PRAIA DA VITÓRIA HARBOUR (AZORES) DAMAGES IN THE BREAKWATER DUE TO THE STORM OF 26th - 27th DECEMBER, 1962

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1 - INTRODUCTION

Praia da Vitória is a harbour in the eastern coast of Terceira, in Azores Islands. These lie in the Nortern Atlantic, between 36° 55' and 39° 43' north latitude and between 26° 46' and 31° 16' west of Greenwich longitude. Terceira island belongs to the central group of islands (fig.1).

The port facilities in Praia da Vitória harbour were constructed with the main purpose of enabling the unloading of liquid fuels, however intense the swell at the entrance of the harbour.

The maritime structures include a rubble mound breakwater about 600 m long, rooted at Ponta do Espírito Santo, and an unloading system, independent from the breakwater and made up of a central hose-handling pier, where tankers can berth, and dolphins to take the mooring lines of the ships. The hose-handling platform and the dolphins are connected with one another and with the shore by means of footbridges (figs. 1 and 3).

2 - DESCRIPTION OF THE BREAKWATER

Given the location of Praia da Vitória harbour in the eastern coast of Terceira and the contour of the island in this zone, the breakwater is free from considerable storm actions, save for NE, E and SE storms. Notice that for the latter direction, S.Miguel and Santa Maria islands restrict the generation of swell, reducing the fetch to a maximum of 120 sea miles. Due to this and to the bottom relief conditions at the southern and the harbour, the breakwater is not subjected to violent actions during SE storms (for a wind of 40 knots, the significant wave height in front of the breakwater does not exceed 4.00 m).

On the other hand, the bottom relief conditions in front of Ponta de Má Merenda, in the northern area of the harbour give rise to a marked energy dispersion for NE storm and consequently wave height near the breakwater do not exceed 65% o

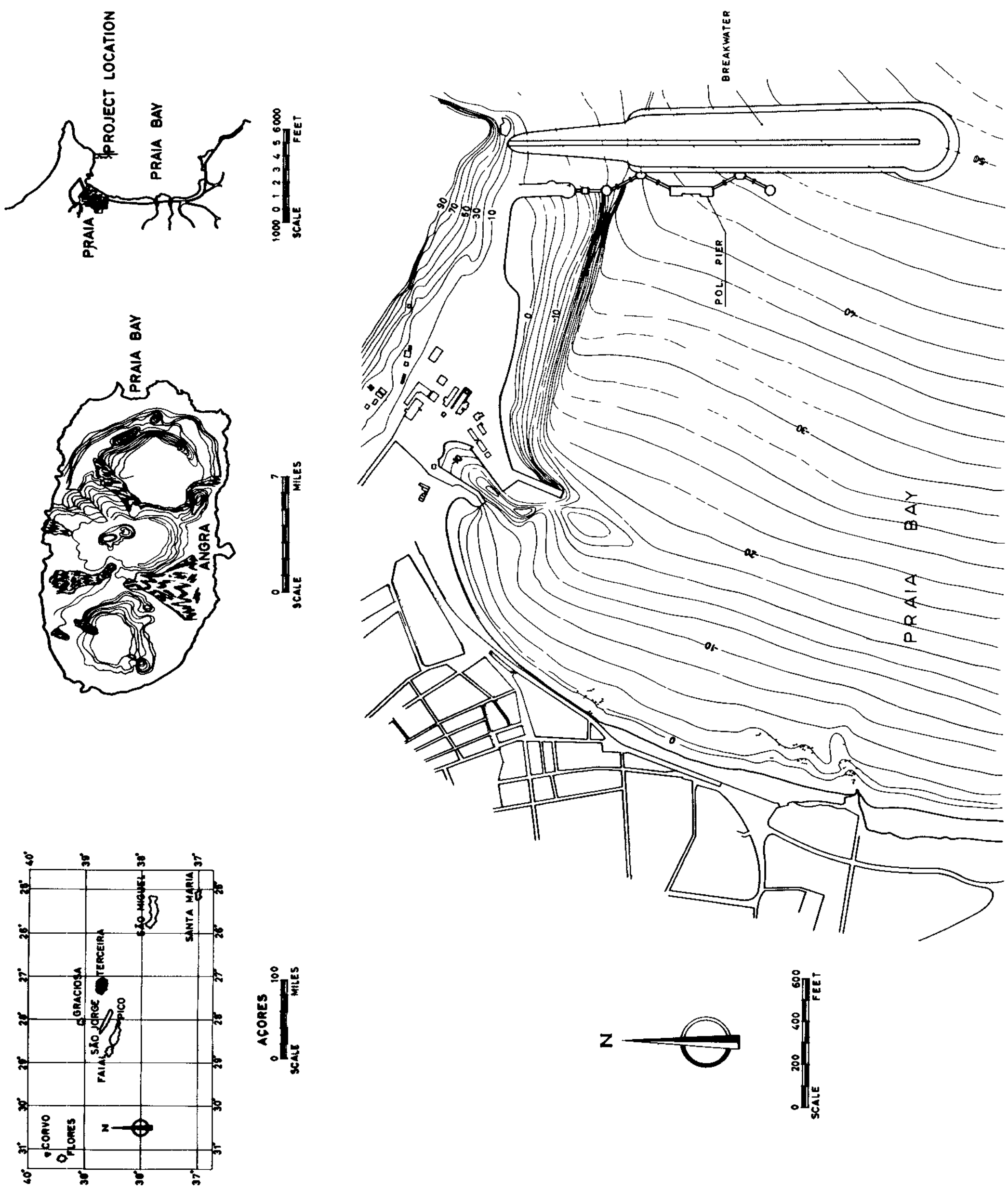
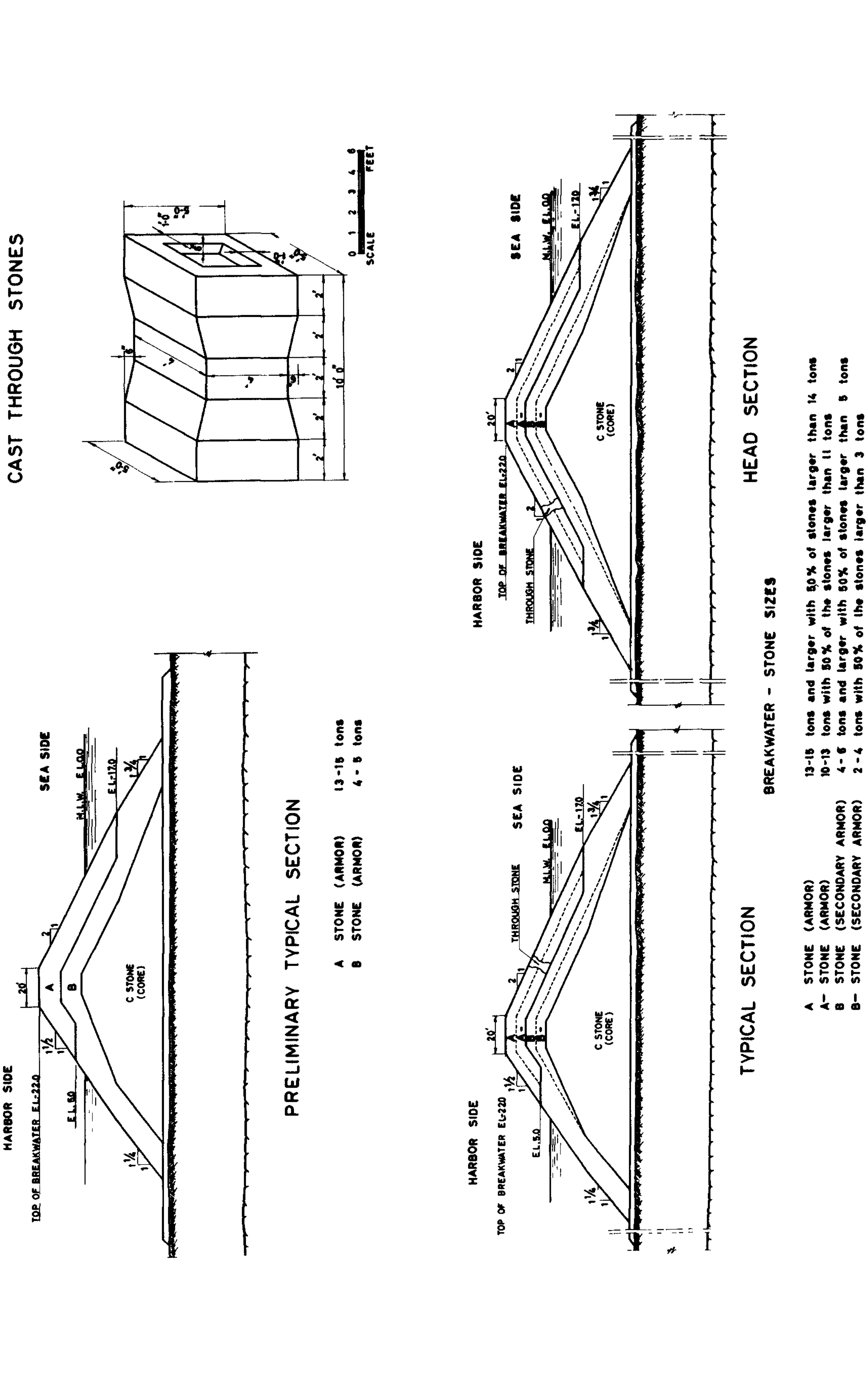


FIG. 1. PRAIA DA VITORIA HARBOR: GENERAL ARRANGEMENT


CAST THROUGH STONES
SCALE
FEET
10' 0"
PRELIMINARY TYPICAL SECTION
HARBOR SIDE
SEA SIDE
TOP OF BREAKWATER EL+22.0
EL.6.0
M.L.W. EL.0.0
EL.-17.0
C STONE (CORE)
A STONE (ARMOR) 13-15 tons
B STONE (ARMOR) 4-5 tons
TYPICAL SECTION
HEAD SECTION
THROUGH STONE
EL.5.0
BREAKWATER - STONE SIZES
A STONE (ARMOR) 13-15 tons and larger with 50% of stones larger than 14 tons
A- STONE (ARMOR) 10-13 tons with 50% of the stones larger than 11 tons
B STONE (SECONDARY ARMOR) 4-6 tons and larger with 50% of stones larger than 5 tons
B- STONE (SECONDARY ARMOR) 2-4 tons with 50% of the stones larger than 3 tons

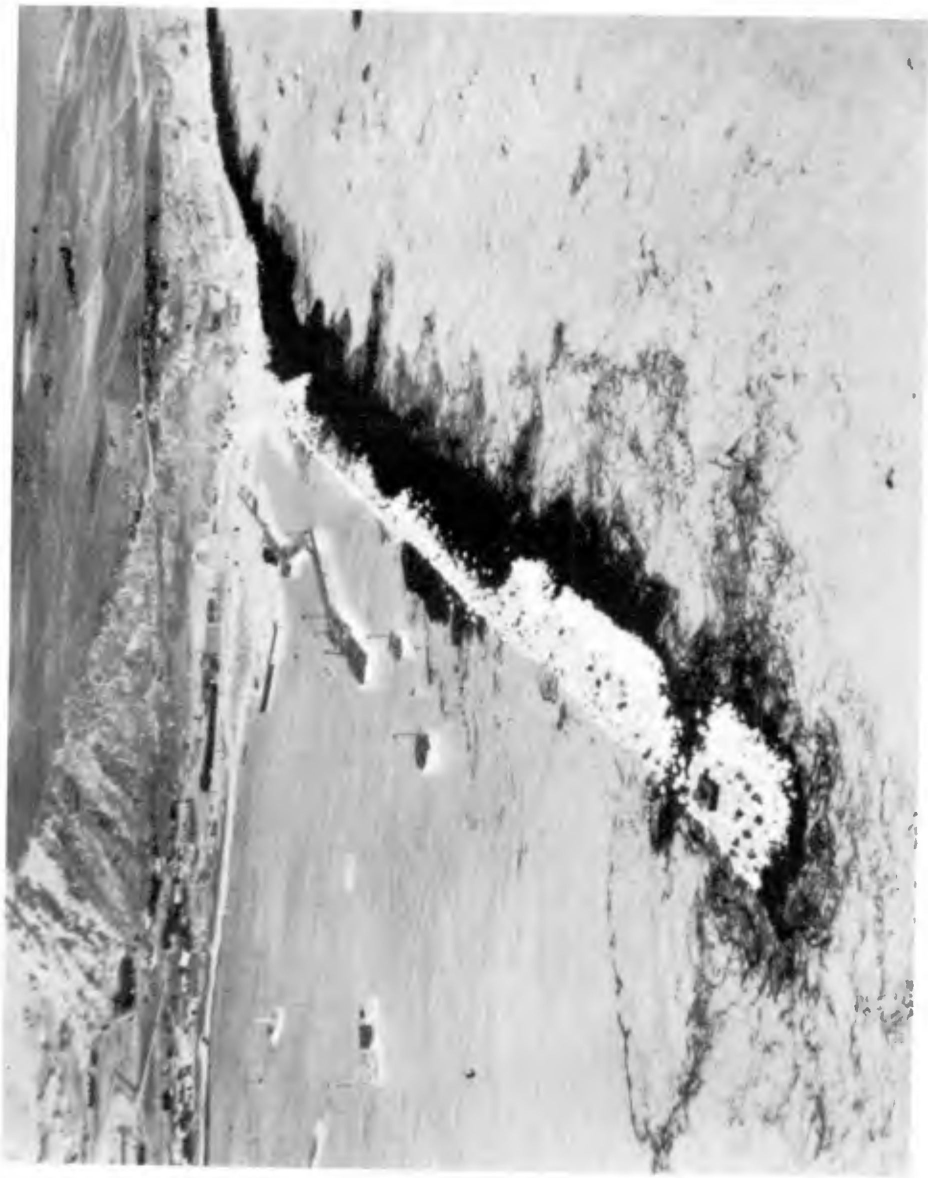


FIG. 3. GENERAL VIEW OF HARBOR

their height offshore.

Thus only storms with about easterly directions offshore can hit the breakwater with full violence.

When the breakwater was being designed, the maximum wave height to be considered was chosen from data from the "Sea and Swell Charts-North Atlantic Ocean", from wind speeds and wind directions recorded at the meteorological post of Lajes airport (Terceira Island) and on values obtained in local observations of swell.

A comparative analysis of the two first sources showed that eastern winds blow during about 20% of the total time with a velocity below 27 knots.

It should be noted at once that, eastern storms being due to eastern winds blowing in the Atlantic area between Azores and the European Continent, it was in this area that the maximum speed of the wind and the extent on which its action could be felt had to be known.

Local observations recorded wave heights of 5.50m during an E storm in November 1955 and waves with estimated heights between 5.50 and 6.00m in February 1956. Nevertheless, these values were not taken into account when the design wave characteristics were chosen. As far as is known the wave-heights in Praia da Vitória harbour in these dates were not checked with values obtained from the synoptic charts.

The wave height for calculating the breakwater was determined by the Sverdrup-Munk method, by means of the charts revised by Bretschneider, for a wind velocity of 27 knots. In these conditions the significant height is about 5.20m and this value was adopted as the maximum for storms with directions between NE and SE offshore. The period arbitrated for this swell was 12 seconds.

3 - PROFILE OF THE BREAKWATER

The profile designed on basis of this maximum wave height consists of a type C stone core (up to 8 000 lbs), covered with B (4-6 short tons) and A (13-15 short tons) armour stones in the zones under the direct action of the waves (fig.2).

It being difficult to obtain in a quarry the required percent age of A armour stones, the limits between the different stone types were changed so that 10-13 short ton stones were used in the lower layers and 2-4 short ton stones in the secondary armour of the harbour-side slop (fig.2). The rockfill has a specific gravity of 174.3 lbs/cuf (2.79 ton/m^3).

Hudson's formula applied at the profile for a wave-height of 5.20m yielded

$$K = \frac{\gamma_r \; H^3}{W \; (S_r - 1)^3 \cot \alpha} = \frac{2.79 \times 5.20^3}{12.7 \times 1.72^3 \times 2} = 3.00$$

This is less than the value recommended by Hudson, which apparently indicates that the pair of values angle of slope-weight of armour stones was fixed with a certain margin of safety.

Nevertheless the subdivision of armour stone A in two sizes (A, 13-15 and A-,10-13 short tons) decreased the safety of the structure as the loss of blocks in the sea-side layer for wave heights above the design values, which is always possible and actually occurred latter on, exposed the lighter layers to the direct action of the waves. For these layers the coefficient K has a value

$$K = \frac{3.0 \times 12.7}{10.43} = 3.65$$

which exceeds the figures recommended by Hudson.

Therefore, due to the changes in armour stone A, the structure had an insufficient margin of safety and a sufficiently prolonged storm, severer than the storm considered in the design, could give rise to the development of a chain destruction phenomenon with serious results, as each layer successively exposed to the action of the sea would be less stable than the preceding one and consequently would be more easily destroyed. This is what actually happened late in December 1962.

In an attempt to obviate the disadvantages of the subdivision of the A armour stone size, it is recommended in the design of the breakwater to use special shaped artificial blocks, cast through stones, placed so as to be included in two armour stone layers. These cast through stones, weighing about 16 short tons, were placed following no special rules, obeying one principle alone, that at least one of every ten A stones placed on the slope in the A stone surface layer must be a through stone.

Notice that, as regards the stability of the slope, the weight of cast through stones exceeds but slightly the weight of the surface layer blocks so that differences between the individual stability of one or other of these stone sizes is not very marked. On the other hand, as each artificial block is surrounded by natural rockfill the stability of the whole is that of its least stable component, notably as the percentage of cast through stones is very small. Their presence therefore contributes but little or nothing at all to the stability of the slope. That was also, in fact, the conclusion drawn from the model tests carried out to investigate the damages in the breakwater.

Three other characteristics of the profile of the breakwater are noteworthy. The first concerns the bottom elevation of the primary cover layer, which in the present case lies 17,0 f below the M.L.W. As in the low water of spring tides the water level can drop about 1,0 m below the M.L.W., the adopted level or (-17!0) does not strictly obey the current requirements and this could lead to suspect that the collapse of the breakwater could have started just in the zone between A and B armour stones The model tests showed that this was not the case however. Additionally, according to Per Anders Hedar's tests, damages in a rubble mound breakwater with a 2/1 slope never occur below an elevation of 0.8 H below S.L.W which confirms the results of the model tests.

In the harbour side the A armour stone cover extends down to level (+5!0), the B armour stone layer being therefore exposed for all the tide levels save the high water spring tide. This solution is not currently adopted when overtopping is possible.

Technical Report nº 4 "Shore Protection Planning and Design" recommends the use of a primary cover layer extending down to the minimum S.W.L. in the harbour side.

It was in the head of the breakwater, however, that the structure deviated more considerable from the currently followed rules. In fact it is generally known that the components of the head of a rubble mound breakwater, as that of Praia da Vitória, are subjected to more intense actions, than the components of the profile, due to the additional effect of the jet generated by the wave breaking in this zone. On the other hand, due to the curvature of the head of the breakwater, the components of the internal sector of the head are less stable than the components of the remaining sections. Due to the joint action of these effects, heavier elements are required in the head of breakwaters in special inside and between the minimum and the maximum s.w.l. In the case of rubble mound breakwaters, Technical Report nº 4 recommends an increase of weight of about 10% in the head zone.

Hence the conclusion that the head of the breakwater of Praia da Vitória harbour had not the same factor of safety as the other sections. Nevertheless, for the reasons indicated below, the head of the breakwater underwent but slight damages in the internal sector. It should be noted, however, that a decreased strength in the head of a breakwater is extremely dangerous, as any damage in that zone can easily extend to the remaining sections even if these by themselves could resist the storm.

4 - STORM OF 26-27th DECEMBER 1962

Late in December 1962, Azores islands were struck by a violent storm. According to meteorologic data (fig.4) supplied by Serviço Meteorológico Nacional (the Portuguese Weather Bureau), a long depression with its centre over Santa Maria island after the 00.000 hours of 25th December 1962 gave rise to strong east winds in its northern margin. This depression remained in approximately the same position, sometimes with more than one isallobaric nucleus, until after the 27th, although with a reduced influence thereafter.

During period of most marked influence, the depression valley extended from Azores to North Africa, producing east winds of 25-40 knots between the Portuguese coast and Azores.

These particularly unfavourable conditions gave rise to a sufficiently extensive fetch and a duration of the wind producing maximum swell for wind speeds below 40 knots.

Unfortunately the meteorologic records available did not enable to obtain a perfect definition of the field of wind velocities in the fetch. It was only possible to conclude that the velocity of the wind causing the storm was comprised between 30 and 34 knots which, according to Neumann's graphs, amounts to a significant wave-height that could have ranged between 6.50m (30 knots) and 9.00m (34 knots). It should be noted, nevertheless, that the velocity of the east wind, as recorded at the Lages meteorologic post (Terceira Island), did not exceed 25 knots.

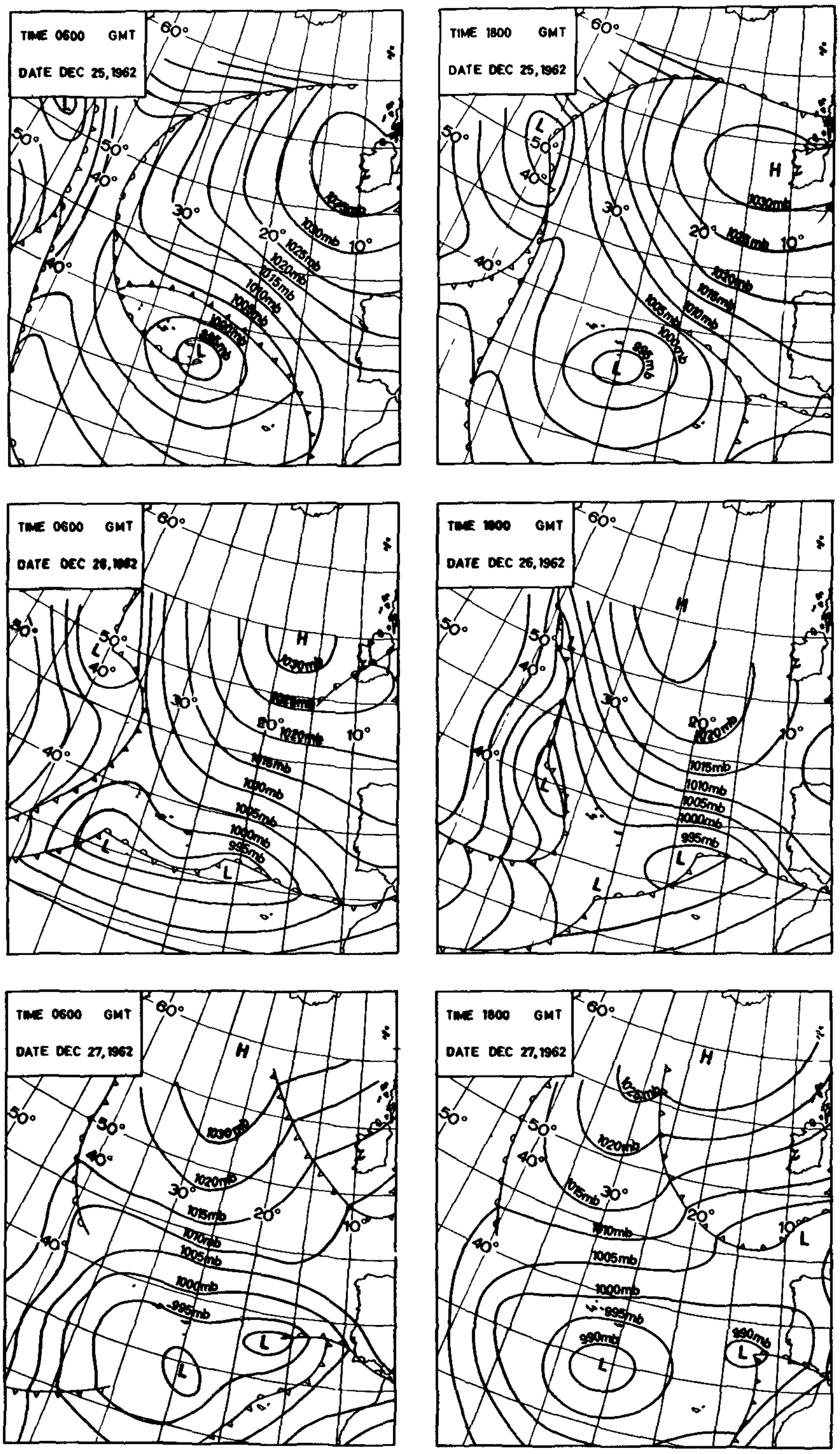


FIG 4 SYNOPTIC MAPS FOR NORTH ATLANTIC OCEAN·
STORM OF DEC 25-27, 1962

According to data supplied by Junta Autónoma do Porto de Angra do Heroismo (Angra do Heroismo Port Authority) (Terceira Island), the breakwater of Praia da Vitória was continously overtopped between the afternoon of 25th December and the morning of the 28th, notably during the high water. In these periods the spray due to the waves overtopping the breakwater reached beyond the tanker-berthing structure itself.

The swell height (amplitude) on the 26th and 27th were calculated by a quick method. The values obtained, average of records during periods of 30-40 minutes, were the following:

Date	Hours (G.M.T)	H meters	Date	Hours (G.M.T)	H meters
12.26th.62	10.00	≃ 8.00	12.27th.62	10.00	≃ 8.70
"	14.00	≃ 8.20	"	15.00	≃ 8.00
"	17.00	≃ 8.70	"	17.00	≃ 8.70

According to the records, individual wave heights did not exceed these mean values by more than 0.90m in each period of observation.

From these values and an analysis, as detailed as possible, of the fields of velocities of the winds blowing in the North Atlantic, it can be concluded that the storm that hit Praia da Vitória should have begun at 00.00 hours of 25th December (G.M.T.). At 12.00 (G.M.T.), the significant height was about 3.00 m, increasing to 6.00 m at 00.00 cours (G.M.T.) of the 26th December. The storm meanwhile grew more violent in this same day, reaching probably its maximum intensity about the 18.00 hours (G.M.T.). Thereafter, swell remained practically constant until about the 18.00 hours (G.M.T.) of the 27th, decreasing then very fast. The period corresponding to the storm was 13 to 14 seconds, in the periods of most violent storm.

5 - DAMAGES. THEIR ANALYSIS

At the date of the storm the breakwater was practically completed save for armour stone A which had still to be placed between elevations (-12!00) and (-22!0), between the root and the profile 175 metres.

The first signs of damage were observed on the 26th in the morning, consisting in the disappearance of some stones in the submerged zone of profile 335 metres, where armour stones A were being removed and rolled along the profile, disappearing under the water. Surveys carried out after the storm located them at base of the breakwater.

A preliminary conclusion can be drawn from these data. The damages were due to insufficient stability of the sea-side slope and not to overtopping. This would have produced a collapse starting with displacements of stones in the harbour-side slope.

On the night of 26th to 27th, when the storm reached its maximum intensity, the breakwater was particularly hit and damaged. The damages extended to the whole structure and, on the 27th in the morning, a deep breach was visible just near the head, where the storm had ruined the whole profile above elevation (-0.00), attacking even C stones (core).

On the 27th the destruction of the breakwater went on, to such an extent that the overtopping waves, in special in high water, endangered the berthing structure itself, already damaged in the preceding night. Happily the storm abated on the night of 27th to 28th so that no new destructions were observed on the 28th.

The damages undergone by the breakwater during the storm can be summed up as follows (fig.3,5,6,7,8):

- profiles 0 to 176 metres: this section of the break-water, completed up to elevation (-12'.0), underwent but slight damages at the surface; on the other hand both the sea-side and the harbour-side slopes were covered by a considerable volume of small stones, removed from the coast north of the breakwater;

- profiles 176 to 291 metres: this section remained in good conditions, as the only damages observed were some A armour stones removed from the sea-side slope and some slight settlements at the top; nevertheless, several B armour stones were displaced from the harbour-side slope below elevation (-5'.00),

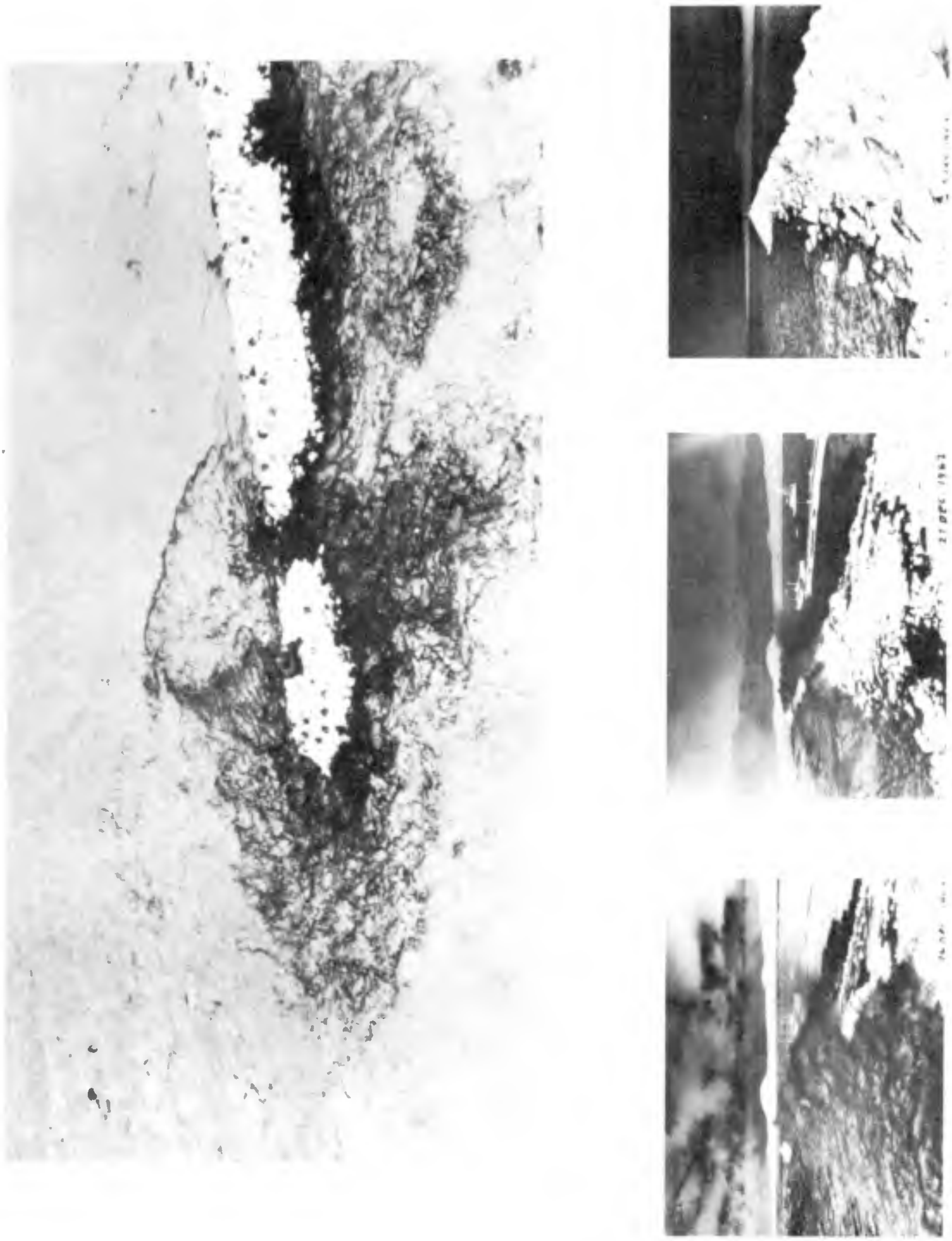


FIG. 5. VIEWS OF THE DAMAGES

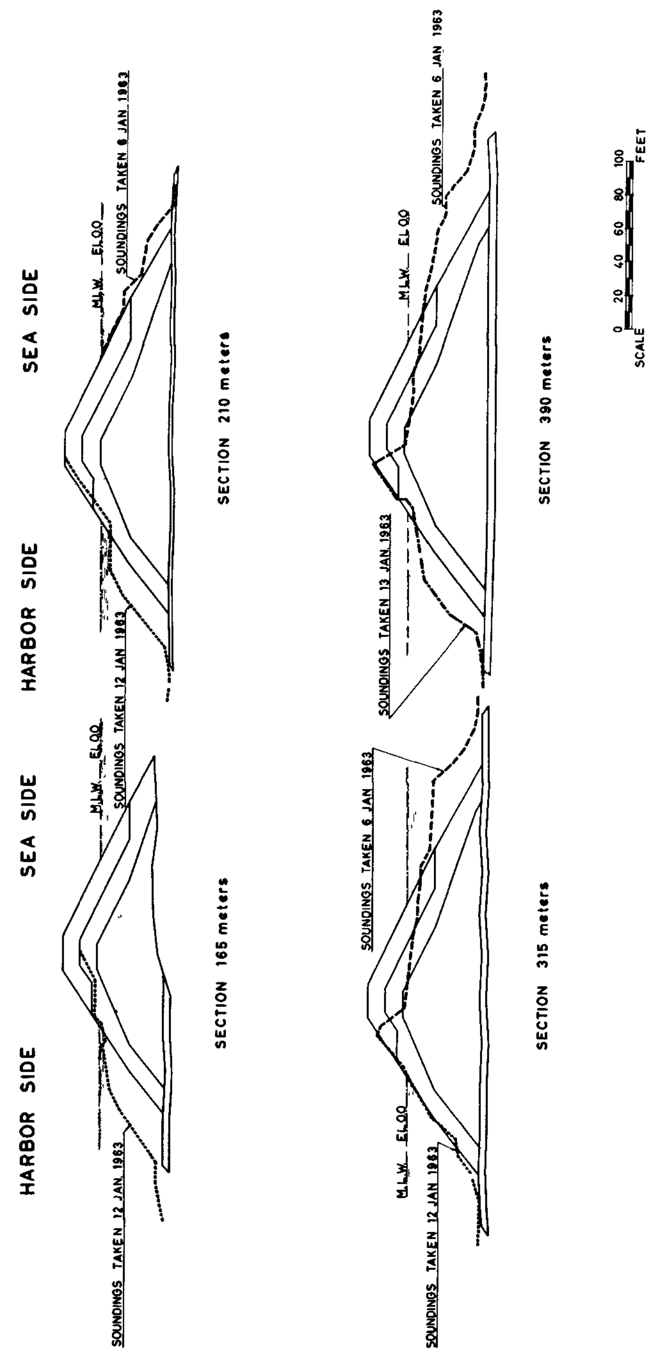


FIG 6 BREAKWATER SECTIONS AFTER STORM

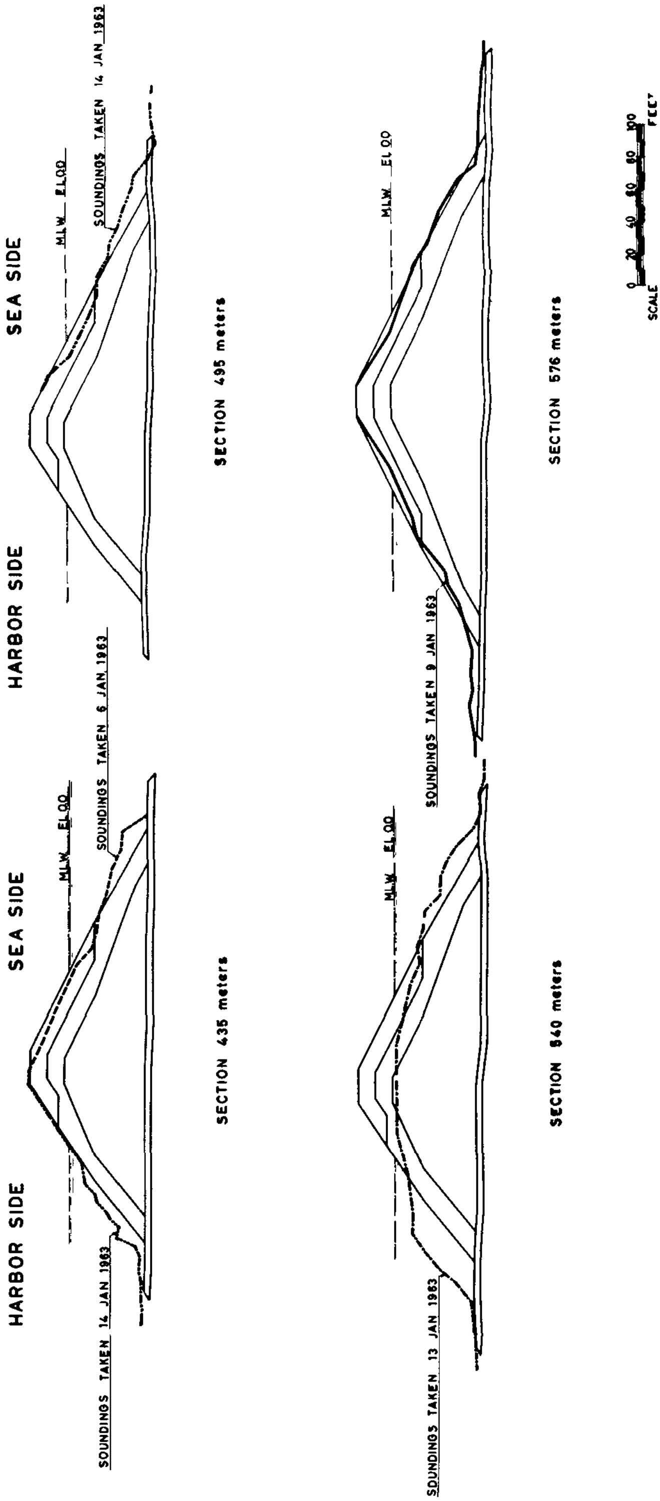


FIG 7 BREAKWATER SECTIONS AFTER STORM

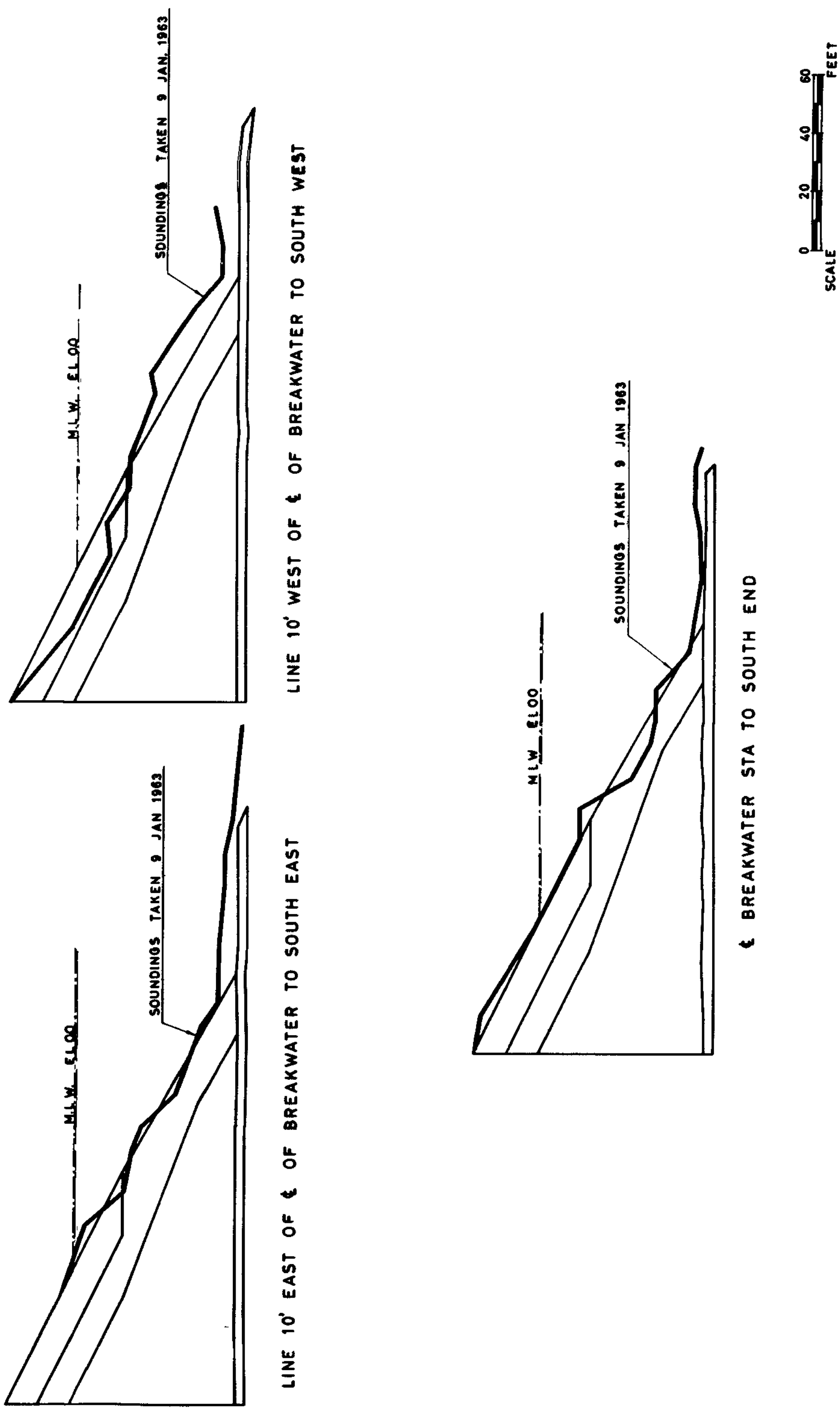


FIG 8 BREAKWATER SECTIONS AFTER STORM

which shows that the overtopping waves had harmful effects on this section;

- profiles 291 to 442 metres: this was one of the most severely hit sections, all the A armour stones of the sea-side slope and the top having been removed, rolling over the sea-side slope to the base of the breakwater, together with the B armour stones placed below; nevertheless, some A armour stones and cast through stones remained in place, although in very precarious equilibrium, in the harbour-side slope: if the storm had persisted somewhat longer, these blocks would also have collapsed and this section of the breakwater would have been razed to a level of about (- 0'.00); the type of damage undergone by this section of the breakwater confirms the observations of the preceding sections, showing that the collapse started in the sea-side slope;

- profiles 442 to 530 metres: this section was less damaged than the former as a length of about 45 metres remained almost intact;

- profiles 530 to 565 metres, this was the section where the most severe damages were observed: the breakwater was razed to elevation (- 0.00);

- profiles 565 to 585 metres (head of the breakwater) : this section remained in good conditions as only some stones were removed in the harbour-side slope below the water level, thus confirming our present knowledge on collapse phenomena in the heads of breakwaters.

The first conclusion to be drawn from the preceding analysis of the damages observed along the breakwater, is their extremely irregular distribution: the head remained practically intact, the adjoining section presents a breach, then a length of about 90 metres underwent only slight damages, but just beyond the breakwater was severely hit in an extent of 150 metres.

This extreme irregularity had the advantage, however, of enabling a reconstruction of the evolution of collapse in the visible portion of the breakwater, clearly evinced in the variable extent of the damages indicated above: at first A armour stones were removed, rolling down along the slope; the B armour stone layer thus remained exposed, stones being then also removed to the basis of the slope; finally after, having also removed some core stones, the waves pushed inside the few remaining blocks still in position in the harbour-side slope, producing a breach in the breakwater similar to the one near the head. It was impossible, however, from the surveys carried out in January 1963 to reconstruct the development of the collapse in the submerged zones, as the stones removed from the upper portion of the breakwater were concentrated at the base.

In the harbour-side slope, overtopping waves displaced some B armour stones alone.

Two factors can have caused this irregular distribution of damages: marked changes of the wave height along the breakwater or variable construction details from zone to zone, evinced by a storm more violent than the one considered in the design.

According to the two wave patterns drawn, one along an eastern directtion offshore and the other with an E-10°-N direction, the sea attack was frontal in the former case, with the following variation of wave-heights along the breakwater: a slight concentration near the root, followed by a slight decrease towards the head, where a marked local decrease is observed notwithstanding a slight concentration of energy in the just preceding section.

For the latter direction, the angle of the sea attack with the structure was small, without any apparent variation of the wave height along the breakwater. The analysis of the (fig. 5) photographs taken during the storm shows that the attack was always practically frontal.

The variation of the wave-height along the breakwater for an east wave offshore explains the absence of damage in the head and the breach in the adjoining profile, but it cannot account for the conditions observed in the remaining portion of the structure, where severely damaged sections alternate with zones practically intact. Apart from the fact that the breakwater was hit by waves higher than the design values and for a long time, these differences have apparently to be ascribed to different constructional methods alone.

6- MODEL TESTS

With a view to clearing up some points of the collapse of the breakwater, it was deemed of great interest to perform some model tests.

These were carried out in a wave channel so that only the case of the standard profile under frontal sea attacks could be considered.

The tests in a 1/50 model concerned first the high water and the low water levels without variations in level and then variations in level alone so as to reproduce local tide conditions.

According to the results of the tests, a reproduction even discontinuous of the tide produces more damages for the same wave heights than one water level alone. That is why the last tests comprised a reproduction of the tide for different values of the wave height.

All these tests showed that the first damages occured for wave-heights of about 6 metres, which confirms that the breakwater was well designed for the wave-heights adopted.

The first damages occurred between the high water and the low water levels, the stones rolling to the base of the slope. Then, as the wave height increased, the damaged zone extended progressively both towards the top and the base. Notice, however, that B armour stones below elevation (-17'.0) were never displaced, although for the highest waves this zone was already covered with A armour stones removed from the upper portion of the brea kwater.

The top of the breakwater was not damaged until the wave height reached 7.50 metres.

It follows that the maximum wave-height during the storm should have approximately this value, so that the tests were continued under this wave height for a period of time corresponding to the time during which the most unfavorable swell conditions prevailed in Praia da Vitória.

Notice that, in the harbour-side slope, B armour stones below elevation (-5!00) began to be displaced for wave heights of about 7.00 metres, which confirms that the maximum wave height during the storm was about 7.50 metres. In fact, for higher waves, damages in the harbour-side slope would be much more extensive than those actually observed in Praia da Vitória.

Likewise, damages in the sea-side slope were extraordinarily aggravated by an increase even slight of the wave height, and when this reached 7.70 metres it produced a breach analogous to the one observed near the head of the breakwater, which therefore can be easily explained by the slight energy concentration disclosed by the wave patterns in that zone of the breakwater.

Although the damages for the maximum wave height of 7.50 metres can be slightly different in the different tests, the very irregular distribution of damages observed could not be explained save by possible differences in constructional details in the different zones of the breakwater, possibly due to the presence in certain zones of A armour stones heavier than those recommended in the project. In fact we are aware through local sources that 20 large tons stones were placed in the breakwater, although their exact location is not known.

7 - CONCLUSIONS

From a joint analysis of the observed damages and of the model tests, the following conclusions can be drawn about the behaviour of the Praia da Vitória breakwater:

- The characteristics of the most violent storm that will act on a maritime structure must be chosen with great care; a deficient estimation can result in serious damages, as in the present case: observation programs with a suitable equipment are recommended; in addition to the probability of occurrence of certain wave-heights, it is also of interest to determine the maximum possible continuous duration of the storm (in the present case, the prolonged action of the storm was a deciding factor in the extent of damages);

- every breakwater design must be checked by model tests in order not only to disclose any deficiency but also to determine the factor of safety of the structure and its behaviour under wave-heights above the design values;

- a very satisfactory agreement having been observed between the damages observed in nature and the results of the model tests, these together with data on sea conditions near the breakwater enable this to be designed so that its total cost is minimum.

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