100%	100%	100%	100%	100%	100%	100%
Sines Dolos	100%	89%	108%	79%	102%	71%	78%

The design of the armour face was based upon the recommendations of Zwamborn and Beute (ref. 13) and 155 units were specified per 1,000 sq. m of armour face. To ensure maximum interlock it was required that the complete face be built in a single operation rather than in separate layers.

This method has also been specified for placing rock armour (ref. 14) and is no more difficult than proper placing of separate layers. Hydraulic model tests showed that the number of dolos required could be placed in three separate layers but comparative tests confirmed at a very early stage that the specified method of placing in one operation gave a much more stable face. No comparisons were made with the thinner, two layer, armouring recommended by Waterways Experimental Station (ref. 8) but the author considers that detailed comparisons would show that the two layer system would require heavier dolos to give equal stability.

The specified method of placing is particularly advantageous in assisting interlocking on steep slopes, such as the 1 : $1\frac{1}{2}$ slope adopted at Sines, and it is considered that there would be little if any improvement in stability if a flatter slope were adopted because interlock during placing would be reduced.

Full records of damage during placing have not yet been made available to the designers, who are not responsible for site supervision, but it is reported that breakages of the order of 2-3% have occurred. Broken units are removed unless they are so interlocked that removal is impracticable. This rather high rate of damage no doubt stems from the use of floating cranes, already available to the contractor from another project, instead of cranes mounted on the breakwater. On such an exposed site this involves placing dolos with waves of up to 1 m and even so there is considerable down-time even during summer months.

Concrete Capping

The capping carries a road and pipelines. Its width reduces beyond each oil berth as fewer pipelines are required. To avoid damage to the pipelines the rubble mound is allowed to settle for 3 months before concreting commences. Despite this precaution the slab is heavily reinforced to resist stresses induced by settlement which may be expected to continue particularly during storms or earthquakes. Some damage to pipelines and minor structures during extreme storms and severe earthquake is of course acceptable provided that the main structure survives.

Fig. 9 shows the capping proposed by the contractor which model tests indicated would not be satisfactory, even after incorporating the vents shown in Fig. 10. The tests showed that it is important to have a sliding joint between the capping slab and the small caissons at the rear of the breakwater. Details are shown in Fig. 11. The difference in type of failure is illustrated in Figs. 12 and 13. Without a joint the cap tends to lift and rotate about the back corner of the caisson causing a local failure at the back of the rubble mound. With the joint the cap slides over the caisson and the cut off at the front of the cap tends to dig into the top of the rubble. Provided the cut off is sufficiently strongly reinforced, failure can only occur by a deep shear through the main body of the rubble mound.

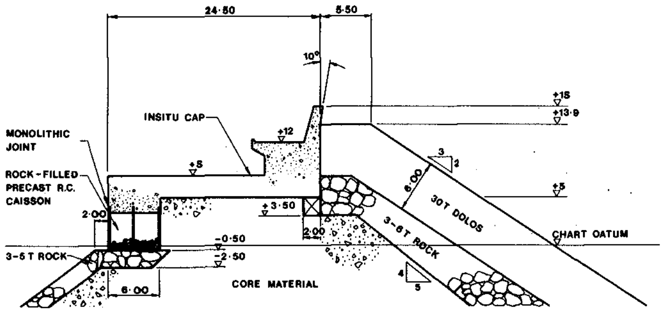


Fig. 9 Contractor's alternative capping design

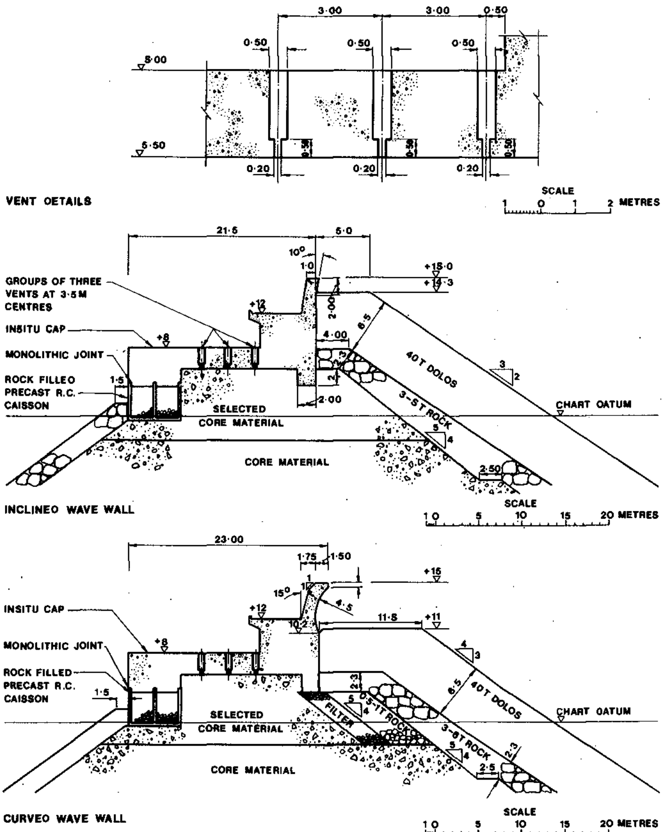


Fig. 10 Model test on contractor's alternative capping design

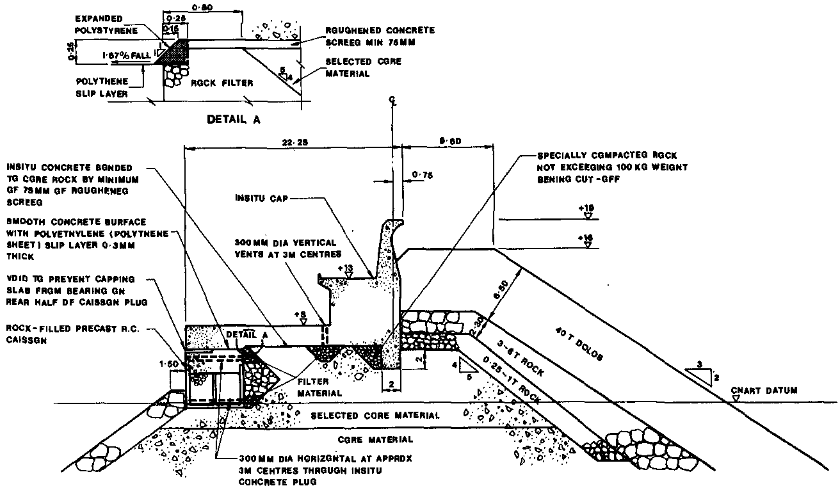


FIG. 11 FINAL DESIGN CAPPING DETAIL

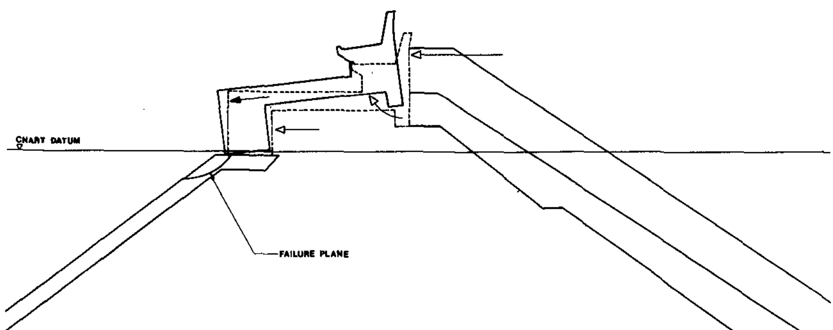


FIG. 12 MODE OF FAILURE – CONTRACTOR'S ALTERNATIVE CAPPING DESIGN

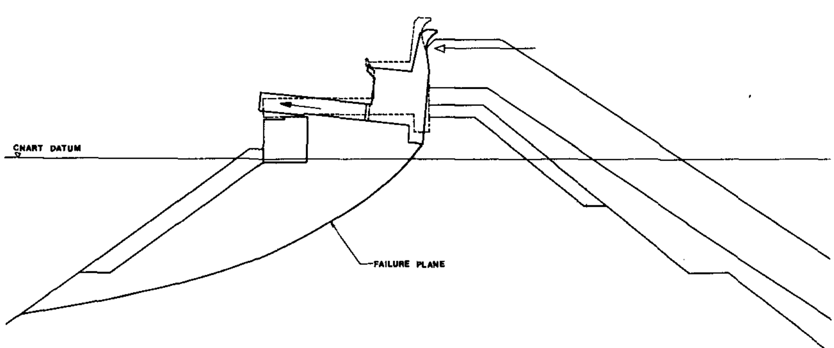


FIG. 13 MODE OF FAILURE – FINAL CAPPING DESIGN

Model tests on the alternative wave walls shown in Fig. 10 showed the value of curving the front of the wall, above the top of the dolos, to throw back the stream of water which comes over the dolos, thereby delaying the onset of overtopping. The proportions are based upon extensive research by Vera-Cruz at the LNEC hydraulic laboratory in Lisbon. Tests were made to determine the bending moments due to wave forces on the upper part of the wall which are of course very substantial. (Fig. 14)

The cap is cast in lengths of 15 m between joints. The total weight of each bay (excluding pipelines and the caisson below the sliding joint) ranges from 4,000 t to 5,000 t. The total wave force with the maximum individual wave in the 100 year storm is calculated to be 60 t/m, producing a bending moment at the base of the wave wall of 200 t f-m/m. Measurements on a model of the section of wave wall above road level gave a maximum prototype bending moment of 180 t f-m/m. Reinforcement in the cut off is designed to transfer a force of 160 t/m or 2,400 t/bay.

The vents in the cap, which are designed to minimise uplift pressures from water movements in the rubble core, are 300 mm diameter and are provided at 3 m centres. The bottom of each vent pipe is surrounded by a filter of 50 mm to 100 mm size rock to prevent leaching out of fine material. The vents have been observed to pass a mixture of air and water during a storm with maximum individual wave less than 10 m.

The joint between the capping slab and the small caisson is formed by a very careful steel float finish to the concrete which is covered by polythene sheeting to ensure a free sliding joint. Expanded polystyrene, as shown in Fig. 11, prevents any possible transfer of shear forces through stones being caught between the slab and the caisson. The sliding joint is formed only over half the top of the caisson. The other half, near the harbour, is covered with sand which is subsequently flushed out to leave a 100 mm gap. This is to ensure that vertical loads are applied to the caisson in a way which gives the most favourable distribution of stress on the foundation.

Root of Breakwater

The topography of the site is such that there is a clearly defined section of breakwater on shallow rock foundations before the bed drops steeply into deep water where the main cross-section, already described, is required.

The root section is not only protected by shallow water but also by a number of rocks to seaward so that waves are depth limited and tend also to be divided. The length closest in-shore is armoured with natural rock. Fig. 15 shows the design for a typical length of the root section in a depth of about 4 m at low water where the armour is 15 t dolos. It will be seen that the wave wall design is more closely related to the contractor's alternative cross-section (Fig. 4) for the main breakwater but the crest height and cut off arrangements were modified in light of the model test results for the main cross-section. No model tests

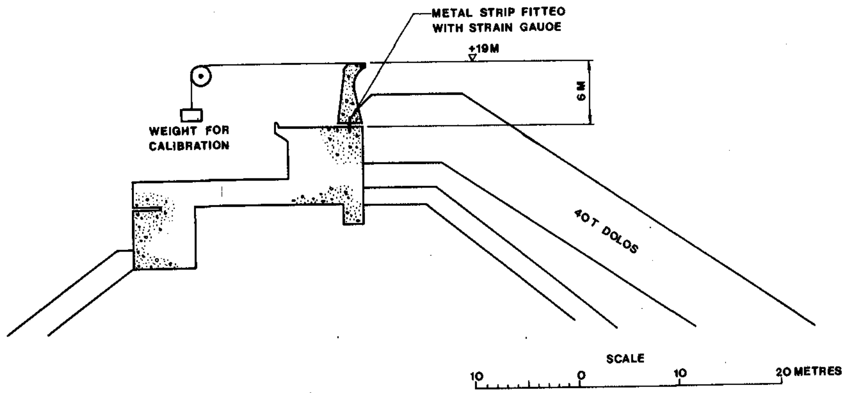


Fig. 14 Final design model study of forces on wave wall

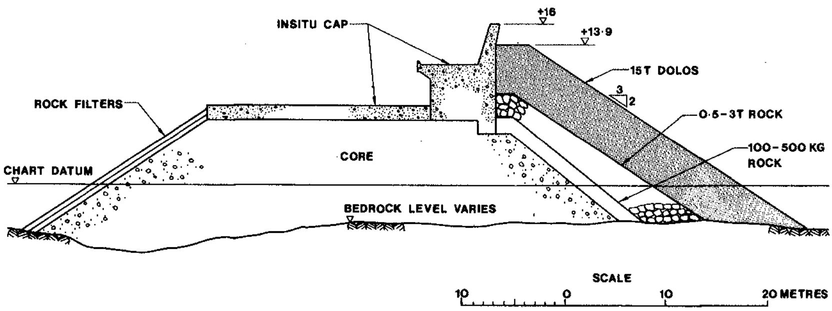


Fig. 15 Root of breakwater

were made for the root section because the broken nature of the sea bed makes any flume tests unrealistic.

Breakwater Head

The design of the head of the breakwater is still being studied on a wide flume at the LNEC in Lisbon. The original intention was to use a caisson head in the form of a vertical cross wall founded at about -24 m CD. This design would have had the advantage that future extension is very easy.

At this site it is necessary to design for earthquake forces and the worst wave attack is from WNW. Both factors require any caisson to be very wide to resist heavy forces from the shoreward side making this design more costly than a round head armoured with dolos.

Since the need for further extension, which would provide a second berth for ships of 1,000,000 DWT, appears to be very remote it has been decided to adopt a roundhead design.

Acknowledgements

This paper is produced with the kind permission of the Gabinete da Area de Sines which is gratefully acknowledged. Thanks are due to Mr. Berry of Bertlin and Partners who helped in the preparation of this paper. The author wishes to acknowledge the kind help of Mr. Merrifield who invented the dolos unit and generously made it freely available.

He also thanks many others interested in research on the behaviour of dolos units and in their use who have exchanged information privately and through papers and discussions at the 13th and 14th International Coastal Engineering Conferences.

References

1.	Merrifield, E.M.	"Dolos - a new breakwater and coastal protection block". The Dock and Harbour Authority, April 1970.
2.	Morais, C. Campo	"Irregular wave attack on a Dolos breakwater" 14th Conference on Coastal Engineering, Copenhagen, 1974.
3.	Castonho, J.P. Carvalho and Vera-Cruz	"Estudo da agitacao maritima no futuro porto de Sines" Lisbon 1972.
4.	Carvalho, J.R. and Barcelo	"Agitacao maritima na costa oeste de Portugal Metropolitano" Memoria no. 290 do Laboratorio Nacional de Engenharia Civil, Lisbon, Portugal, 1966.
5.	Battjes, J. A.	"Long term wave distribution at seven stations around the British Isles". N.I.O. Internal Report No. A.44, July 1970.
6.	Ouellet, Y.	"Considerations on factors in breakwater model tests" 13th Conference on Coastal Engineering, Vancouver, 1972.
7.	Hudson, R.Y.	"Laboratory investigation of rubble mound breakwaters" Proc. A.S.C.E., Waterways and Harbours Division, Sept. 1959.
8.	Hudson, R.Y.	"Concrete armor units for protection against wave attack - Report of ad hoc committee on artificial armor units for coastal structures" U.S. Army Engineer Waterways Experimental Station, Vicksburg, 1974.
9.	Bjerrum, L.	"Geotechnical problems involved in foundations of structures in the North Sea" Geotechnique 23, No. 3, 1973.
10.	Newmark, N.M.	"Effects of earthquakes on dams and embankments" 5th Rankine Lecture, Geotechnique 15, No. 2, 1965.
11.	Goodman, R.E. and Seed	"Earthquake-induced displacements in sand embankments". Proc. A.S.C.E. Vol. 92, SM2, March 1966.

12. Lillevang, O "Experimental studies of stresses within the breakwater armor piece "Dolos"." 15th Conference on Coastal Engineering, Honolulu, 1976 and private communications.

13. Zwamborn, J.A. and Beute "Stability of dolos armour units." ECOR Symposium, Stellenbosch, 1972.

14. Mettam, J.D. "Larnaca : new port for Cyprus." The Dock and Harbour Authority, June, 1969.