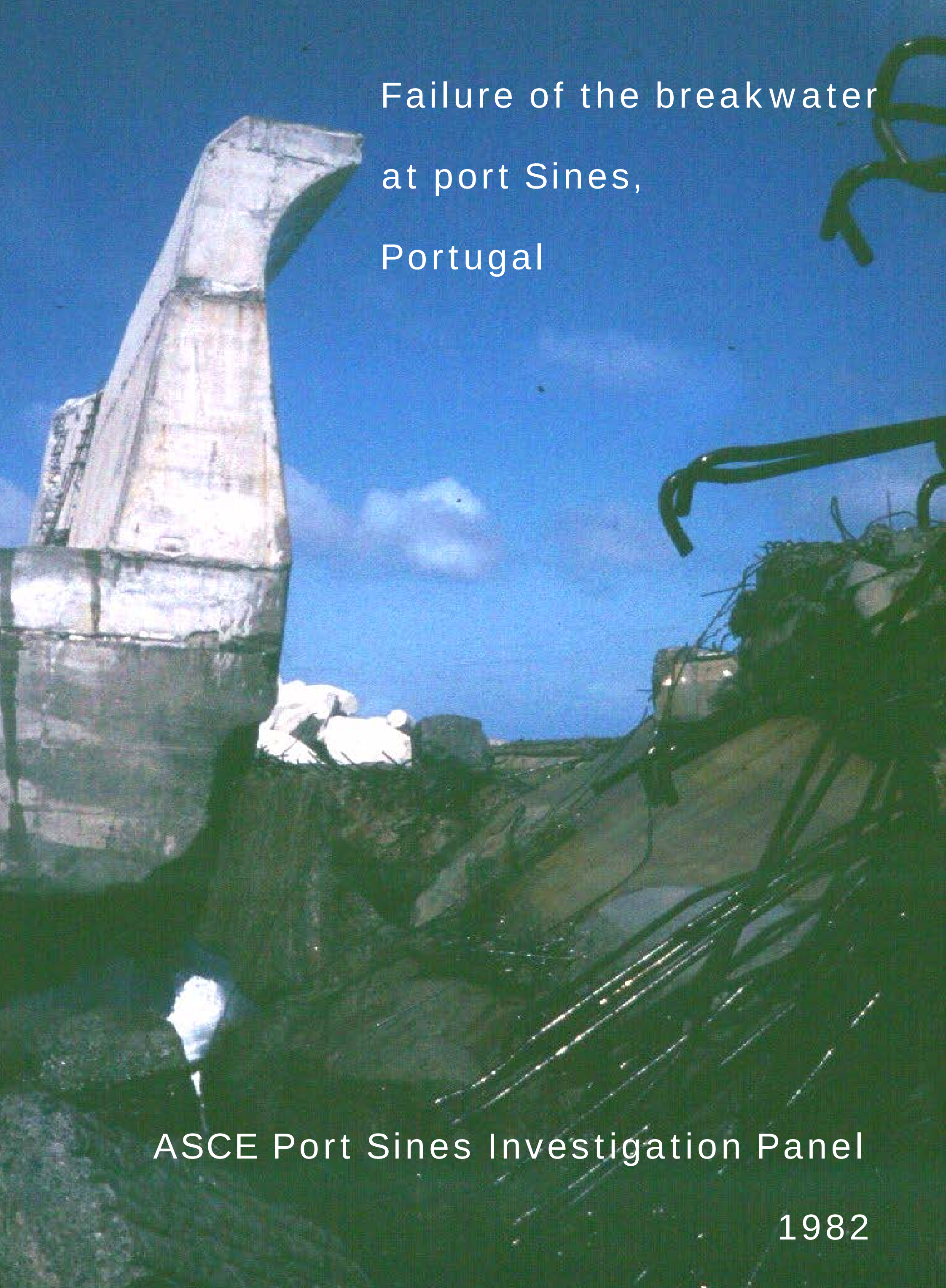
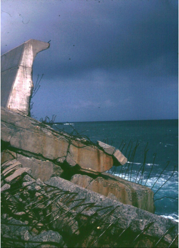
Failure of the breakwater at port Sines, Portugal



ASCE Port Sines Investigation Panel

1982

This pdf contains a series of pictures of the failure, followed by the ASCE report



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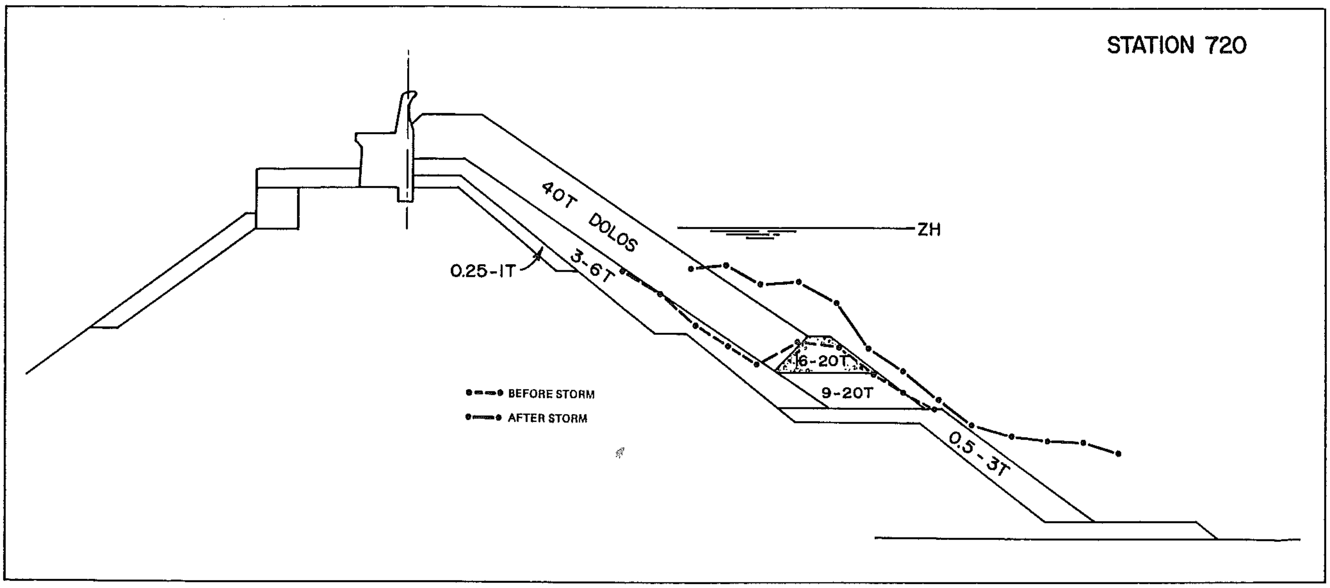
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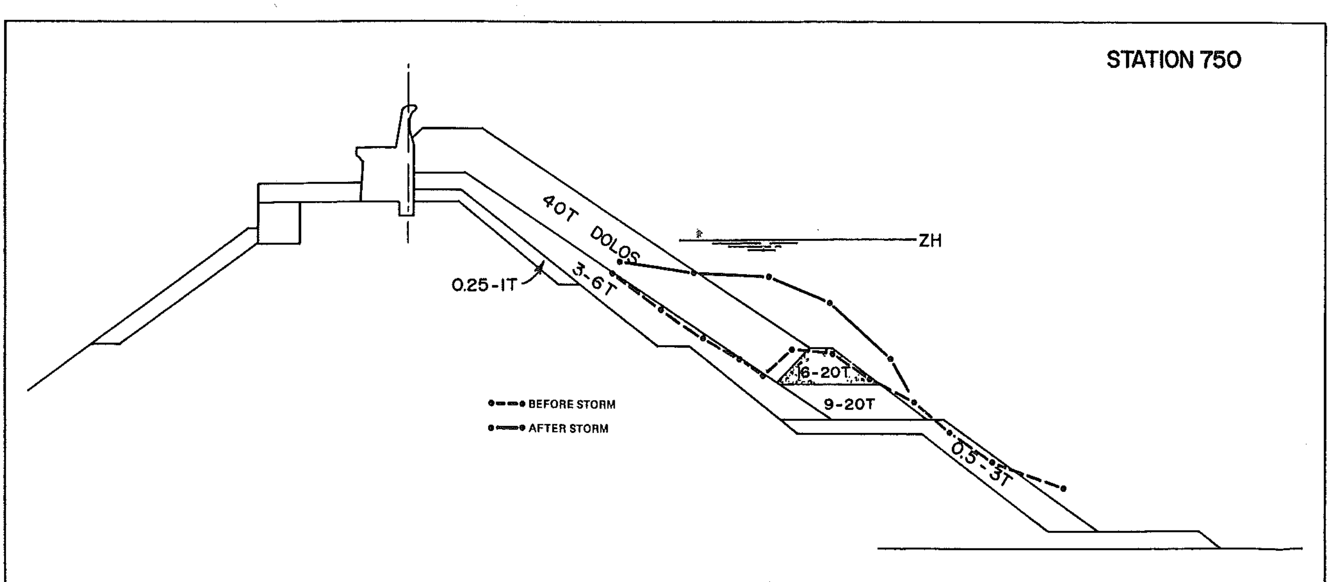
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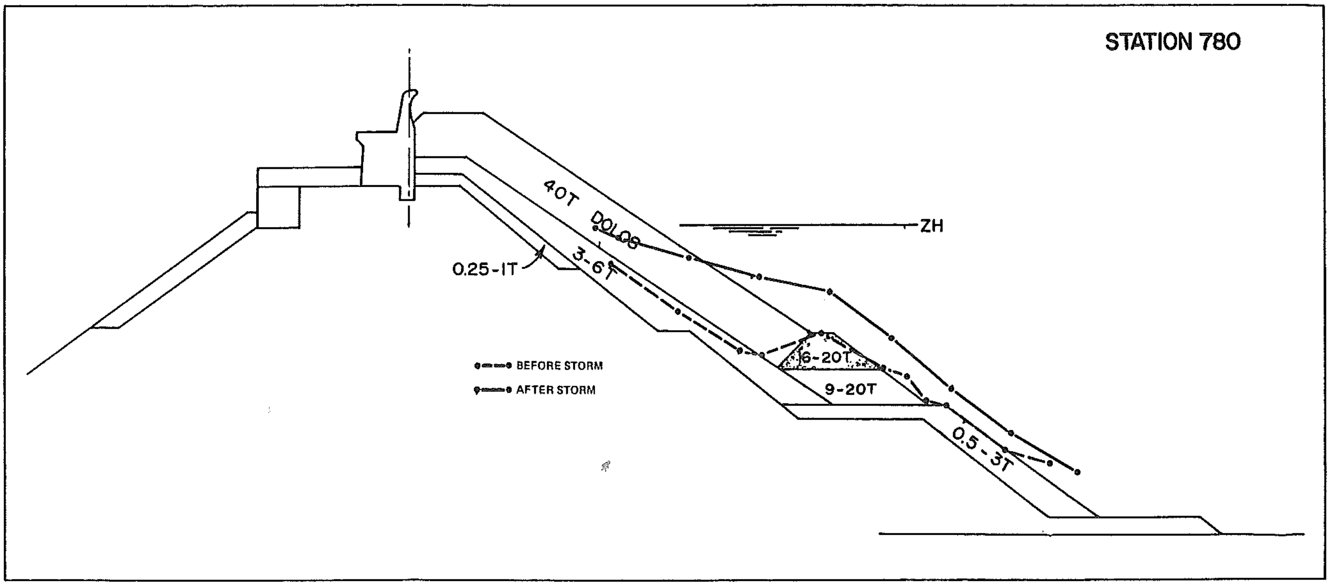
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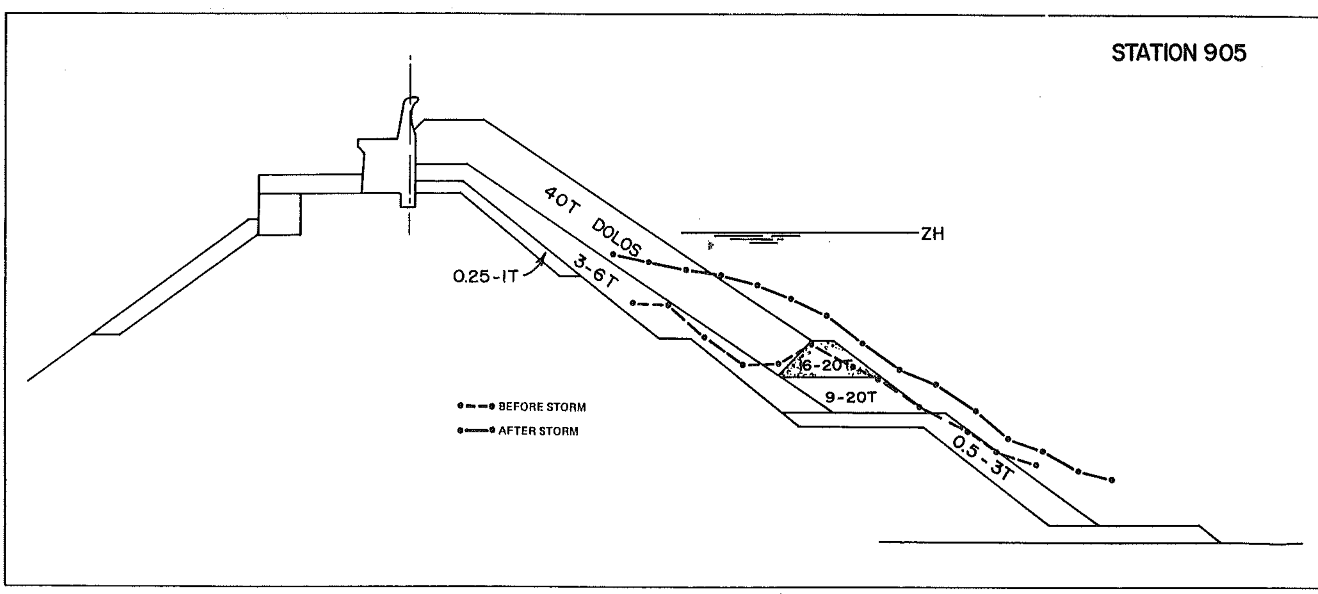
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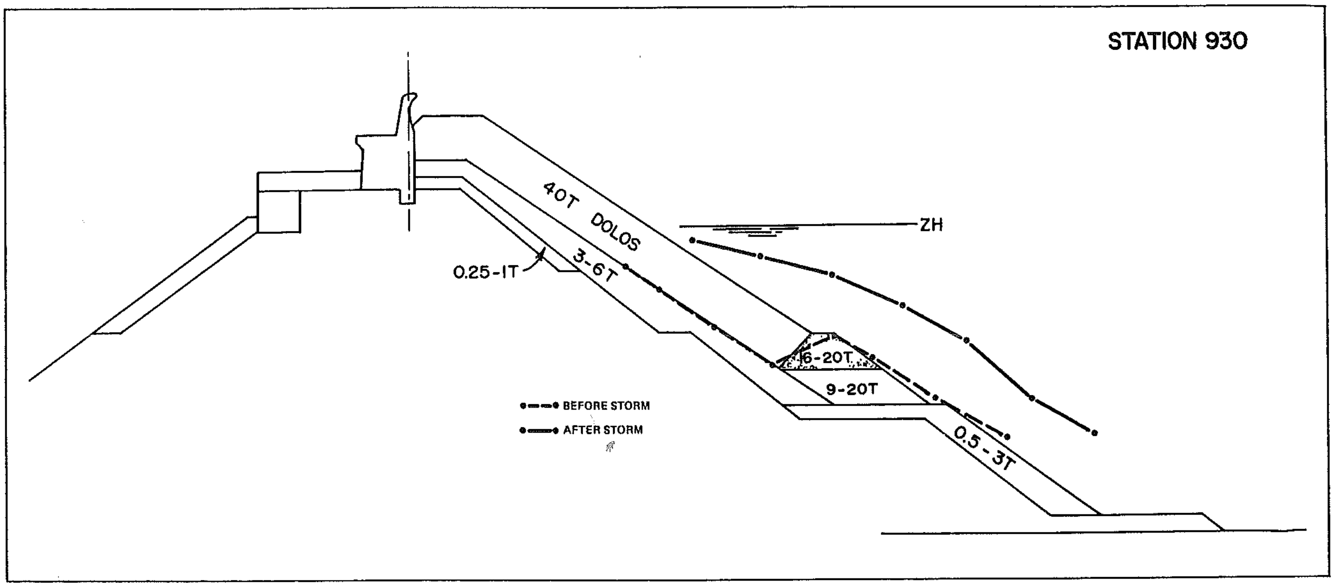
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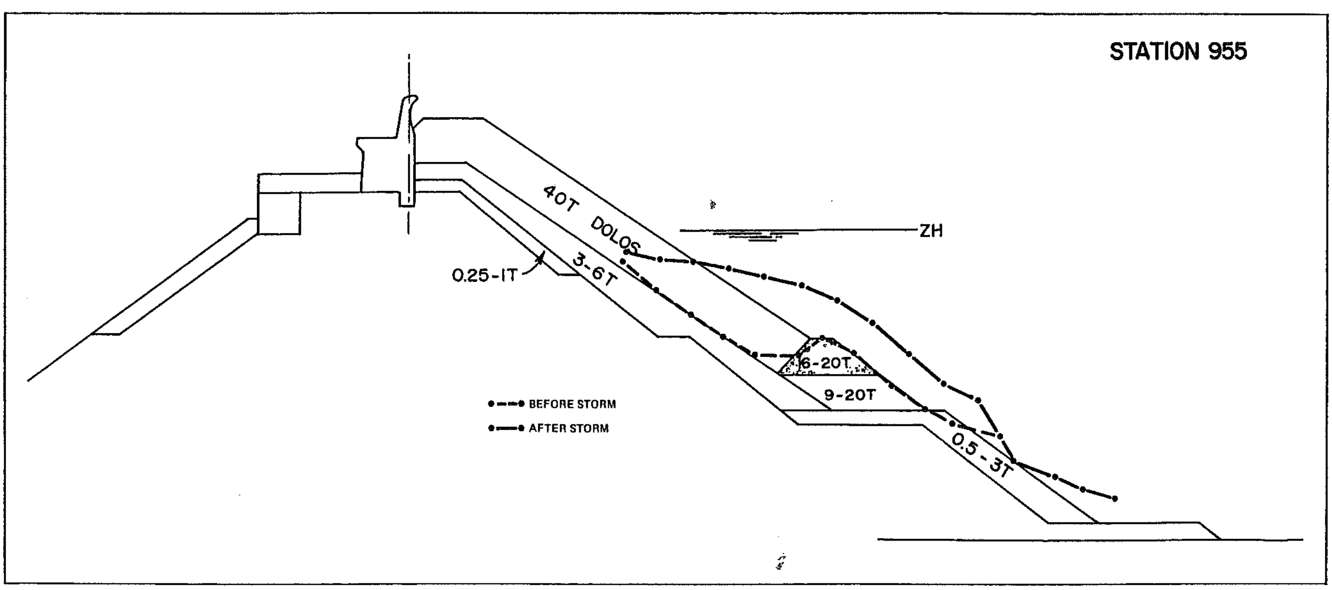
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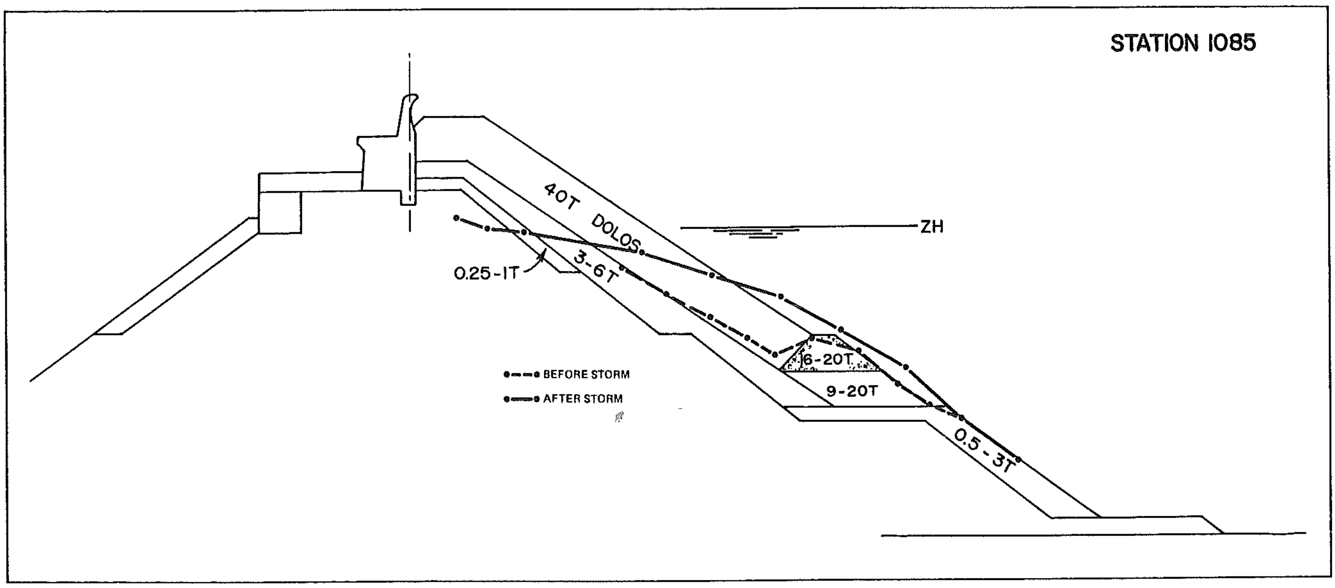
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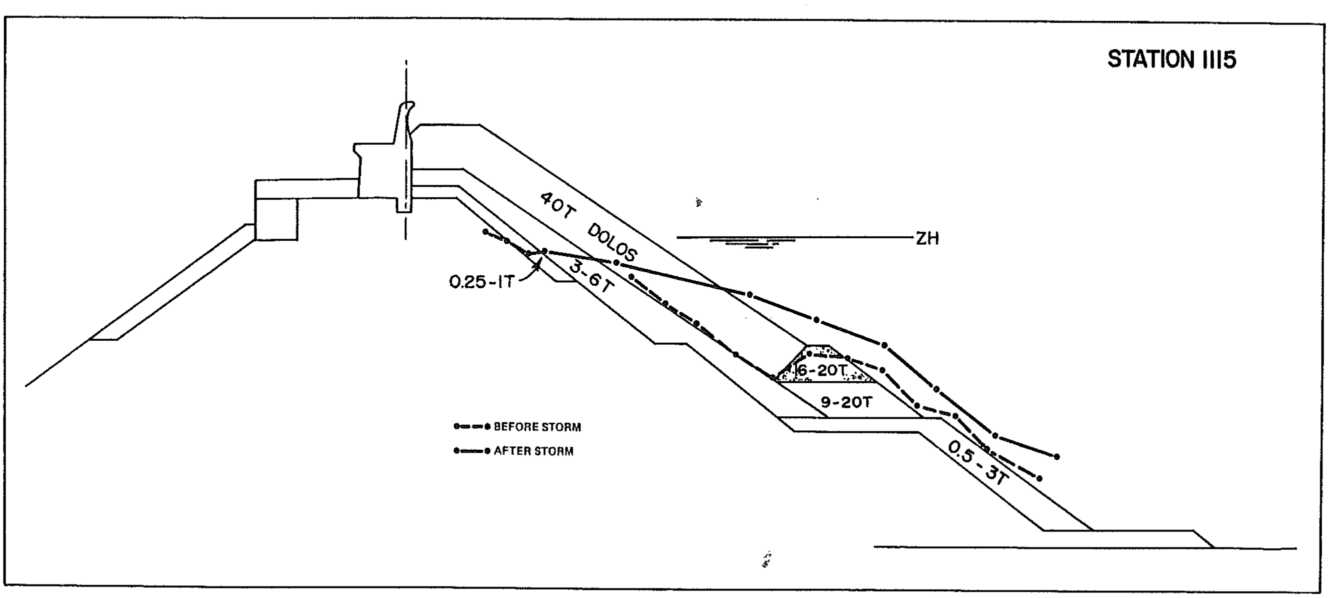
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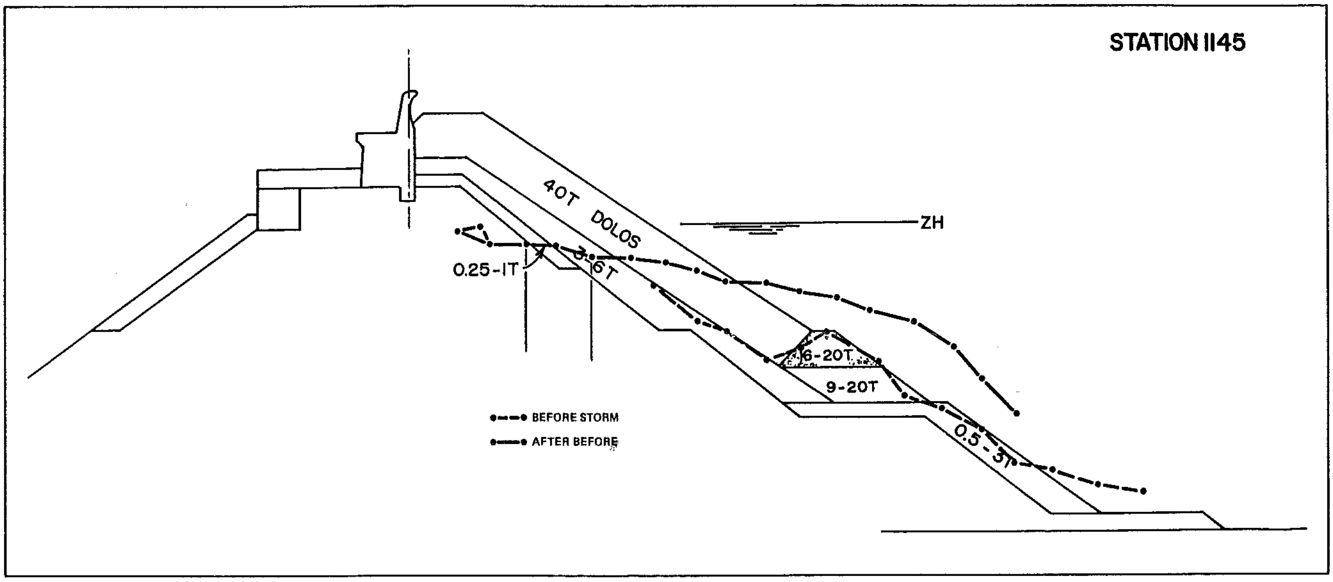
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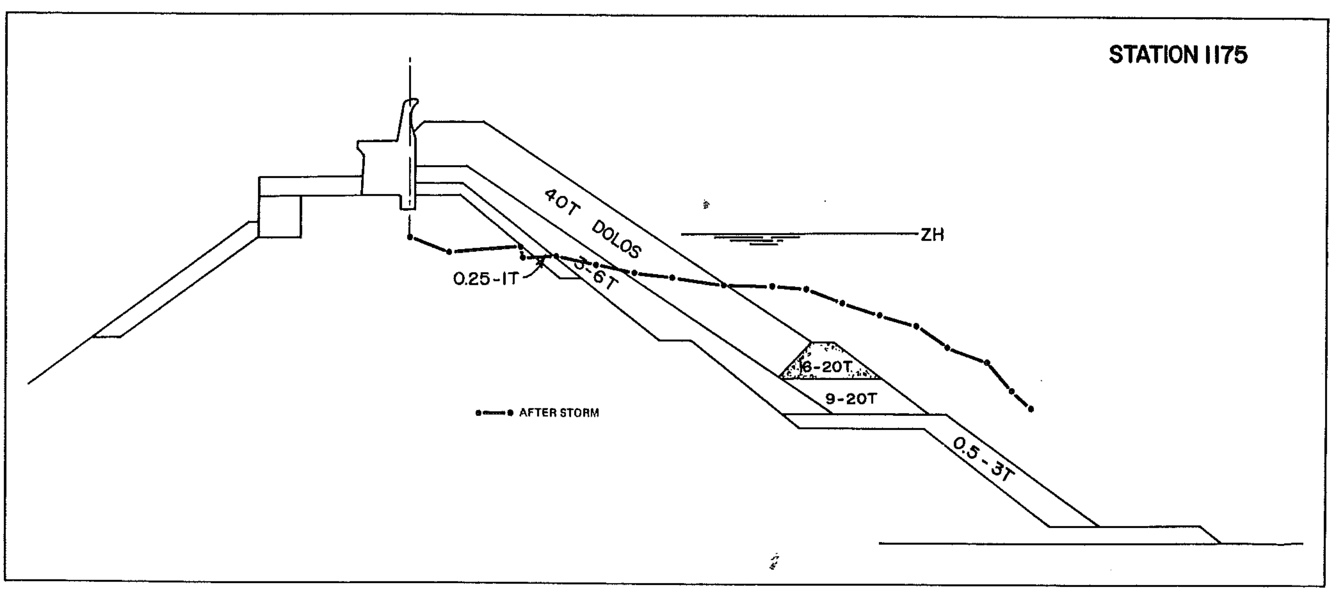
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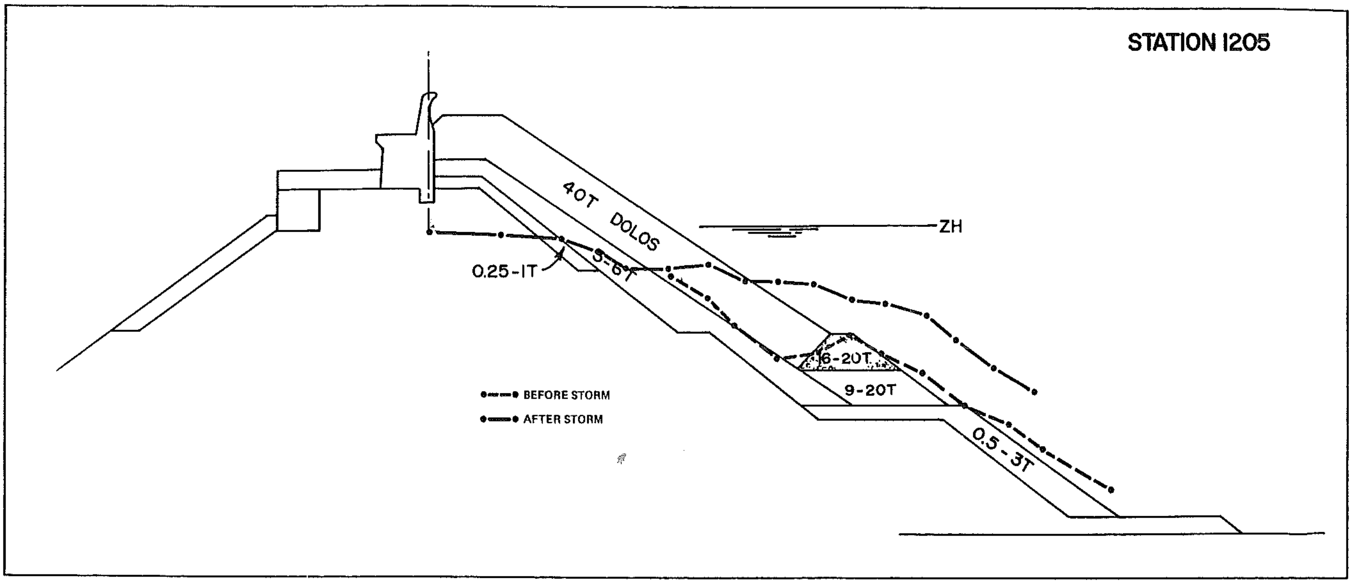
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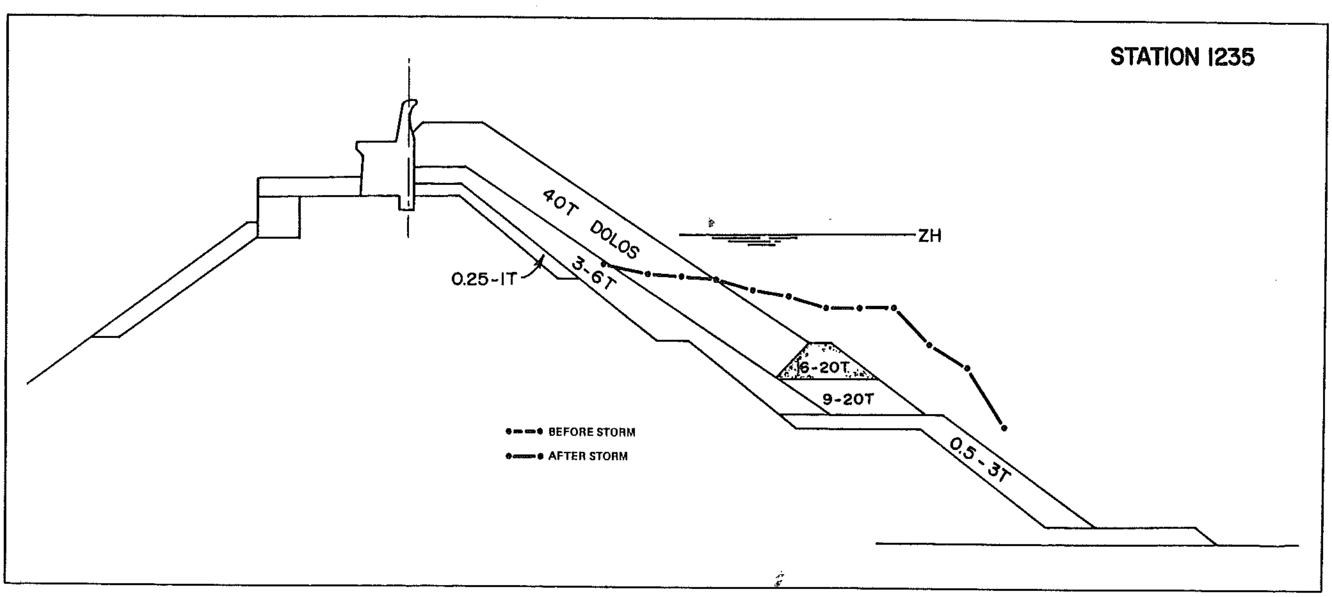
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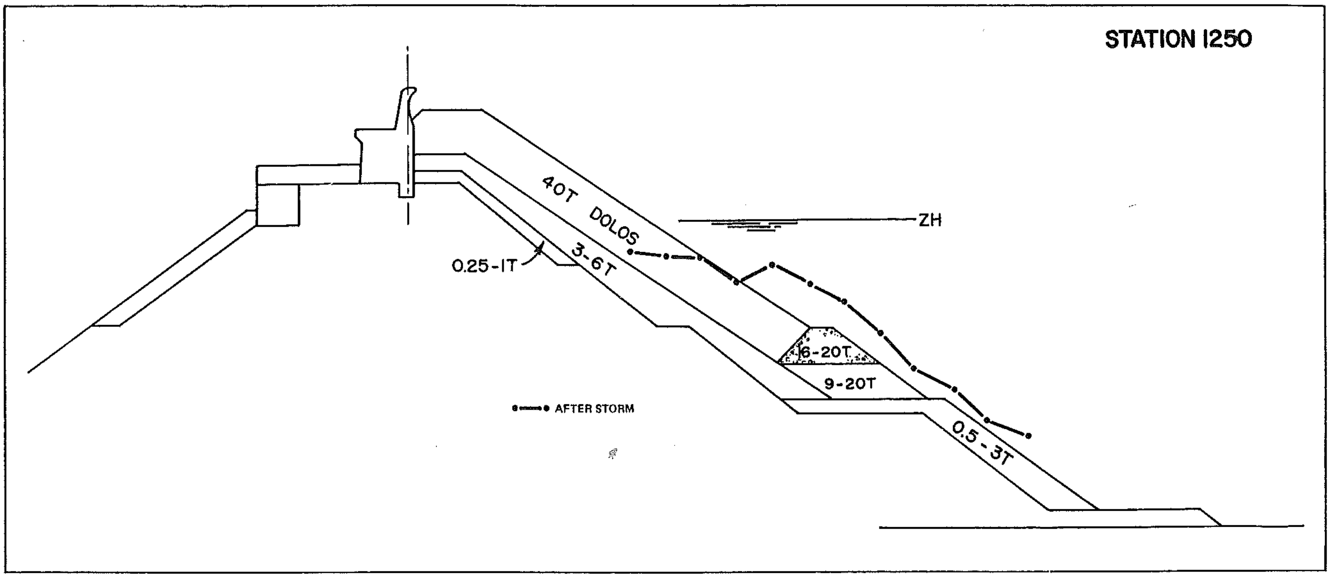
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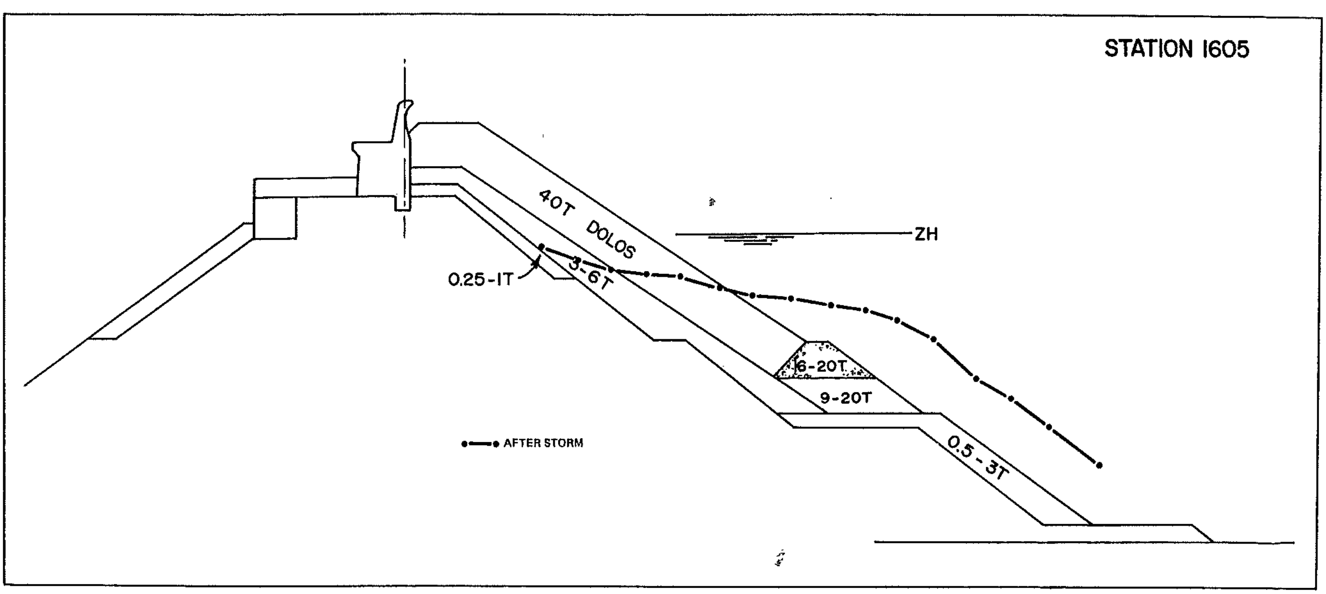
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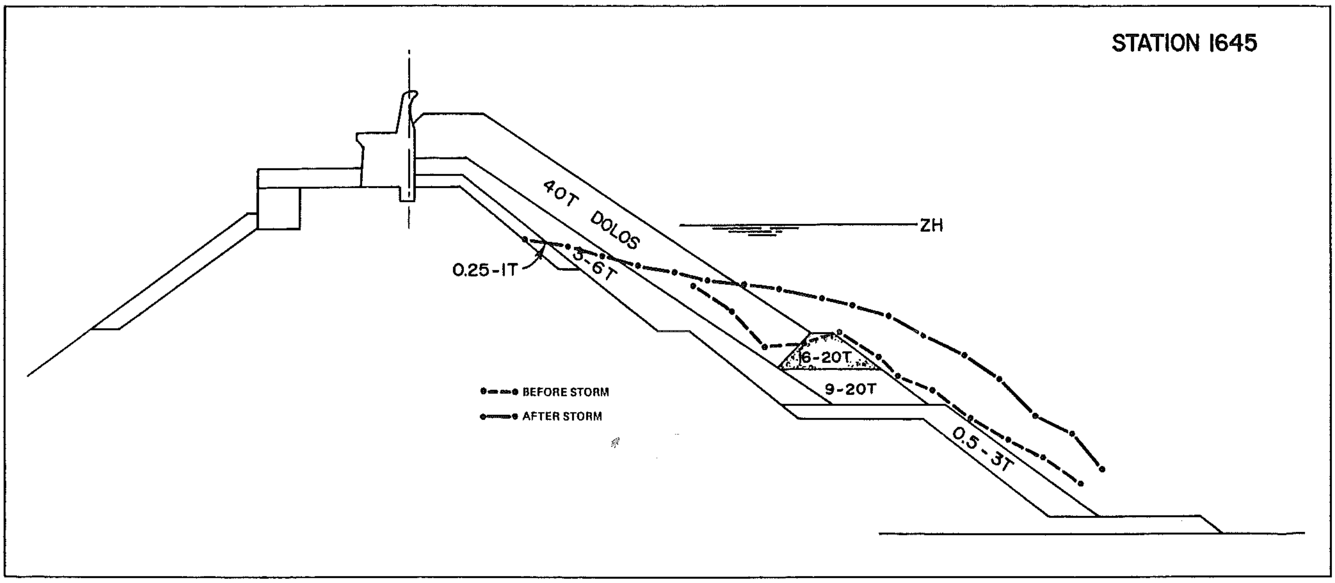
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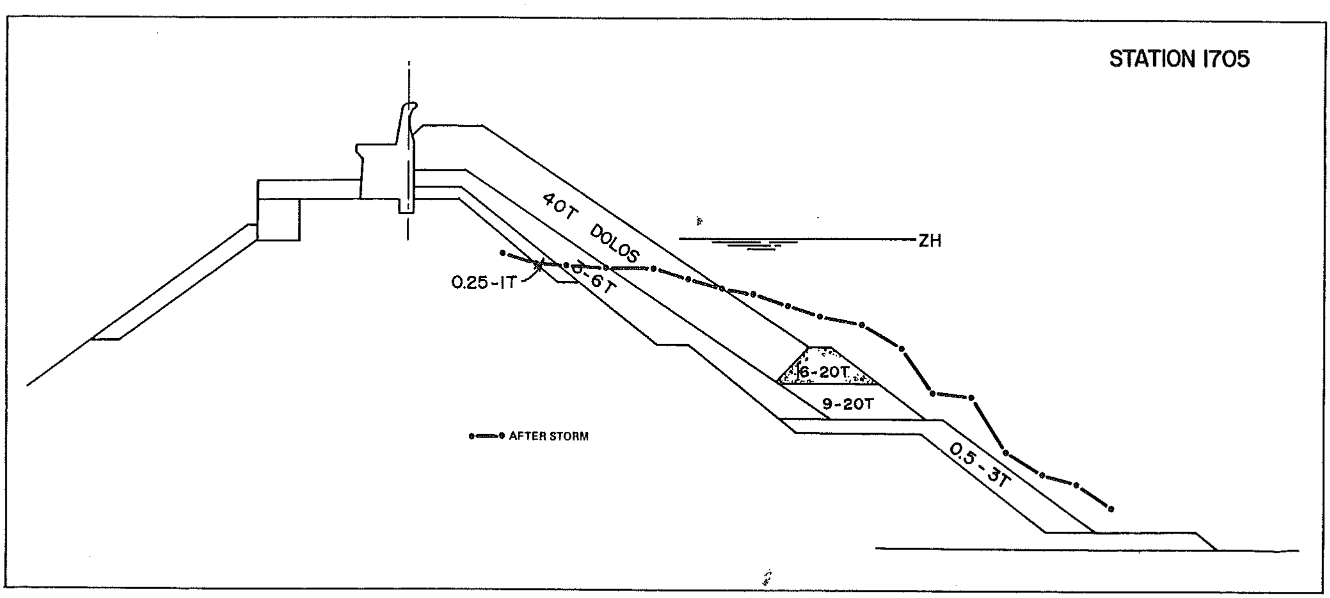
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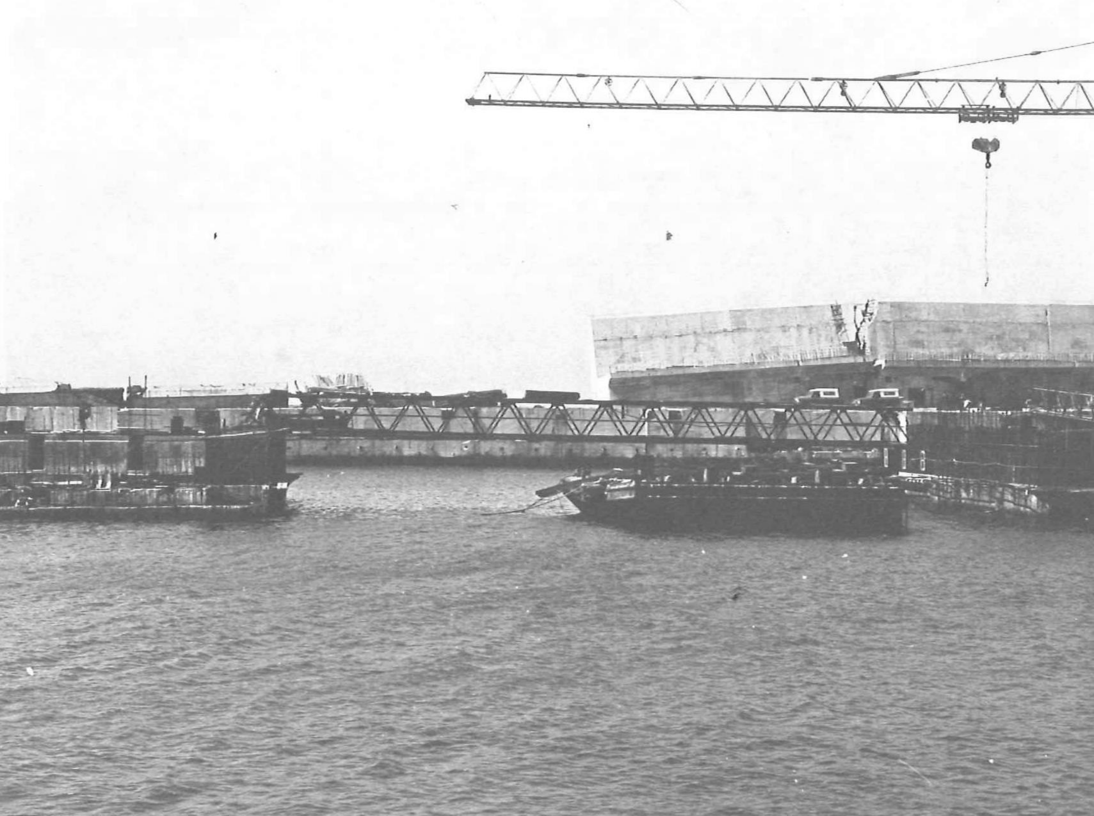

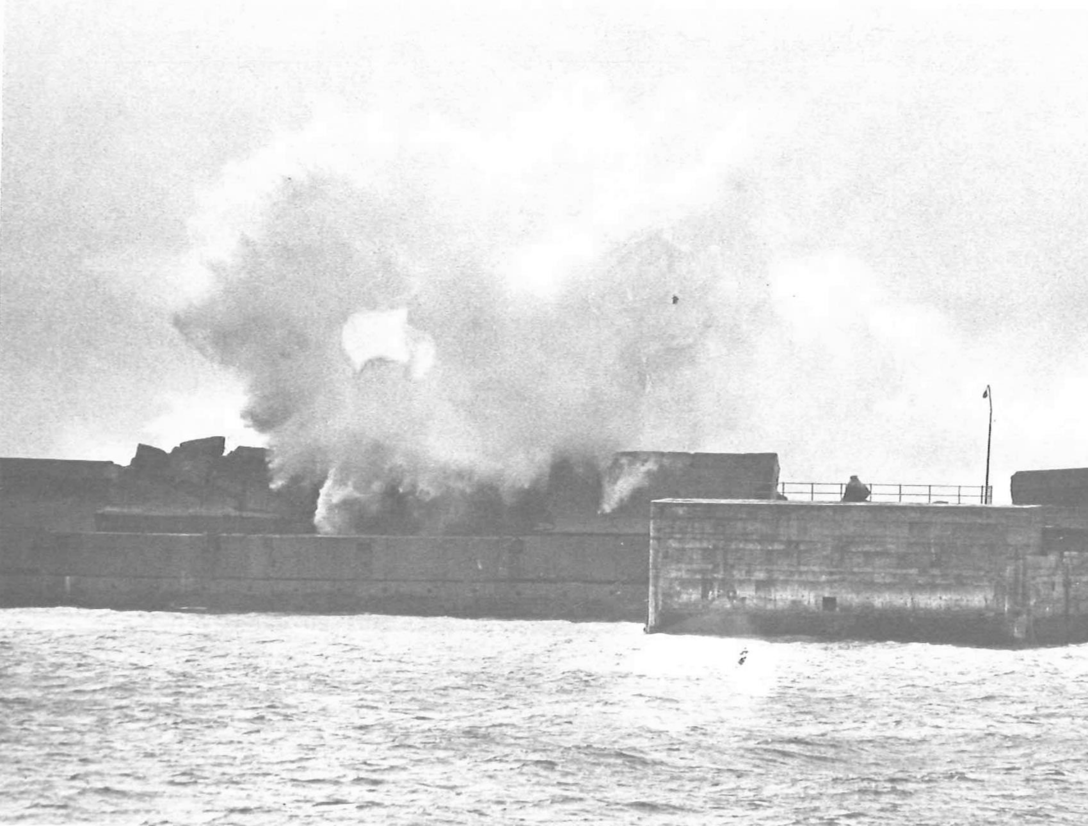


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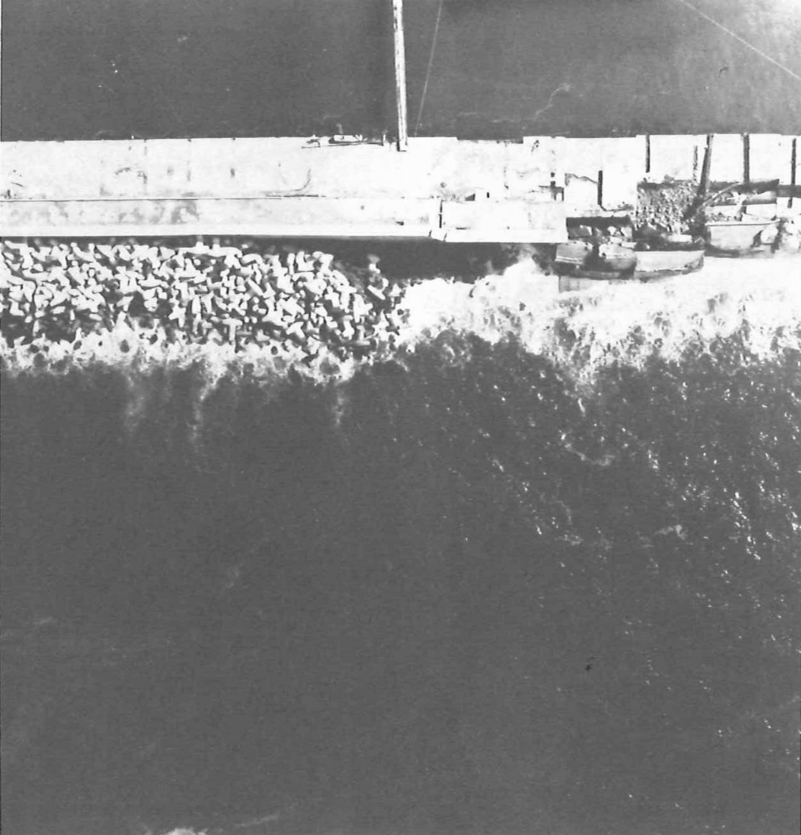

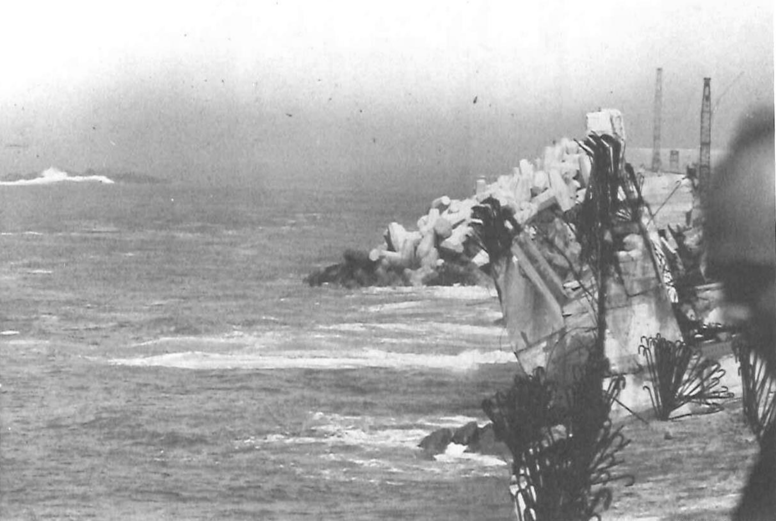

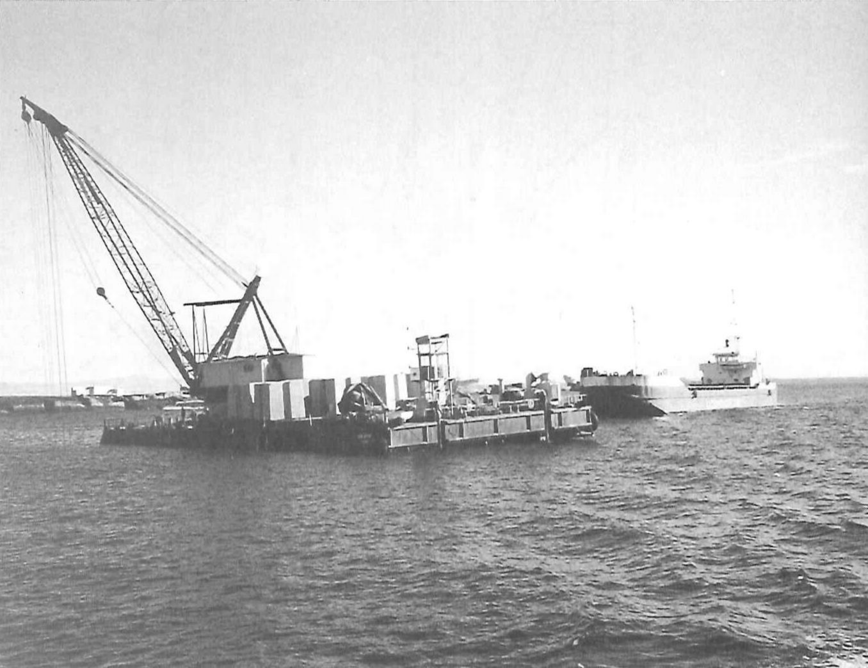

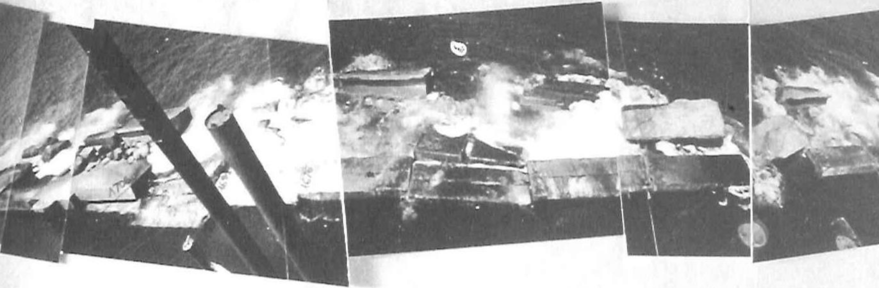

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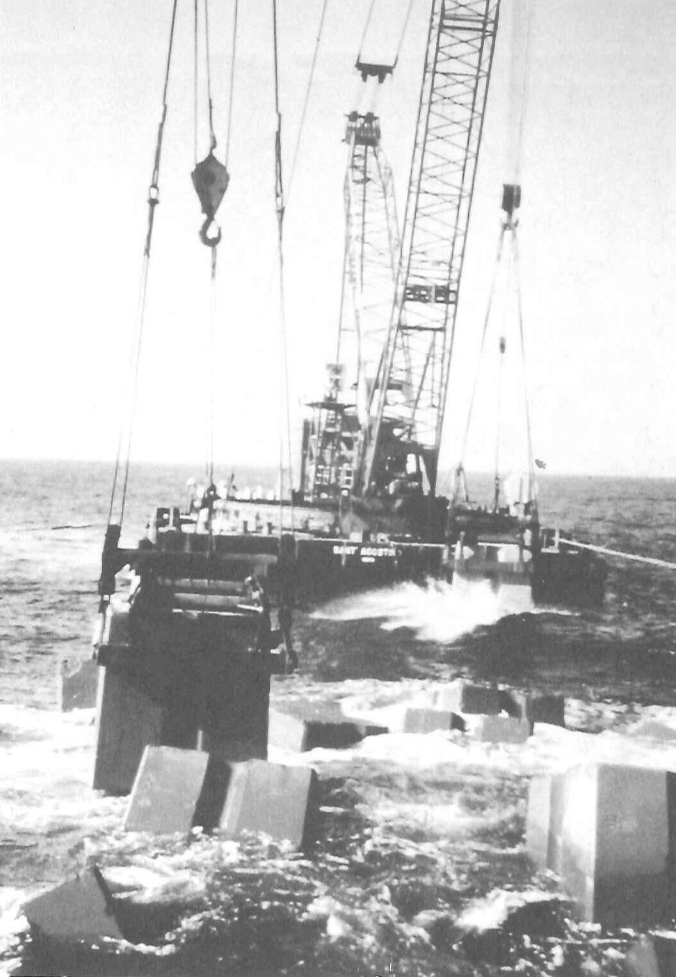

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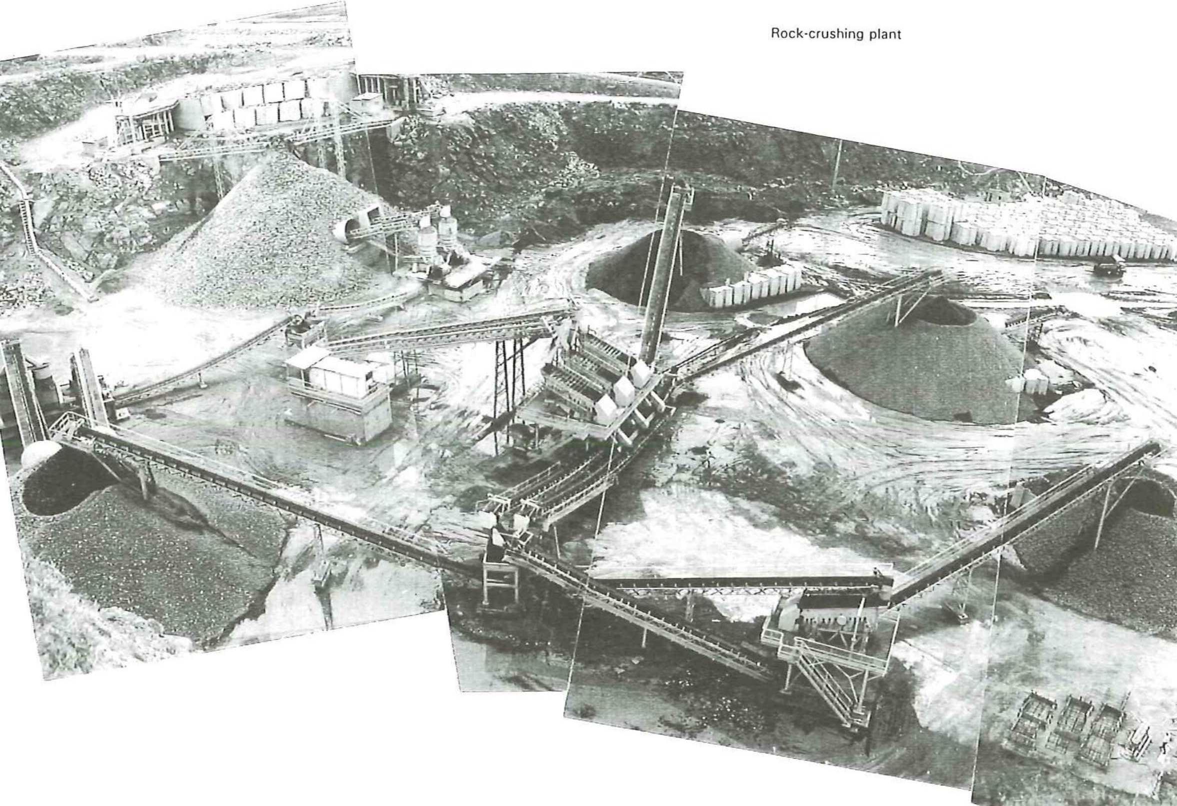

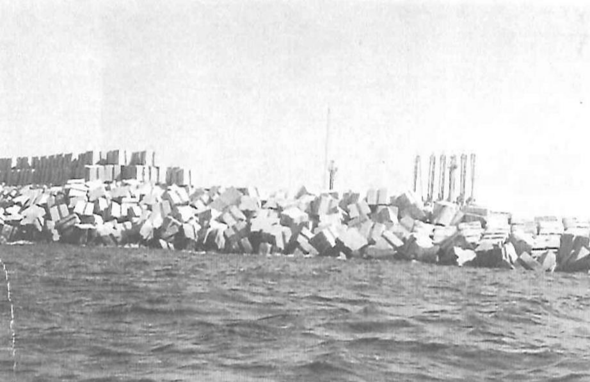

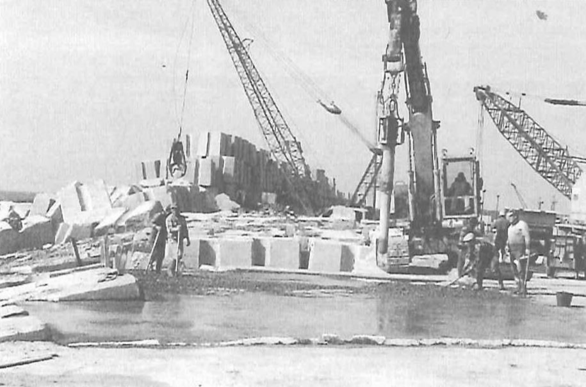

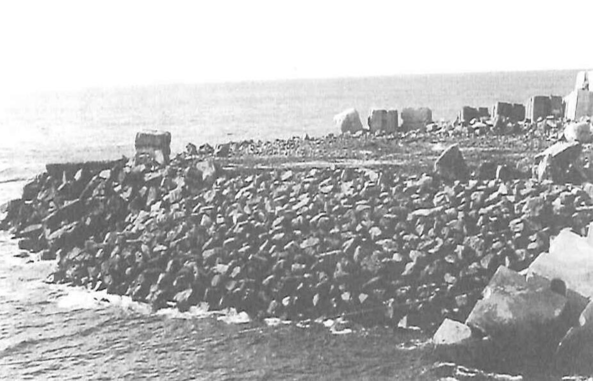

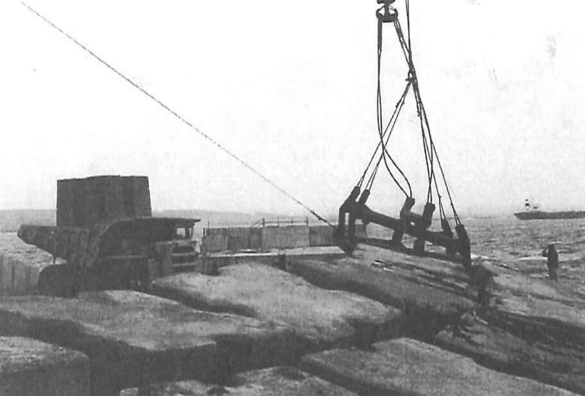

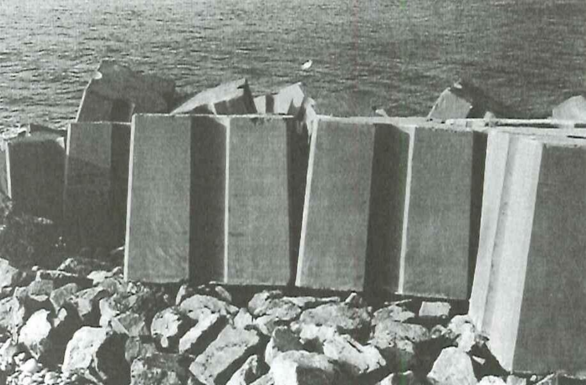

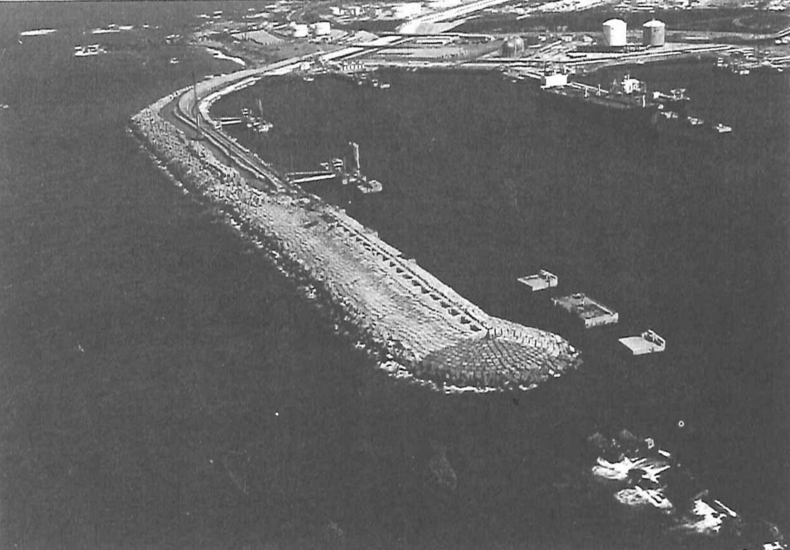

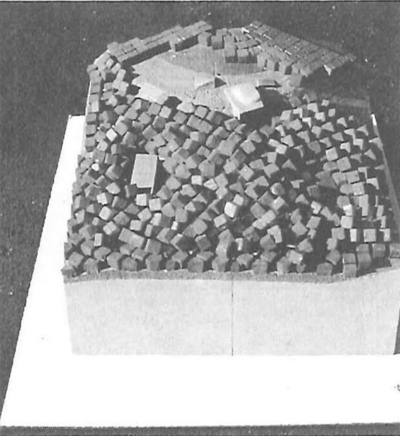

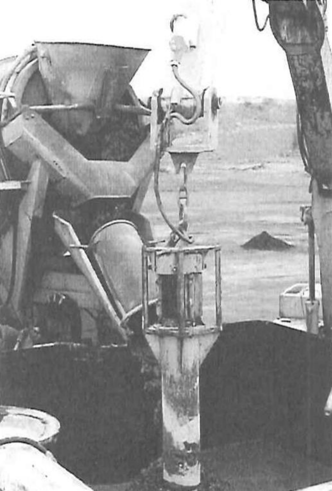

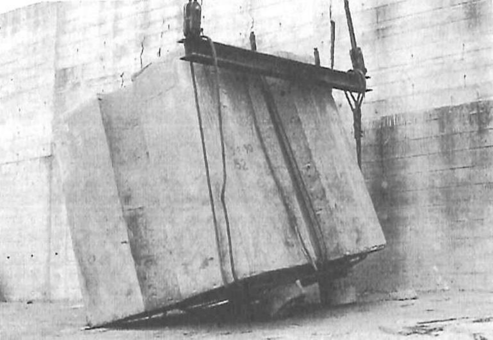


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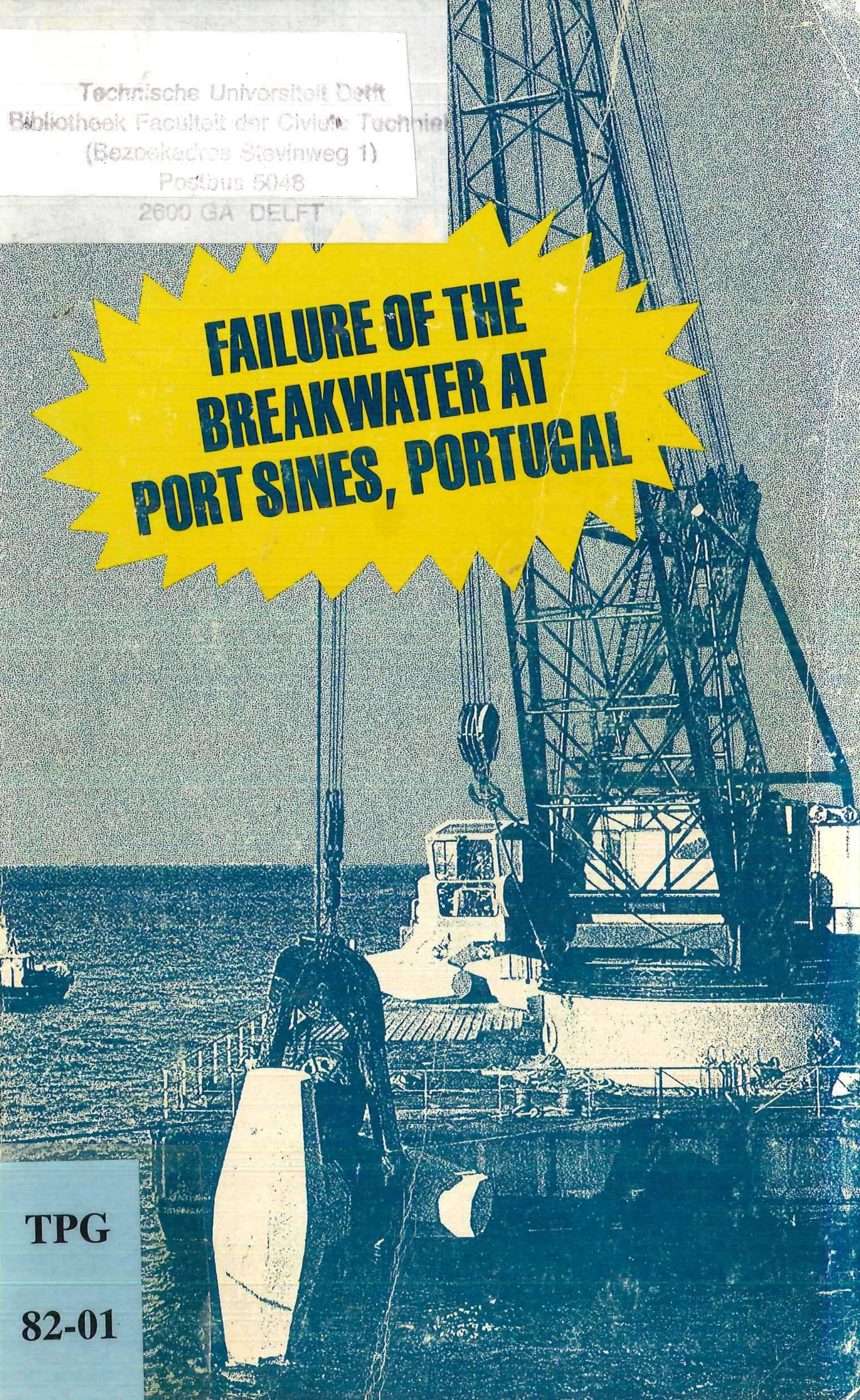
FAILURE OF THE BREAKWATER AT PORT SINES, PORTUGAL



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FAILURE OF THE BREAKWATER AT PORT SINES, PORTUGAL

Prepared by
PORT SINES INVESTIGATING PANEL

Approved for publication by the
Coastal Engineering Research Council of the
American Society of Civil Engineers

This report is an investigation of the breakwater failure that occurred on February 26, 1978, and a compendium of data and reports relevant to this failure.

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Dedicated to the Memory of

JOSEPH M. CALDWELL

ACKNOWLEDGEMENTS

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CHAPTER 1

Introduction

This report summarizes the efforts of the Port Sines Investigation Panel, funded by the National Science Foundation and headed by Dr. Billy L. Edge of Clemson University, Clemson, South Carolina. The objectives of this panel were to collect perishable data on and to identify possible causes of the failure on February 26, 1978, of the rubble mound breakwater at Port Sines, on the Atlantic coast of Portugal.

The structure is the largest of its kind in such an exposed environment. The breakwater is situated in a previously untried combination of unusually deep water (about 50 meters at the seaward terminus) and a high-energy marine setting (the 100-year return period significant wave height was estimated to be 11 meters). Construction was nearly complete when critical damage was sustained in storm waves thought by most to be below the 11 meter significant wave height for which the structure was designed. The damage consisted of the complete loss of some two-thirds of the armor layer of 42 ton* dolos units. At a few locations the concrete superstructure was severely damaged as a result of undermining and of wave impact on the front face where loss of the dolos had occurred.

The breakwater was to have sheltered within its reach a port for an ambitious industrial complex on which the economic future of Portugal is in large part predicated. The port includes berths for deepwater oil tankers and terminals for liquified petroleum gas, ore carriers and break bulk cargo. Had the Sines Port been fully operational when the damage occurred, the economic consequences would have been multiplied many times over.

As the world energy search accelerates, engineers will be designing more and more energy-related facilities along shorelines which may require deep water access and may therefore involve breakwater structures. The integrity of associated wave protection structures may have other than economic significance.

The objective of this investigation, as delineated in the proposal dated March 1978 to the National Science Foundation for the Sines investigation, was to collect perishable data that could be used in subsequent studies to evaluate five simplified failure possibilities:

* metric tons

(1) that design criteria were exceeded by the February 1978 storm;

(2) that the breakwater construction was faulty;

(3) that the materials used for breakwater construction were substandard;

(4) that the procedures followed during the design of the breakwater were incomplete or incorrect for this specific set of environmental conditions;

(5) that the design was inadequate.

This investigation was in response to a series of rubble mound failures which have recently been observed around the world. Edge and Magoon (1979) have noted the details of several of these failures.

The investigation undertaken by this committee was based on two site visits (May and August 1978) and on discussions with Portuguese authorities responsible for the project, the design engineers, the engineers of the Portuguese Hydraulic Laboratory (LNEC), site supervision staff and the officials of the construction company.

The objective of this investigation was not to find a crucial error or omission. Indeed, given the immense size and complexity of this project, many factors stand out for consideration. The objective of this investigation was to report on possible problems, omissions, and errors that could have contributed to the failure of the Sines breakwater. It is the hope of the panel that, as the result of this report, engineers will be able to benefit from the experience at Sines in the design and construction of future breakwaters.

The structure has sustained further damage since the February 1978 storm. In December 1978 and in February 1979 storm action removed all armor protection, including some temporary remedial works placed in the Fall of 1978, from the seaward 1-1/2 km of the breakwater. Much of the concrete superstructure has been lost.

CHAPTER 2

Project Description

The Sines breakwater is the single most critical component of a vast industrial complex planned by the Portuguese government. The Atlantic deepwater port and a host of landside facilities depend on the efficient, safe operation of the port, which in turn depends on a secure breakwater. The entire complex is sited immediately south of Cape Sines because of that area's deep water. The Cape, about 100 km south of Lisbon along the Atlantic coast (see Figure 2.1), lies along the present international routes for crude oil and iron ore carriers. The site has the steep ocean falloff needed for a supertanker port; required depths for an oil terminal occur within 1/2 to 1 1/2 km of the shore.

Development of a major port at Cape Sines involved the construction of a breakwater in water depths of up to 50 m. A layout of the port facility is given in Figure 2.2. Design was begun in 1972 by Bertlin and Partners of the United Kingdom and Consulmar and Lusotechna, both of Lisbon; the contractor chosen was Condotte d'Acqua of Italy. Construction began in mid-1973.

The breakwater is a dual-purpose structure, supporting oil pipelines as well as providing shelter from the Atlantic Ocean for the port. A quarry fill core is armored with larger stone, and on the seaward side that "selected" stone is blanketed with 42-t concrete dolos. A concrete superstructure includes a wave wall, an inner (portside) roadway, and support for the oil pipelines. The latter are intended to serve the three harborside berths, built on caissons and connected to the breakwater. The three, beginning with the closest to shore, are intended to accommodate 100,000, 350,000 and 500,000 DWT tankers, respectively.

About 160 square miles of inland facilities also depend on the port. Those include an oil refinery, a steel mill, a pyrite plant, a petrochemical complex, and other industry, both heavy and light. Yet another aspect of the master plan is a "new town", for which a population of about 100,000 has been projected. Over 5,000 dwellings are complete, and the construction of schools, recreation centers, shopping facilities and related infrastructure is underway.

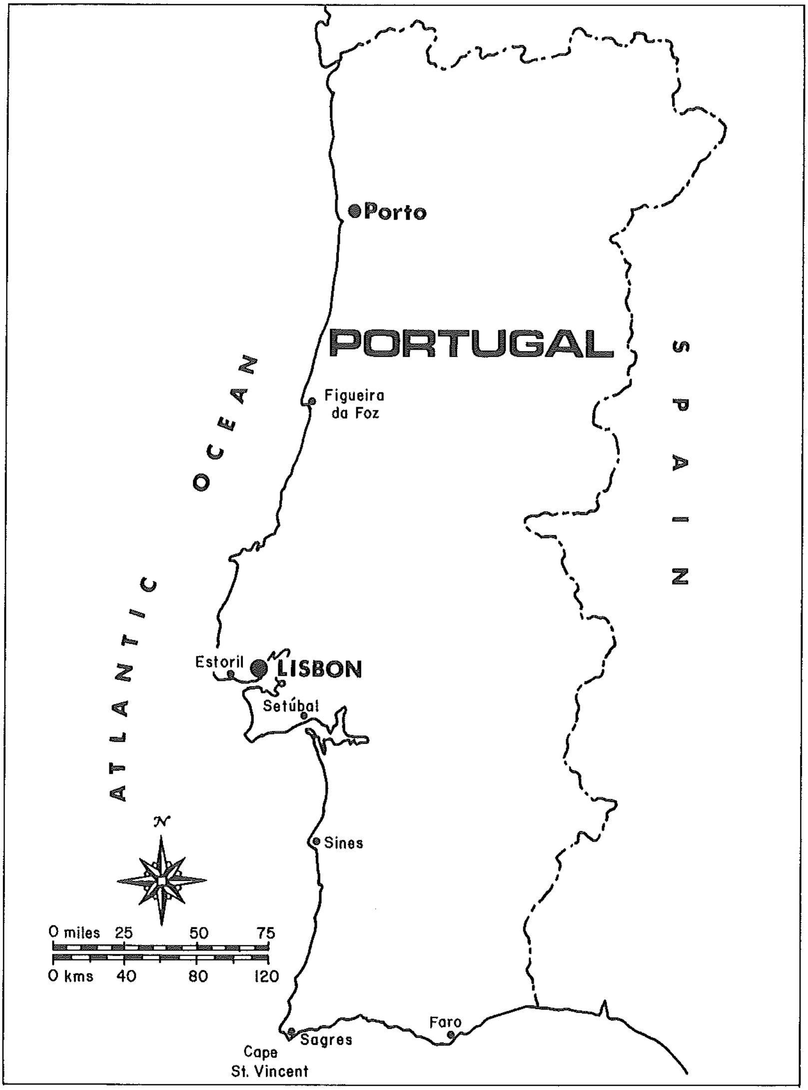

FIGURE 2.1 LOCATION OF PORT SINES

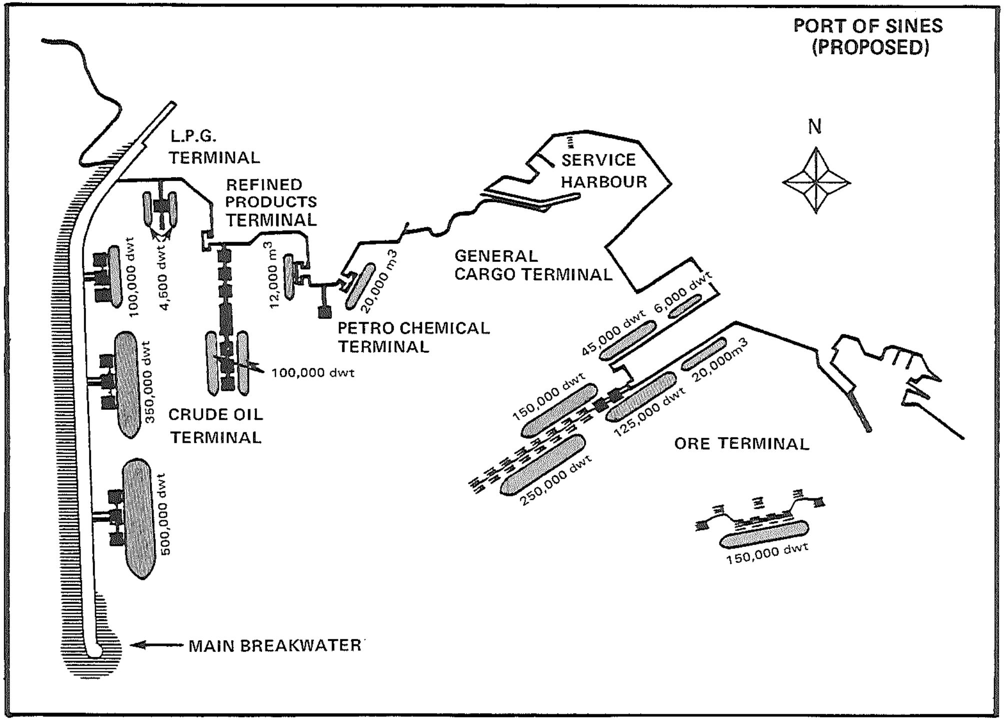

FIGURE 2.2 LAYOUT OF THE PORT OF SINES

The Gabinete da Area de Sines (GAS) has been the responsible Government agency for the entire Sines development since the project inception over a decade ago. The government of Portugal has undergone several changes during those years, most notably a revolution in 1974, the year that design was completed and construction began. The Sines project originally included private capital. Some portions of the project were nationalized after the events of 1974. At the time of the failure in February 1978, the Portuguese government is reported to have spent $176,000,000 on the breakwater alone.

The breakwater was designed by the consortium Bertlin-Consulmar-Lusotechna. Bertlin and Partners of Redhill, England, led the joint design effort with the two Portuguese firms. The breakwater was constructed by the Italian firm Societa Italiana per Condotte d'Acqua of Rome through their Lisbon office.

CHAPTER 3

Physical Setting

The local environmental factors which would affect the design and construction of the Sines breakwater are winds, waves, tides, coastal currents, earthquakes and the regional geologic setting.

The wind and wave climate of Portugal is dominated by the "Azores High." This is a high pressure system frequently present over a group of islands some 2,000 km west of Portugal. While this high pressure is stationary, low pressure disturbances travelling across the Atlantic Ocean from west to east are forced to travel to the north of the coast of Portugal. The high pressure system may effectively remain over the Azores for a number of years. When this high pressure region is displaced, low pressure disturbances may move directly towards the coast of Portugal; 1978 was a year characterized by such a situation.

Winds

The winds affect the breakwater principally by generating waves which attack the breakwater, and this is the major factor influencing its design. Another effect of the wind is in producing a storm surge, but, as explained in a later section, this is a minor concern at Sines.

Precise data describing the wind climate at Sines, or offshore from Sines, and the wind speeds and directions associated with the February 1978 storm are not available.

Severe storms over the Atlantic Ocean that are not recorded on the coast of Portugal may produce very large waves which can reach the Port of Sines. However, until recently, these waves were not measured. It was such a condition that existed on 26 February 1978. These winds may be felt on the Portuguese coast, but the speeds may be far less than they are over the open fetches of the ocean where the waves are generated. Thus, measurements of wind speeds, durations, and directions on the Portuguese shore are not reliable indexes of the winds that may be expected to generate waves reaching the Portuguese coast.

Wind velocities over the ocean approaches to Sines are measured aboard vessels traversing the waters off the Portuguese coast. One set of these data has been collected and summarized by the U.S. Navy Oceanographic

Office, and the summations appear in that office's Publication 700, Ocean Atlas of the North Atlantic, Section IV "Sea and Swell", 1963 (Reprinted 1970). For the ocean area immediately offshore from Sines, this Atlas shows that winds in excess of 28 knots (14.4 m/s) occur during each month of the year except June, July, August, and September. Unfortunately, the velocity of the maximum winds is not given, the data being confined to the statement that the winds "exceeded 28 knots."

Another set of wind data appears in the Summary of Synoptic Meteorological Observations, Western Europe, Vol. I, (SSMO) published by the U.S. Naval Weather Service in January 1974. These data are also based on shipboard observations, covering the period 1911-1971. Table 3 of the above SSMO for the 5° square off the mid-Portuguese coast shows that winds in excess of a mean velocity of 48+ knots have been observed in 0.1% of the observations (total observations, 12,900). Almost all of the 48+ knot winds have come out of the directions SW through W to N. Winds between 34 and 47 knots have been observed in 2.3% of the total observations, with direction characteristics similar to the 48+ knot winds.

It is of some interest to note here the wind-wave generation relationship given in Figure 3-16 of the Coastal Engineering Research Center (CERC) Shore Protection Manual (1975). A tabulation from Figure 3-16 is given on Table 3.1, showing a few of the combinations of wind velocity, fetch, and duration required to generate wave trains with a significant height of 8, 10, and 11 meters. The 8-m wave generating conditions are shown because the February 1978 storm which severely damaged the breakwater is believed to have generated waves of about this size. A review of Table 3.1 indicates that an 8-m significant wave is probably not a rare occurrence at Sines. This is indicated, for example, by the fact that a 43-knot wind blowing for one day out of the westerly quadrant could generate 8-m waves. Moreover, a 45-knot wind blowing for 33 hours could be expected to raise a 10-m wave and 48-knot wind blowing for 32 hours to raise an 11-m wave. These values are based on a 500 mile fetch which is possible for this site.

The purpose of this presentation is to show that winds of sufficient velocity to generate 10-m, 11-m, and even higher waves have been

TABLE 3.1

EXAMPLE CONDITIONS REQUIRED TO

GENERATE SIGNIFICANT WAVE HEIGHTS OF 8, 10, AND 11 m

To generate 8-meter (26-ft) waves

Wind	Fetch (miles) 100	300	500	1000
Velocity (knots)	60	43	38	35
Duration (hours)	8	23	37	70

To generate 10-meter (33-ft) waves

Wind	Fetch (miles) 100	300	500	1000
Velocity (knots)	73	53	45	41
Duration (hours)	7	20	33	62

To generate 11-meter (36-ft) waves

Wind	Fetch (miles) 100	300	500	1000
Velocity (knots)	77	55	48	43
Duration (hours)	7	19	32	60

(This table is based on Figure 3-16, Volume 1, U.S. Army Coastal Engineering Research Center Shore Protection Manual, 1975 Ed. The stated wave heights are "significant" heights.)

observed on shipboard in the 5° square off Lisbon. These observations were primarily for the 60-year period, 1911-1971. The observations indicate that on 16 occasions over this 60-year period, the winds were in excess of 48 knots. The "duration" of the winds is not given in the referenced SSMO.

It must be pointed out that the above data should only be used to provide an overall understanding of the climate of the Atlantic Ocean adjacent to Portugal. To provide useful information on waves generated by wind, the entire characteristic of the windfield during the storm must be known.

Waves

Three sets of wave data applicable to the Sines area are readily available. Recorded wave data have been obtained over a seven year period with wave gauges located at Figueira da Foz, some 250 km north of Sines. A description of the Portuguese wave climate measurement program existing in 1964 is provided by J.P. Barcelo (1964). However, these data are taken very near the shoreline and are likely to be depth limited.

The second set of available data consists of shipboard observations made from vessels traversing the ocean waters offshore from Portugal. The U.S. Navy Oceanographic Office analysis of these data, as given in their publication 700, is summarized below. These data are observations by ship's officers of "the average height of the larger well-formed waves" and are considered to be approximately equivalent to the significant wave height. The significant wave height is defined as the average of the one-third highest wave heights in a record. The wave heights are normally defined using a zero-crossing procedure (Draper, 1966).

TABLE 3.2

PERCENT OF TIME IN EACH MONTH THAT

SIGNIFICANT WAVE HEIGHT EXCEEDS STATED VALUE

	2.4 m (8 ft)	3.7 m (12 ft)	6.1 m (20 ft)
Jan.	-	-	-
Feb.	10%	5%	2%
Mar.	15	5	2
Apr.	15	5	2
May	7	2	1
June	5	2	1/2
July	5	2	1/2
Aug.	5	1	1/2
Sept.	4	2	1
Oct.	8	3	1-1/2
Nov.	10	4	1
Dec.	15	5	2

The above tabulation is of wave heights that are being actively generated by the wind. The following tabulation is for "swell", that is, for waves which have left the generation area and are no longer under the influence of the winds which generated them.

TABLE 3.3

PERCENT OF TIME THAT "SWELL" FROM N AND

NW EXCEEDS 3.7 m (12 FT) IN EACH SEASON

Jan.-Feb.-March	15%
Apr.-May-June	8%
July-Aug.-Sept.	2%
Oct.-Nov.-Dec.	10%

TABLE 3.4

TOTAL NUMBER OF SHIPBOARD OBSERVATIONS BY SEASONS AND THE MAXIMUM OBSERVED SIGNIFICANT WAVE HEIGHT RECORDED DURING THE SERIES OF OBSERVATIONS

Season	Number of Observations	Maximum Significant Wave Observed
Jan.-Feb.-March	2044	8.2 m (27 ft)
Apr.-May-June	2916	7.6 m (25 ft)
July-Aug.-Sept.	2230	9.1 m (30 ft)
Oct.-Nov.-Dec.	1770	7.6 m (25 ft)

The above data are useful in giving a sense of the wave climate as assessed from visual shipboard observations. These data show that a significant wave height in excess of 6 m (20 ft) is not an uncommon occurrence off the coast of Portugal and that a significant wave height of 9 m (30 ft) has been reported. It should be noted that the accuracy of the visual observations is probably not high.

A third set of wave data, also shipboard observations, appears in the U.S. Naval Weather Service Command SSMO, Volume 1. For the 5° square off Lisbon, Table 18 of the SSMO shows that significant wave heights of between 7.9 and 9.8 m (26 and 32 feet) occurred in 0.2% of the observations. Significant wave heights of between 10.1 and 12.2 m (33 and 40 feet) were reported, but for less than 0.1% of the observations. No values greater than 12 m (40 feet) were reported. The data were recorded between 1963 and 1971. The SSMO report remarks that vessels usually try to avoid the areas of high waves and that, as a result, the observations are probably biased to the low side.

The wave data given are not sufficiently extensive or accurate to undertake an extreme value analysis of acceptable reliability.

Tides

The tidal ranges at Sines as shown in the tide tables are as follows:

Mean Range	6.6 ft (2.01 m)
Spring Range	8.7 ft (2.65 m)

There was an exceptionally high tide predicted for 26 February 1978, the date of the damaging storm at Sines. The predicted tides were as follows:

at 6:03 a.m., 3.75 m (12.2 ft); at 11:46 a.m., 0.58 m (1.9 ft); at 6.23 p.m., 3.63 m (11.9 ft). These 26 February elevations do not, of course, include any additional water surface elevations due to the storm itself. All the above elevations refer to local chart data.

Storm Surges

Storm surge is a function of the wind velocity, the wind fetch, the water depth, and the wind duration. If it is assumed that the wind duration is sufficient to reach equilibrium conditions, an approximation of the surge height against the shore is given by the following equation from the Shore Protection Manual, (CERC, 1977):

$$S = 0.0325 \frac{W^2F}{D}$$

in which:

S = surge height, meters

W = wind speed, kilometers per hour

F = fetch, nautical miles

D = average depth over fetch, meters.

It is assumed in this equation that winds are blowing at a right angle to the shore.

From the above, the following estimates of storm surge heights are obtained.

TABLE 3.5

STORM SURGE HEIGHT (m) FOR EXAMPLE CONDITIONS

WIND SPEED	28 knots (51.9 km/h)		34 knots (63.0 km/h)		48 knots (88.9 km/h)	
DEPTH	100 m	300 m	100 m	300 m	100 m	300 m
	0.37	0.12	0.42	0.18	1.09	0.36
STORM SURGE HEIGHT	0.74	0.25	0.85	0.36	2.18	0.72
	1.11	0.37	1.28	0.54	3.27	1.09

The above tabulation indicates that, even with a sustained wind of 48 kt blowing over a 300-nautical mile fetch, the storm surge would be only 1.09 m for an average depth of 100 m. These factors, as stated, are believed to be conservative, and thus a storm surge on the order of 1 meter is probably the maximum that could be expected.

Coastal Currents

The prevailing coastal current on the Portuguese coast is southerly, with an average persistence of 50% and an average speed of 0.4 to 0.5 knots. The British Admiralty charts indicate that in the immediate vicinity of this coastal area the currents are less than 0.25 knots for one third of the time and less than 1.0 knots almost all the time. They also show that this value is exceeded only for 1 to 2% of the total time and never exceeds 2.0 knots. Prior to the design of the breakwater, a limited field measurement program had been conducted by the General Port Department (formerly the Maritime Services). The field data collected support, in general, the observations presented above. Moreover, the local fishermen confirm that the currents are insignificant and have no dominant direction of flow.

Bathymetry

A general view of the bathymetry of the sea bottom surrounding the Port of Sines is given in Figure 3.1. As can be seen, the depth increases rapidly to 50 m at the head of the breakwater and continues to increase to the edge of the continental shelf, which is 14 km from the port. Near the coast there are numerous rocky headlands, bottom outcrops, and islands. The bathymetry in Figure 3.1 was obtained from a 1977 survey. Subsequent to the failure of the breakwater, extensive and detailed bathymetry surveys were conducted by the contractor.

Although the water is very deep off Sines, it is sufficiently shallow to affect long waves with periods in excess of 10 seconds. Refraction diagrams have been prepared by the Laboratoire Central d'Hydraulique de France (see annotation in Appendix I) and the Department of Public Works of Canada. These works show that significant wave-refraction effects occur seaward of the breakwater. For certain wave conditions, concentration of

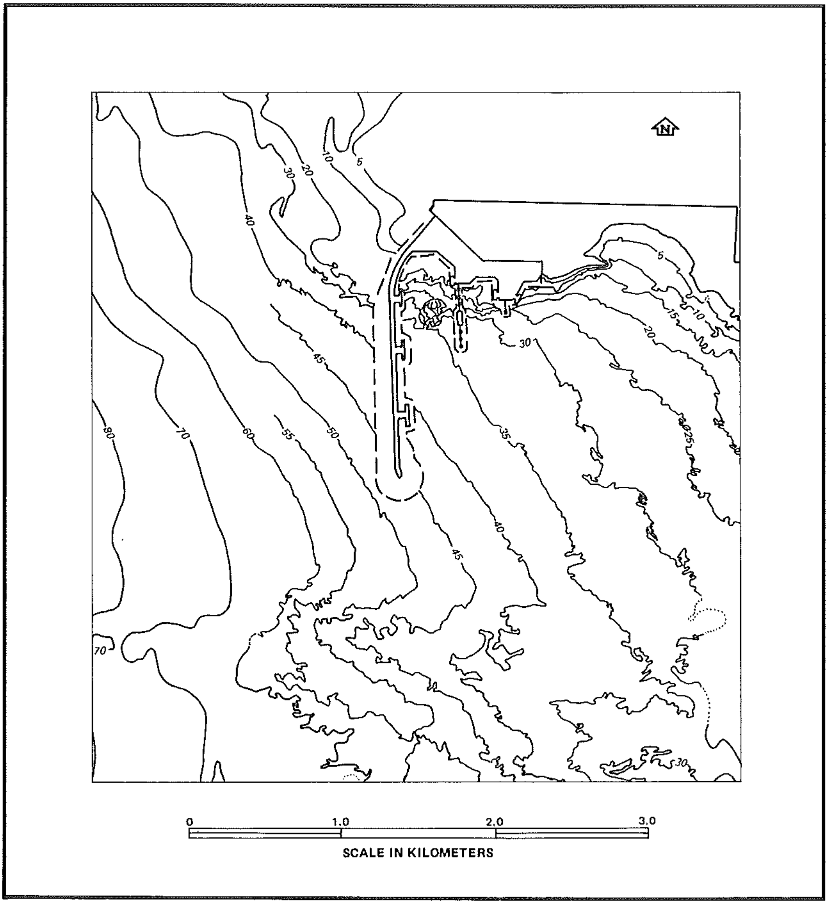

FIGURE 3.1 BATHYMETRY OF PORT SINES (METERS)

the energy of waves occurring from the west and north-west directions may occur in the vicinity of the breakwater.

Regional Geological Setting

Sines Bay is located on the south side of the Cabo de Sines, the most notable promontory of the Portuguese Atlantic seaboard between Setubal and Cabo de Sao Vincente. This headland, together with its offshore extension, Perceveira Island, is formed by a complex of igneous rocks. To the north of Cabo de Sines, the coastline consists of strongly folded grey limestones, which results in quite low ground for some distance inland. To the south of Cabo de Sines, the rocks consist of older sediments and the country rises steadily eastward from a low shoreline to the hills around Cercal.

Regional Geological Structure and Tectonics

The igneous and volcanic complex consists of a low dome which is highest at Chaos (101 meters). The dome dips gently towards the west and forms submarine features. It appears in recent geological history that a downwards movement occurred which led to the submergence of many of the volcanic outcrops under the sea. The volcanic rock outcrops are surrounded by a metamorphic aureole formed by contact with the Paleozoic shaley mudstones (south of Cabo de Sines), or with the Jurassic limestone rocks (north of Cabo de Sines).

The dome has two distinct blocks: a main larger block forming the high ground to the east of this area, and a smaller lower block forming the lower ground around the town of Sines and the Cabo de Sines itself. The dome consists chiefly of olivene gabbro and other associated intermediate volcanic rocks. The rocks are heavily fractured and cut by numerous dykes, some of which are to be found intruding the surrounding country rocks.

Faulting in the gabbro intrusion appears to have been simultaneous with the intrusions. The dykes have been intruded along fault zones in a north-east to south-west direction. Post-volcanic faulting is due to the folding which has occurred parallel to the shoreline. The dolerite dykes west of Sines have been intruded along the fractures and fault zones in an east-north-east to west-south-west direction. The dykes, east of Sines in the Terminal Mineraleiro, are generally north-east to south-west and east-north-east to west-south-west. The likely dip of the shaley mudstone is to the south, in the Terminal Mineraleiro area.

The fracturing that is present in the andesites and dolerites to the west of Sines appears to be oriented north-east to south-west and east 30 degrees south. The fractures to the south of Cabo de Sines have directions which vary between 30 degrees south to east 10 degrees south, and east 20 degrees north to east 30 degrees north.

Seismicity in the Sines-Lisbon Region

Destructive earthquakes in Portugal have been associated with the Azores-Gibralter ridge and more specifically with a fracture zone between the Azores and the southwest coast of Portugal. The great Lisbon earthquake of 1755 is an example of this type of earthquake. Shocks generated in the Azores-Gibraltar fracture zone are usually large (magnitude 7) and of shallow focus, and strike the Portuguese coast in the two distinct modes of seismic earth waves and seismic sea waves (tsunamis). The tsunami in 1755 was estimated to have a wave runnup of up to 15 meters.

Recent examples of seismic activity of the Azores-Gibraltar fracture zone occurred on November 25, 1941 (M = 8.25 and felt intensity VII) and February 28th, 1969 (M = 7.25, focal depth 18 kilometers and felt intensity VII). The 1969 earthquake, which was the most powerful in Europe since 1960, produced a tsunami which attained a height of 1.2 meters in the Gulf of Cadiz. Strong motion instruments recorded aftershocks in the Azores of 15-18% g and in Lisbon of less than 10% g.*

Seabed Surface Conditions at the Port

The seabed topography and surface conditions in this area can be described in terms of two broad zones delineated by the areas inshore and seaward of the 35 m bathymetric contour. In the zone inshore of the 35 m bathymetric contour, the seabed surface topography is quite variable, with numerous steep-sided pinnacles and ridges of igneous rock. Within this area the seabed topography is also characterized by the several seabottom rises.

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*g - acceleration due to gravity.

Inshore of the 35 m bathymetric contour, seabed surface deposits generally consist of brown, medium-to-coarse shelly sands, coarse sandy gravels with occasional boulders, and assorted sub-rounded igneous rock pebbles and boulders. These deposits may be bedrock weathering in-situ, or transported material. In general these surface deposits exist on the shallower slopes and in the hollows of the outcropping igneous bedrock. On steep slopes and points of higher elevation the bedrock is relatively clean and free of superficial deposits.

Seaward of the 35 m bathymetric contour, the seabed slopes relatively uniformly down to the southwest, reaching a minimum elevation of approximately 50 m below datum within the boundaries of the subject site. Boreholes at the breakwater show that the seabed consists of deposits of light brown, medium-to-coarse sand with some shell fragments.

CHAPTER 4

Design of the West Breakwater

The general characteristics of the main breakwater and its location relative to the port facilities are described in the preceding sections of this report. A more detailed description of the design of the cross-section is presented in this section along with a description of the design criteria and the supporting model studies. A description of the design of the main breakwater at Sines is given by Mettam (1976) and Morais (1974).

Design Wave Conditions

A description of the procedure used to determine extreme waves for the design of the Sines breakwater is given by Mettam et al. (1972). When the project began no wave recordings were available for the area near Sines, and extensive consideration was given to wave data recorded at other locations along the coast of Portugal. A summation of the recorded data available in 1972 for design purposes is provided by John D. Mettam of Bertlin and Partners, the lead design firm for the Sines breakwater:

> "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 armor units. To supplement this record an analysis was made to assess the probable wave climate at Sines from some earlier detailed observations made at Figueira da Foz over a period of 7 years." Mettam (1976).

The waverider buoy at Sines has been maintained somewhat intermittently since 1972. In addition to the offshore buoy, one is maintained within the harbor. No effort was made to hindcast wave data from available historical meteorological information.

In 1972 and 1973 GAS received reports from its consultants on analysis of wave data and derivation of extreme wave heights based on both visual observations and recorded values. The report of 1972 presented an analysis of the visual data observed at Figueira da Foz which is approximately 250 km north of Sines and taken between 1954 and 1960. In 1971 additional data were collected at Figueira da Foz and compared with the Waverider measurements at Sines to obtain a "transfer law;" this law was in turn used to translate the seven-year wave climate of Figueira da Foz to Sines. The study showed that wave heights up to 4 meters were 30 per cent higher at Figueira da Foz than at Sines. There was insufficient data for correlation of higher waves. According to Bertlin (personal communication, 1979), a detailed refraction study was carried out to relate wave heights around Sines to the Waverider there. The 1973 report included an analysis of the Waverider data from October 1971 to February 1973. This report further clarified the "transfer law" by using observations at other points along the Portuguese coast.

Thus the final design wave established for the breakwater design was based on approximately one year of wave records obtained at Sines and seven years of observations obtained at Figueira da Foz (Mettam, 1976). The procedures followed in obtaining seven years of data are described by Carvalho and Barcelo (1966). Since Figueira da Foz is located 250 km north of Sines, Mettam recommended (Mettam, 1976) that equal consideration should be given to the one year of data at Sines as to the seven year record. With these data, design specifications were established as shown in Table 4.1.

Design Development

In the initial design, consideration was given to a range of possible solutions. The main choice was between rubble mound construction and composite construction with a vertical-faced caisson structure located on a submerged rubble mound. (Mettam, 1976).

TABLE 4.1

BREAKWATER DESIGN CRITERIA (METTAM, 1976)

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

In the model tests of the breakwater cross-section both regular and irregular waves were used. For the irregular waves, a Pierson-Moskowitz wave spectrum was assumed. When regular waves were used, periods were chosen to represent a fully risen sea. These periods are given in Table 4.2.

TABLE 4.2

WAVE PERIODS FOR REGULAR

WAVE MODEL TESTS (METTAM, 1976)

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

Preliminary tests of the breakwater cross-section showed that very large wave forces would be exerted on a vertical structure. It was thought by the design firm that such forces could probably have been much reduced by the introduction of a perforated face construction. However, despite the construction problems on such an exposed site, the design consortium decided that the proposed rubble mound design would be more economical. The availability of suitable rock in very large quantities was a major consideration in this decision. In selecting the final armor design, the consortium consulted with Mr. Eric Merrifield of South Africa.

> "With the great depths at the site it was important to adopt a form of armoring 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 armor 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 test confirmed this view." (Mettam, 1976.)

A model of the Sines Harbor and all its approaches was built by LNEC. These tests, using regular waves, were used to determine the proper orientation of the breakwater and its length and to identify wave activity in the harbor.

Model Tests of Armor Stability

All model tests undertaken for the designer were conducted by the Laboratorio Nacional de Engeharia Civil (LNEC) in Lisbon. The laboratory conducted model tests to investigate the stability of the armor layer for the breakwater as well as to study wave overtopping, wave forces on the wave wall, and the stability of the armor stone below the armor layer. The stability tests undertaken in a 1.6-m-wide flume at a scale of 1:62 were reported by Vera-Cruz (1972) and Morais (1974).

In the initial design the main armor consisted of 30-t dolos on a slope of 1:1.5. In the model tests using regular waves, about 2 percent damage (displacement) was caused by waves of 11 m and with periods between 12 and 16 seconds. The first movement of dolos was initiated with 6 m waves, and displacement began with 9 m waves.

In response to the invitation for bids by the Portuguese Government, the successful bidder proposed an alternative design which retained the feature of 30-t dolos on a slope of 1:1.5 as specified in the original design. (The principal change according to Mettam, 1976, was the widening of the breakwater crest to provide space for construction equipment and for the oil pipelines, which otherwise would have required a separate structure.) The contractor had model studies performed on his proposed cross-section by the Laboratoire Central d'Hydraulique de France (LCHF). These tests with regular waves, showed damage of 0.5 to 1 percent (displaced dolos) and that 8 percent of the dolos were rocking or displaced at a regular wave height of 11 m.

LNEC performed further tests on the various designs using irregular waves. Because such significant levels of damage were noted in these tests, from 5 to 35 percent, the final design included an armor layer of 42-t dolos (often referred to as a nominal 40-t unit). According to Zwamborn (1979):

> "Tests on the final section in the LNEC irregular wave flume showed an average damage (displacement) for 9 tests of 1.2 percent for irregular waves with H_s = 10 m (reported during laboratory visit in June 1978). A few very short (1.5 hours prototype) tests with the design wave height, H_s = 11 m, were also done to study overtopping. On the basis of the results of these tests, the 40-t dolos armor was reported to be satisfactory."

It should be noted that recent investigations (1978) at a number of laboratories (LNEC, LCHF, and the National Research Council of Canada) have shown significantly different results from those described above. These are discussed in Chapter 8.

In reference to communication of the test results between LNEC and the design consortium Mettam (1979) noted:

"The designers were informed by GAS on the basis of information from LNEC that the proposed section was able to resist satisfactorily the design criteria described in [Table 4.1]. In spite of repeated requests, the designers were not shown or given any of the results of the LNEC model tests. The design consortium also requested GAS to arrange for tests at greater wave heights to investigate the final collapse condition but such testing was not ordered by GAS. Furthermore GAS did not pass to the designers full details of all the tests carried out."

Design Details

Excerpts from Mettam (1976) are given below which provide details of individual components of the breakwater:

Foundation

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

"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 60m of rubble filling. Detailed consideration was also given to the possibility of liquefaction under earthquake conditions 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 materials was placed.

Rubble Core

"The rock, which has a specific gravity [relative to fresh water] of 2.9, consists of gabbro and diorite.

"Two grades of core material were specified. The principal grade [TOT*] 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% by weight.

*TOT is the term used by the Portuguese referring to quarry-run material used in the core. TOT and select TOT are used throughout this report.

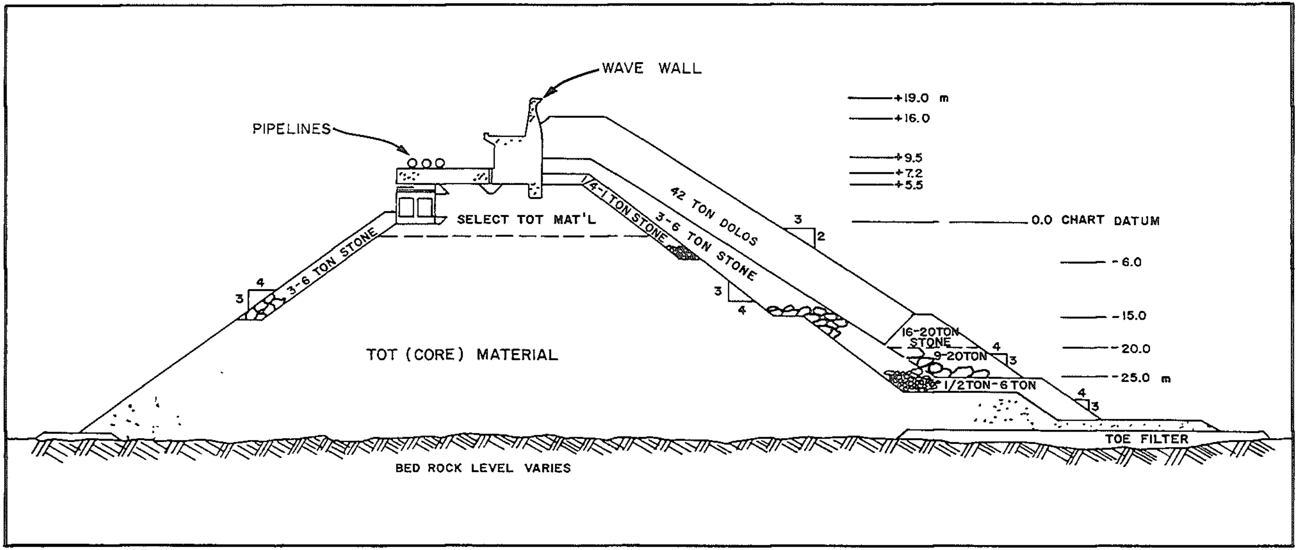


FIGURE 4.1 FINAL DESIGN CROSS-SECTION OF BREAKWATER

(Continued from Mettam, 1976)

"The 'selected' grade [select TOT*] used in the main cross section only above level -2.5m 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.

Rock Armoring and Underlayers

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

"This secondary armor 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 1/2 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.

"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 (Figure 4.1). The toe armor rests on 0.5-t to 6-t rock which also extends down the seaward face.

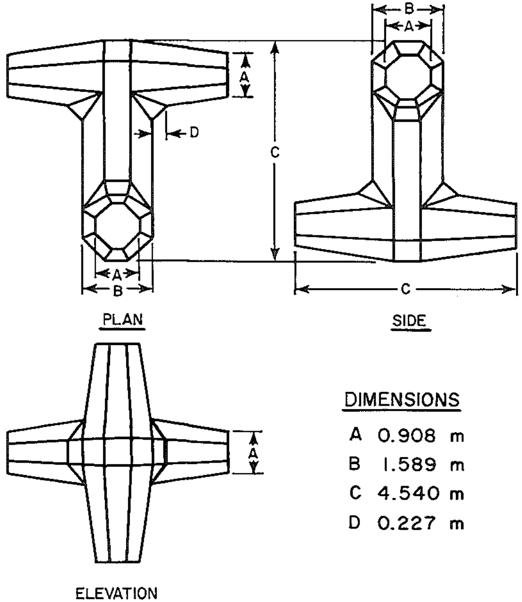
"On the rear face armor is only provided to a depth of 15 m below low water. Rock of 3-t to 6-t weight proved adequate because overtopping water is thrown clear of the vertical back face of the cap and its energy is dissipated in water.

Main Dolos Armor

"The main dolos armor units have a volume of 16.55 cu m and with a concrete specific gravity of 2.53 weigh approximately 42 t.

"The dimensions, provide a waist:leg ratio of 0.35. Fillets at the junction of the legs have dimensions of 5% of the leg length. [Figure 4.2 gives the dimensions of the dolos units.]

*TOT is the term used by the Portuguese referring to quarry-run material used in the core. TOT and select TOT are used throughout this report.



DIMENSIONS

A	0.908 m
B	1.589 m
C	4.540 m
D	0.227 m

FIGURE 4.2 DIMENSIONS OF THE DOLOS UNITS

(Continued from Mettam, 1976)

"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.... A compressible joint in the moulds reduced the tendency for shrinkage to put the trunk into tension.

"The design of the armor face was based upon the recommendations of Zwamborn and Beute (1972) and 155 units were specified per 1,000 sq m of armor 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 armor 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 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, armoring recommended by Waterways Experimental Station 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 1:1-1/2 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.

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 earthquakes is of course acceptable provided that the main structure survives."

CHAPTER 5

Construction at Sines Breakwater

The construction of the Sines Breakwater was initiated in May 1974 by the Italian firm of Condotte d'Aqua. The work was essentially complete, except as noted in the following paragraphs, when the storm of February 1978 occurred. The breakwater construction procedures and theory were translated from excerpts of a paper published in Italian by Paolella and Agostini (1978). Paolella and Agostini describe the theory of dolos armor protection as follows:

> "The way that the dolos work is very simple. One must abide by the precaution that they do not move. Weight is the factor that connects the elements. In addition, at least 60% of the dolos have the vertical leg perpendicular to the face of the breakwater at the seaward end; this in fact adds further reinforcement. In such positions, the dolos in addition to fitting together to create a large net, have less tendency to roll about. This strategy is of greatest importance in placing the first row of dolos that constitutes the support for the successive dolos.
>
> The dolos are manufactured of concrete for higher resistance. Dolos that break during placement are removed from the structure; however, it is not desirable to remove those dolos that break due to storms. This could damage equilibrium already established.
>
> It is not necessary to add steel reinforcing to the units because this does not prevent rotation of the arms. The presence of steel, under attack of seawater, would hasten deterioration of the surrounding concrete."

It is important to note that recent research in the UK and Norway has shown that corrosion of steel in submerged concrete of high quality due to cracks is of negligible concern. Hence the arguments against reinforcement of dolos because of potential corrosion must be limited to those situations which are exposed in the tidal and splash zones.

Construction (translated from Paolella and Agostini, 1978)

For the construction of the Sines breakwater, the designers had specified dolos, as described in the previous section; the cover layer of the breakwater consists of 40-t dolos, while the root and the inside of the head are made of 15-t dolos. The core is built of quarry stone (TOT) with rock weighing up to 3000 kg. The core is protected by a layer of select stone weighing 3000 to 6000 kg with a slope of 3:4.

The exterior of the core* is protected by a first layer, with slopes of 4:5 and stones ranging from 250 to 1000 kg and by a second layer, slope 2:3, with stones from 3000 to 6000 kg resting on a toe composed of elements weighing 500 to 3000 kg. (The cross-section of the breakwater is presented in Figure 4.1).

The density of the dolos placement is 0.16 dolos/m^2 for the 40-t units and 0.318 dolos/m^2 for the 15-t units. It was planned, therefore, to use 21,600 40-t dolos and 3180 15-t dolos.

The seaward layer of dolos is placed to maximum depths of -25 m and is supported on a layer of toe stone varying from 9000 to 20,000 kg.

The stone was obtained from a quarry, located about 5 km from the breakwater; the absolute specific weight of the quarry materials is between the values of 2.88 and 3.00 t/m^3. The width of the base of the breakwater is obviously a function of the depth, and varies from 45 to 231 meters.

Cellular rectangular reinforced concrete cofferdams, filled with quarry-run material, with base at -0.5 m, form the inner side of the breakwater. The purpose of the cofferdams is to protect the breakwater from overtopping. The cofferdams constitute the internal support of the superstructure.

The construction work was completed in the following phase:

(1) Core material was barge-dumped in the middle of the core to form the berm, up to elevation -15 m in the winter, and to -10 m in the summer.

(2) The remainder of the core was placed by direct dumping with 50- and 65-t dump trucks.

(3) The armor stone was placed in different layers by a land-based crane (of 1,000-t) and by a barge-mounted crane (of 1,400-t).

---

*The weights of the elements comprising the layers below the dolos were determined by the following formulas:

1° layer = $W_1 = W_r$

2° layer = $W_2 = W_1/20$

n° layer = $W_n = W_{n-1}/20$

(4) Dolos were placed from the barge to breakwater elevation of 4 m.

(5) Dolos were placed from 4 m to 8 m by land-based crane.

(6) The key of the superstructure foundation was constructed.

(7) Dolos were placed from 8 m to 12 m by land-based crane.

(8) The concrete superstructure was constructed section by section.

(9) The layer of dolos was completed by land to an elevation of 16 m.

Phases 2, 3, 4, and 5 were suspended during the period from December to April in 1975 and 1976; during this time, a temporary head was built for protection of the completed work.

In conjunction with the above-mentioned phases and based on the construction schedule, 36 dolos were manufactured per work day. These were stored near the site in a yard with capacity of 1,000 dolos.

Two work areas were constructed which were served by two land cranes of 50-t and 12-t capacities, with booms of 22 m and of 20 m, respectively. The casting yard for 15-t dolos (located at the root of the breakwater and elsewhere in the harbor) was similar to that for the 40-t dolos, but used cranes of 25-t and 8-t capacities with booms of 18 and 16 m, respectively.

The dolos were constructed with the shaft in a vertical position and legs parallel to the ground. The mold was composed of a movable upper part and a fixed lower part. The latter was constructed from the bottom of the lower leg of two faces of the shaft and of the two lateral legs of the upper section.

The molds were arranged in two parallel rows of 18 units each. Cranes were used to move the upper half of the molds row to row. Every site had available 18 mobile molds and 36 fixed molds. Vibrators on the walls of the molds provided compaction of the concrete.

The progression of work with respect to the 40-t dolos was as follows:

<u>1st Phase</u>: Using the 12-t crane, the mobile part of the mold was placed on the corresponding fixed mold. The fixed part and the mobile part of the mold were connected by bolts. At the middle of the mold were connections for the jacks used for opening the halves of the fixed mold.

2nd Phase: After the two parts of the mold were bolted together, it was placed in the middle of a conveyor that carried the concrete up and into the mouth of the mold.

3rd Phase: The dolos were removed with the 50-t crane and placed on the outside of the site. The crane had two extensions that allowed lateral operations.

4th Phase: After about 5 days outside the mold, the dolos were loaded by crane on a special truck and carried to the curing site.

5th Phase: After the dolos cured (28 days minimum), they were transported to the breakwater to be collected for land-based operations, or to the barges for use in sea-based operations.

The mold was stripped when the dolos reached a resistance of 150 kg/cm^2 (2134 psi), about 48 hours after casting. When lifting the dolos with steel slings checks were made for cracks that might have led to damage of the concrete. For placement of the dolos in the storage site, a tong-type mechanism which grasped the dolos along the shaft was used.

The concrete used for the dolos was type B400 with a 28-day compressive strength of not less than 400 kg/cm^2. The forms had a volume of 8 m^3. The twin batch plants had a capacity of 60 m^3/hr.

Particular attention was given to the manufacture of concrete to reach the high compressive strength required and the correct density. The results were satisfactory. The average compressive strength was 539 kg/cm^3 (7668 psi), the progressive specific resistance was 469 kg/cm^3 (6672 psi), and the coefficient of variation from the mean was 8. Progressive specific weight of the dolos was 2.25 t/m^3 (159 lb/ft^3).

Tractor-trailers were used for transport; these had an opening in which a leg of the dolos could be placed for stability. When arriving at the destination for unloading, the dolos were oriented with the shaft in a horizontal manner so that the crane would not have trouble unloading them.

For the placement of dolos by the floating crane, two self-propelled pontoon barges with 188 HP motors, each having a capacity of 800 t, were equipped with A.H. M25 Revolver Cranes. Each barge could transport 18 40-t dolos. The barges used could function in waves of 2 m and carry 250 dolos per week, amounting to 40 m of breakwater per week. The completion of the breakwater, as previously stated, was executed by land with a crane of capacity 1,000-t.

During the placement of the dolos, the breakage, on the average, was 1.3%, and 0.1% during transport. In the course of work the breakwater was attacked by eleven storms with maximum wave heights varying from 6.9 to 9.3 m.

The marine work was begun in May 1974 and was completed in August 1977. The completed structure contained 12% greater density of dolos than were originally specified.

[Editor's Note: end of material translated from the Italian Report.]

State of Breakwater Prior to 26 February 1978

The construction procedure used for the breakwater as recorded by the contractor was discussed in the previous section. Details of the history of the construction up to the time of the accident as presented in GAS documents are presented in this section. The outer protection of the breakwater was nearly complete by the time of the February 26, 1978 storm.

Wave Conditions

As indicated elsewhere, the wave conditions at Sines are seldom calm. During construction a record of wave activity was made. The maximum and minimum levels of wave activity are given by week for the years 1974-1978 in Appendix L. The values presented in these tables are not significant wave heights but instantaneous maximum values. The lowest values are the smallest values of the maximum wave height recorded in any one record during a week of observation. It can be seen from these data that the maximum wave height exceeded 8 m and occasionally 9 m during construction of the breakwater. Zwamborn (1979) reported that between 3.5 and 16.8% of the dolos in a sampled section were broken by August 1977 due to either placement, settlement or storms.

Progression of TOT

Placement of the quarry-run material (TOT) in the core began in November 1974 and was completed during early summer of 1977. Figure 5.1 shows the progression of the core of the breakwater by month. (Inspection of the figure also shows the winter head of the breakwater). A temporary winter head was placed on the structure in 1974, 1976, and 1977. By the winter of

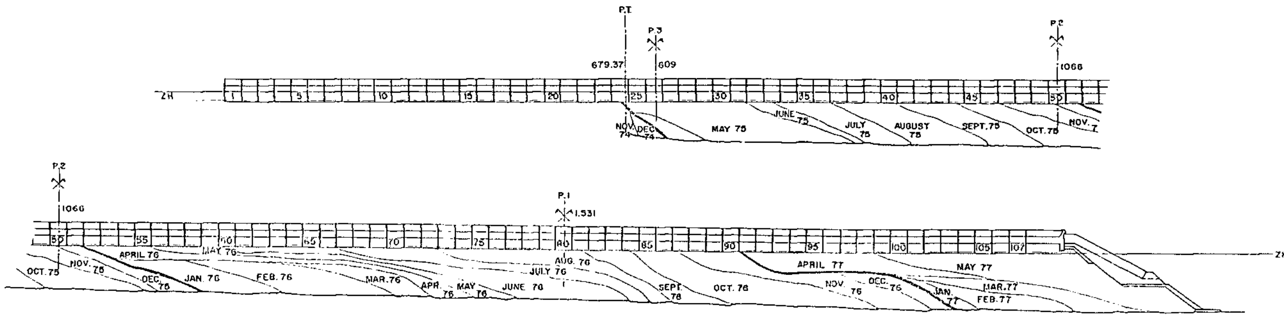


FIGURE 5.1   PROGRESS OF CORE CONSTRUCTION BY MONTH

1978 the permanent head was in place. Tables in Appendix L give a record of the placement of all quarry material on the breakwater during 1976-1978. The tables are complete through March of 1978. The quantities in the tables are in hundred thousand tons (metric) and are the actual emplaced values for which the contractor was paid. Data for 1974 and 1975 were not obtained from Gabinete. As noted in the tables, the select TOT (a material similar to the ordinary TOT but with few fines) progressed very closely behind the ordinary TOT.

Progression of Superstructure

The initial pour began in March 1975. The final pour had not been made at the time of the storm. Because of the mild maritime climate, concrete was placed throughout the year.

Placement of Dolos

Dolos were first placed in September 1974. They were placed by land-based and floating cranes as described in the section covering construction procedure. The rate of placement (dolos/day) varied greatly depending on the execution of other phases of the project. Placement occurred only when the waves were less than 2 m. The maximum placement by sea was 126 per day while that from land was 112 per day. A complete record of dolos placement by day from 1974 to 1978 is given in Appendix F. A summary is given in Table 5.1. The only dolos placed in 1978 (before the February storm) were replacements for broken armor units; whether these were broken during placement or by waves is unknown. From the tables the total armor units placed were 10,369 by sea and 9,433 by land. This gives a total of 19,802 placed dolos.

Strength tests were routinely made for the concrete used in the dolos. The tests were performed by the Fiscalization Department of GAS; a sample of the strength tests is given in Table 5.2. These tests were made in 1977 and are similar to other years of production.

Missing Stone and Dolos

At the time of the February 26, 1978, storm some construction still remained to be done near the head and at berths 1 and 2. The superstructure was incomplete at the head (the wave wall had not yet been poured) and some 3-6 ton stone and dolos were missing. The locations of missing 3-6 ton stone are shown in Figure 5.2. The absence of dolos in front of the wavewall is shown in Figure 5.3. A summary of the missing dolos is given in Table 5.3.

TABLE 5.1 SUMMARY OF NUMBERS OF DOLOS PLACED EACH MONTH

(GABINETE DA AREA DE SINES)

		Year				
		1974	1975	1976	1977	1978
January	Sea			118		42
	Land		182	325		
February	Sea		36	102		14
	Land		200	221	197	
March	Sea		78			
	Land		303	199	102	
April	Sea		108	94	329	
	Land		314	70	275	
May	Sea		128	564	890	
	Land		117	250	347	
June	Sea		393	442	573	
	Land			385	291	
July	Sea		724	770	868	
	Land		178	317	248	
August	Sea		324	608	467	
	Land		319	457	601	
September	Sea		441	404		
	Land	75	266	231	409	
October	Sea		229	312		
	Land	478	145	134	75	
November	Sea		255	491		
	Land	542	126	67	235	
December	Sea		500	124		
	Land	391	197	161		

TABLE 5.2 RESULTS OBTAINED FROM COMPRESSION TESTS ON CONCRETE CYLINDERS MADE FROM DOLOS USING B400 BD 11 CONCRETE AT 28 DAYS IN 1977 (GAS)

Average Strength

Month	Strength (pascal)
January	5501
February	6100
March	5355
April	5247
May	5237
June	5247
July	5090
August	4688
September	4560
October	4384
November	*
December	*

*Dolos were not cast in these months.

Values Obtained During the Period 6/2/77 to 6/18/77

Standard Deviation	369.3
Coefficient of variation	82.3
Specific Strength	4519.8
Average Strength	5256.6

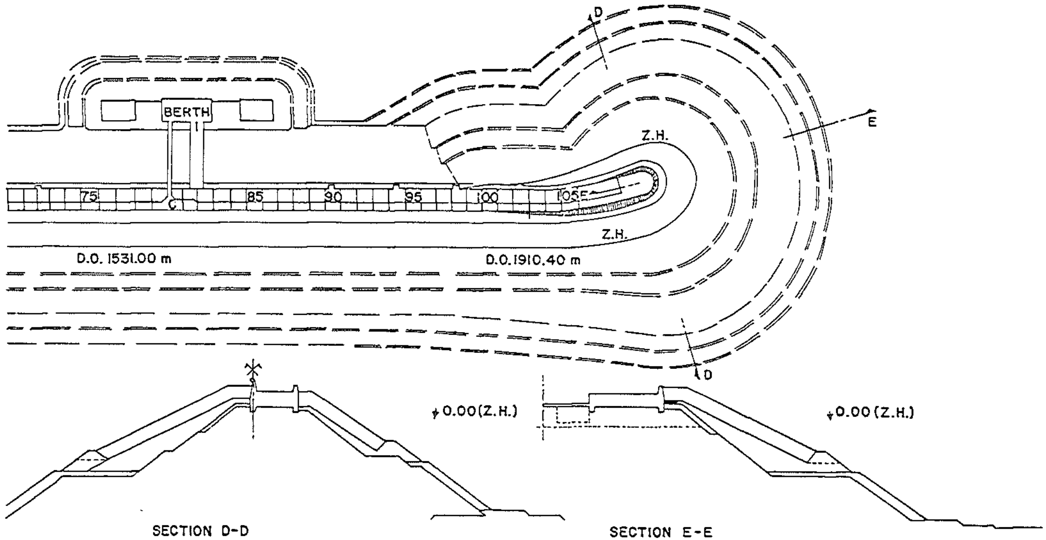


FIGURE 5.2 LOCATIONS OF MISSING 3-6 TON STONE AT TIME OF STORM

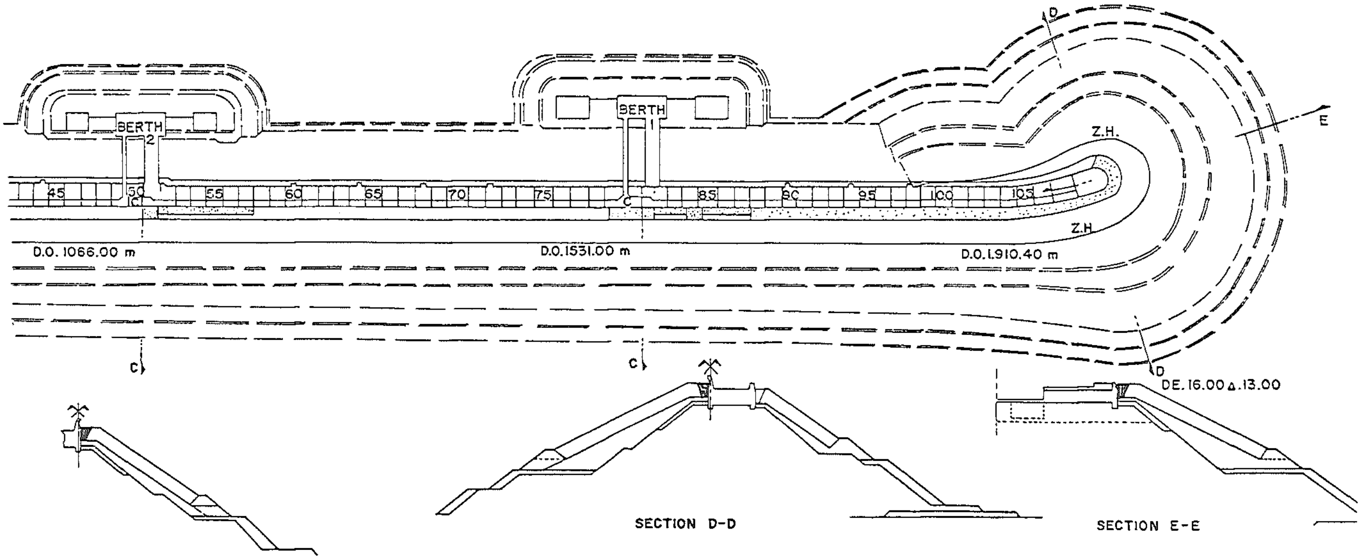


FIGURE 5.3 LOCATIONS OF MISSING DOLOS AT TIME OF STORM

TABLE 5.3

UNPLACED DOLOS ON 26 FEBRUARY 1978

Caisson number	Unplaced Dolos
51	15
52-56	30
72	15
78-81	45
82-83	26
84	15
85-87	26
88-end	as many as necessary in final phase

Cross-Section of Completed Breakwater

As construction progressed, profiles of the structure were made above and below the water surface. The available cross-sections are given in Appendix G. These cross-sections below the water surface give the top elevation of the 3-6 tone filter layer and the surface of the toe of the structure. Profiles of the final dolos layers were not made. These, of course, would be difficult not only to measure but also to interpret.

Additionally, longitudinal surveys of the structure were made regularly to monitor settlement of the superstructure. Figure 5.4 gives a longitudinal profile of the structure before the storm. It is compared in Chapter 6 with the conditions immediately after the failure of February, 1978.

Construction Inspection

In addition to the official onsite job inspection by the Gabinete, the design consortium of Bertlin-Consulmar-Lusotecna made visits to the site for their personal inspection of the construction. Reports were frequently submitted to the GAS, recording deviations from the design and shortcomings in the construction which they perceived. They also expressed their concern at the construction supervision being carried out by GAS and at what they perceived to be a lack of record keeping. One item in particular was the lack of continuous underwater supervision--a difficult task at best considering the wave climate. Some of these reports are summarized or

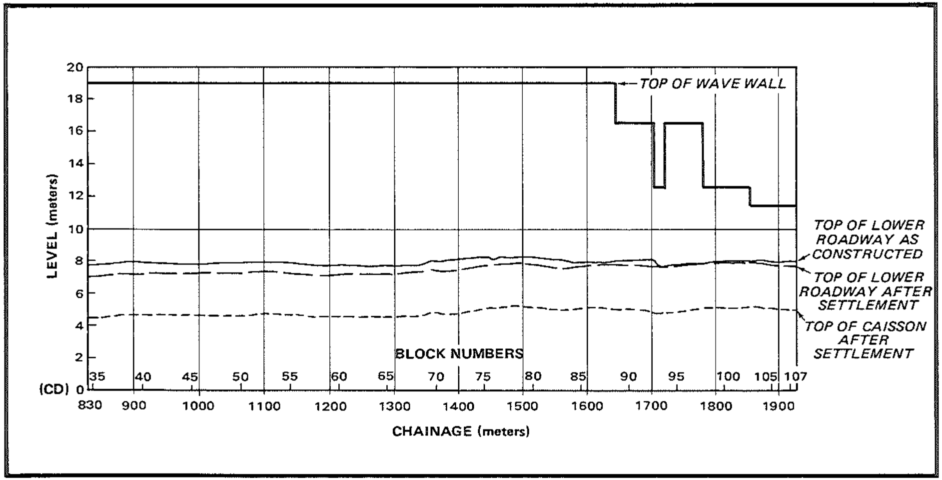


FIGURE 5.4 LONGITUDINAL PROFILE OF SUPERSTRUCTURE BEFORE STORM

referenced in their report to the Official Portuguese Investigating Committee. An English translation of that document is included as Appendix A. The reader is referred to that Appendix for review of the report filed by Bertlin-Consulmar-Lusotecna.

It should be noted that the design firm has publicly stated (Mettam, 1979) that a number of differences between the constructed breakwater and the design specifications exist. Furthermore, Bertlin and Partners maintain that there were deficiencies in the construction. These allegations are denied by the contractor and have not been discussed by the GAS. Specifically, the following items have been noted by the design firm as possible or known deficiencies that may have contributed to the failure: "incomplete wavewall, a dolos wedge unplaced, secondary armor omitted, secondary armor placed too low, dolos incorrectly laid, selected TOT too fine, position of winter head (may not have been fully removed), secondary armor wrong size, toe armor (wrong placing), armor layer (wrong placing), TOT grading too fine, TOT toe protection (wrong placing), filter layer ommitted."

There is further discussion of these points in Appendix A and in Appendix B presenting the contractors rebuttal.

The design firm has also publicly criticized project management at Sines, the organizational structure, and the lack of provisions for inspection by the engineering firm. John Mettam (Anon., New Civil Engineer, June, 1979) cited the fact that during the design phase all relevent information flowed through the client (GAS) rather than through the engineer. The firm thus had no studies, or analysis of wave records. Instances were cited of verbal transmittal of important information, particularly information having to do with the results of the hydraulic model tests conducted by LNEC. Project management in the construction phase was similarly criticized and the exclusion from inspection and supervision of the work cited. Figures 5.5 and 5.6 illustrate the design and construction relationships between those associated with the project. "Enormous relaxations" of the specifications were claimed; some of the specific items are outlined in the previous paragraph. Mettam visited the site at the expense of Bertlin and Partners and wrote to GAS about observed "shortcomings" in the work. The GAS on-site complement of engineers were described by Mettam as being

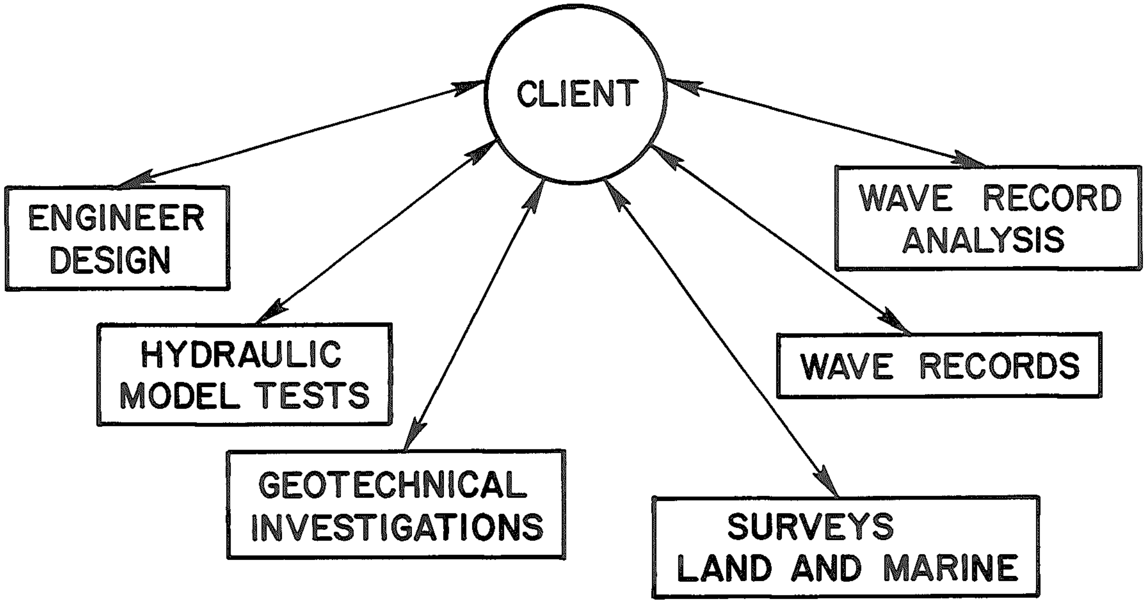


FIGURE 5.5 PRINCIPAL RELATIONSHIPS FOR THE SINES PROJECT DURING THE DESIGN PHASE

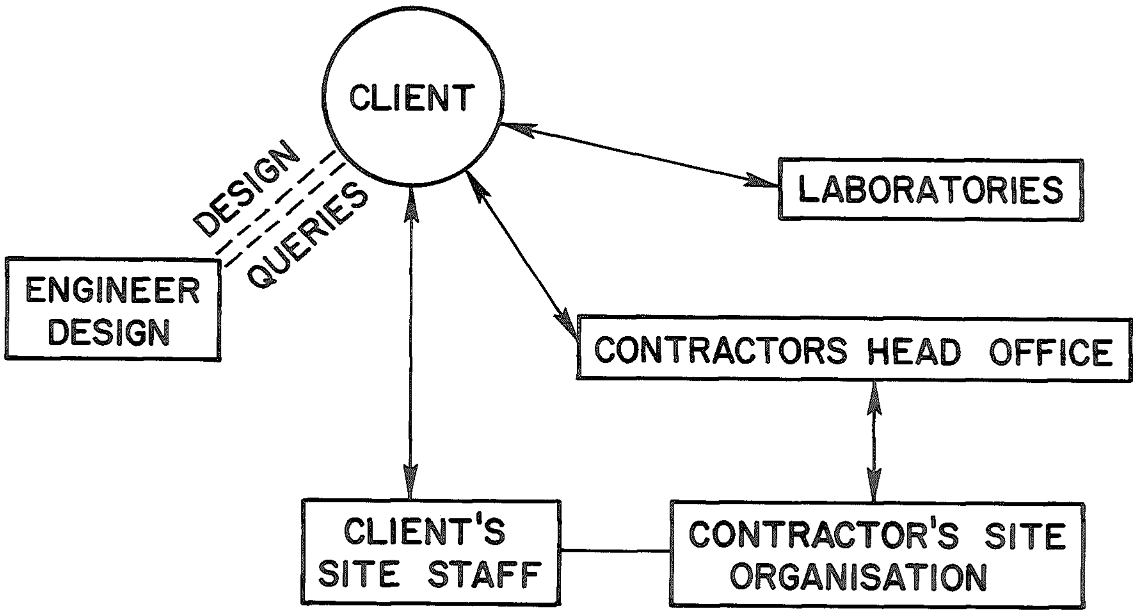


FIGURE 5.6 PRINCIPAL RELATIONSHIPS FOR THE SINES PROJECT DURING THE CONSTRUCTION PHASE

inexperienced as well as too few in number.

Again, GAS has remained silent on these allegations.

The contractor, Condotte d'Acqua, responded to some of these claims in the statement to the Official Portuguese Investigating Committee.

> "Globally examining the document of the Consultants we may conclude that the remarks therein are extremely oversimplified, not to use stronger word.
>
> "They consist in ascribing to the execution some defects that not even the Consultants themselves know if they exist - if they knew, they would not call them "probable" or "possible" - and, although not putting it in writing, they want to make believe that all those pseudo defects of execution were the cause of the accident. However, they did not even attempt to demonstrate, defect by defect, what was their possible influence in the accident as it occurred.
>
> "And, on the other hand, they completely ignored all the circumstances - in addition to such pseudo defects - which could have caused the accident. Thus, they were not interested in knowing what had actually been the condition of the sea during the storm period; they did not take into consideration the tests in model performed at the laboratories in Lisbon (L.N.E.C.), of Canada (National Research Center) and of Paris (L.C.H.F.); they did not take into consideration whatsoever the qualified opinions of experts recognizedly experienced in maritime works in deep waters. For them everything comes down to throwing doubts on the Inspectorate and the execution of the works in an arbitrary and oversimplified manner."

The complete statement, translated from the Portuguese, is in Appendix B.

Chapter 6

The Storm of 26 February 1978

A series of major low pressure disturbances were located off the coast of Portugal during the latter part of February 1978. The first reached the mid-Atlantic about February 20 or 21 and the second followed about 3 or 4 days later. The wave action accompanying this storm was described briefly in the Mariner's Weather Log (May, 1978)*. At noon on the 24th, the second center was at about 48° N, 37° W, with the vessel Cheshire at 43° N 38° W reporting 12.5 m seas and 20.1 m swells. At 1800 on the 25th, the vessel, Moreton Bay, at 43° N, 21° W, reported 18.0 m swells at 1800. The Moreton Bay was at that time about 580 miles west of Sines. The major damage to the Sines breakwater is reported to have taken place on the afternoon of the 26th.

In addition to the descriptive information found in the Mariner's Weather Log, wave conditions off Sines have been estimated from meteorological conditions and compared with visual observations made from passing weather ships reporting to the United Kingdom Meteorological office. A report prepared for the Port Sines Investigating Panel by Mr. Laurence Draper of the U.K. Institute of Oceanographic Sciences (IOS) is given in Appendix D. The U.K. Meteorological Office prepared a summary of the wind-fields over the area between 5° and 40° W and between 30° N and 50° N. From their results wave hindcasts were prepared using the IOS oceanic wave-prediction graphs, and the swell component was estimated based on an analysis of swell decay by the London Weather Center. However, it was determined that the effect of swells arriving at the coastline at the same time as the storm-generated waves was very small. The U.K. Meteorological Office has listed visual estimates of wave conditions within 3° longitude west of Sines, and these are presented in Appendix D. It is emphasized that these visual estimates, along with the visual observations referenced from the Mariner's Weather Log, must be treated with considerable caution.

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*Environmental Data Service, National Oceanic and Atmospheric Administration Volume 22, No. 3, May (1978).

Results of these analyses are presented in Figure 6.1. This figure presents a comparison of visual observations of waves and wave recordings obtained at Cabo da Roca with the hindcasted values for Sines. The "broken" visual line is used to connect observations which were taken at intervals larger than six hours. The wave recordings at Roca are used for comparison, since the Datawell Waverider at Sines was not operating during the peak of the storm. (The details of the measured wave data from Cabo da Roca and Sines will be described below.) Comparison of the hindcast, visual, and waverider data show reasonable correlation for the maximum significant wave on February 26. Based on this correlation, the statistics of the wave data from the waverider at Roca are used to infer the wave statistics at Sines. From the figure, the storms preceding and following the peak on February 26 are very obvious. On both February 23 and 25 the waves had a significant height of 5 m and again on the 28th the significant wave exceeded 5 m. Based on this figure it can be assumed that the maximum significant wave height offshore of Sines was at least 8 m.

A cursory analysis by the U.K. Meteorological Office gave the following return periods for windspeed off the Portuguese coast from Sines.

TABLE 6.1

RETURN PERIOD FOR WINDS WEST OF SINES

Wind Speed (kt)	Return Period (years)
62	50
56	10
48.5	1

The analysis by the U.K. Meteorological Office showed that although winds in excess of 60 km were present offshore, these winds did not persist long at Sines if they existed at all. According to Draper:

> "It may be assumed that whilst the wind-field at or near Sines may not have been worse than, or may even not have achieved, the 10-year level, the swell arriving at the coast would probably have been more severe than the Sines winds indicated, but that there is no firm evidence on which to base an objective assessment of the rarity of the event. It is possible that the average return period of the waves was of the order of 10 years."

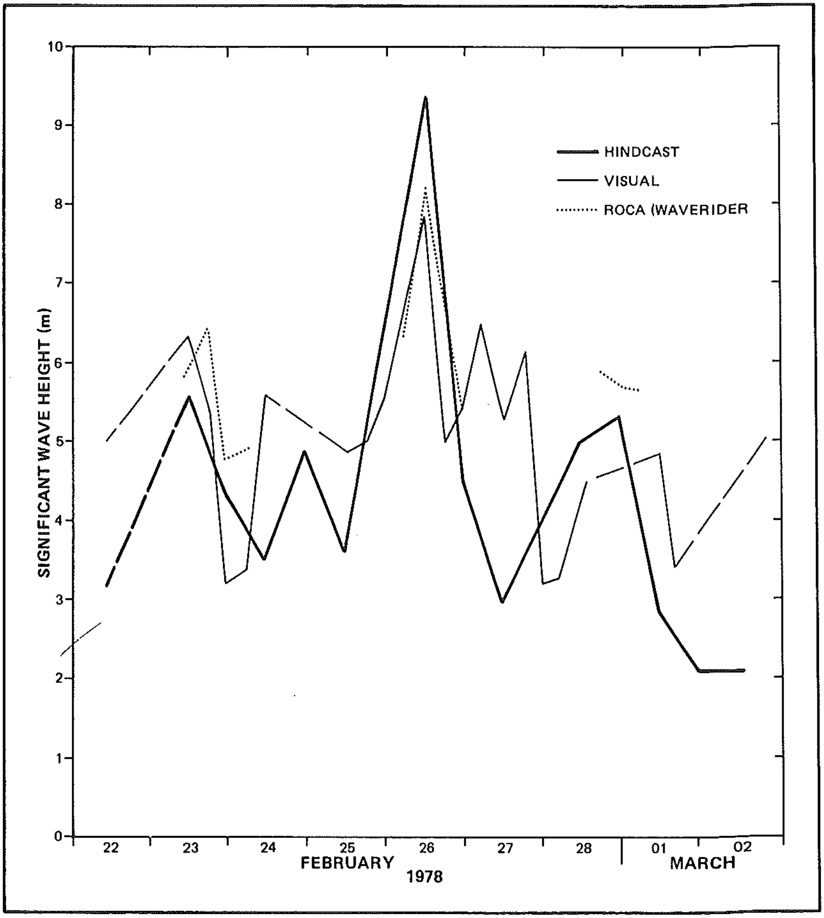

FIGURE 6.1 COMPARISON OF HINDCASTED WAVES WITH THOSE OBSERVED FROM PASSING SHIPS AND THOSE MEASURED AT CABO DA ROCA FROM FEBRUARY 22 TO MARCH 2, 1978

Before, after and during part of the storm of February 1978, two Datawell waverider buoys recorded the waves inside and outside the main breakwater. The outside buoy was located one km southwest of the head in 64 m depth. At Sines the waverider outputs were programmed to be recorded graphically with records of 20 minutes for the outside site and 30 minutes for the inside site at 6 hour intervals. Unfortunately, the outside waverider did not function during the most intense part of the storm. At Cabo da Roca (approximately 100 km north of Sines) a waverider was able to record wave data throughout the storm. The signal from the buoy at Cabo da Roca recorded for 20 minutes every 6 hours. In addition, on the 26th at 1630 hours a 90-minute record was made at the Cabo da Roca site.

These data were analyzed by LNEC using the Tucker-Draper Method of analysis for the Sines data. For Cabo da Roca data the Tucker-Draper analysis was augmented by spectral analyses. A summary of maximum wave height in excess of 7.0 m is given in Table 6.2. Correspondingly, significant wave heights were determined. These results are presented in Table 6.3 for significant wave heights in excess of 5.5 m. In Table 6.4 values of mean periods from zero upcrossing analysis are given for records with T_z greater than 10.0 sec. Evidence before and after these storms indicates that the generating conditions were such that the observations at Roca could be translated to Sines with some confidence. The actual recordings at Sines before and after the instrument failure there on 25-27 February show good agreement between the outside gauge and the gauge at Cabo da Roca. Therefore, it has been assumed that the data presented in Tables 6.2-6.4 are a reasonable estimation of the wave activity offshore of Sines. An indication of the reasonableness of this assumption can best be seen in Figure 6.1 presented previously, comparing the significant waves at Cabo da Roca with the visual observations and hindcasts. The significant wave heights presented in the figure are based on a Tucker-Draper method of analysis and are somewhat different from those given in Table 6.3. During the maximum intensity of the storm, the maximum significant wave was about 8.5 m with a peak period of 18 seconds.

A history of the wave heights during the storm was synthesized for Sines from the records at Cabo da Roca. This is shown in Figure 6.2. Note that all times given are local times. A sample of the wave record of

Cabo da Roca is shown in Figure 6.3. The data of Cabo da Roca have not been treated to account for any refraction that may exist there. (LNEC had some refraction diagrams made for Cabo da Roca. The data from Cabo da Roca, unfortunately, did not by any means represent offshore conditions. November 1979 correspondence from LNEC.)

Table 6.2

MAXIMUM WAVE HEIGHTS* DETERMINED

FROM TUCKER-DRAPER METHOD AT CABO DA ROCA

H (m)	Day	Time
8.88	23 Feb. 1978	1200 hours
9.60	23	1800
7.33	24	0600
7.90	26	0000
10.40	26	0600
11.37	26	1200
10.40	26	1630
10.23	26	1800
8.45	27	0000
7.40	27	0600
7.07	28	1800
8.83	1 Mar.	0000
8.40	1	0600
7.82	1	1200

*In excess of 7.0 m.

An independent analysis of the wave data was made by the Laboratoire Central d'Hydraulique de France. In addition, they hindcasted the waves that existed at Sines during the February 1978 storm. The hindcast effort gives an incident deep-water significant wave of 6.25 m on the 23rd and 10 m on the 26th. The corresponding wave periods were calculated to be 13.8 sec and from 18 to 20 sec, respectively. LCHF suggests that during the peak of the storm the following conditions prevailed (See Appendix D):

H_s = 9.5 to 10 m (significant wave height)

H_{max} = 14 to 17 m (maximum wave height)

T_z = 12 to 14 sec (Zero-crossing period)

T_p = 18 to 20 sec (maximum, peak period)

Additionally LCHF performed numerical and model refraction studies. The refraction analyses showed that in certain areas of the breakwater the refraction analysis for westerly waves with 16 to 22 sec periods increased the wave height on the average by 20%, with maximum values of 50%. In the area opposite berth number 2 the waves were reduced up to 15% from the incident wave conditions.

Table 6.3

SIGNIFICANT WAVE HEIGHTS IN EXCESS OF 5.5 m

BASED ON TUCKER-DRAPER ANALYSIS AT CABO DA ROCA

H (m)	Day	Time
5.56	23 Feb. 1978	1200
6.39	26	0600
8.06	26	1200
6.60	26	1630
6.62	26	1800
6.04	1 Mar.	0600

Table 6.4

VALUES OF MEAN PERIOD FOR WAVES

WITH T_z GREATER THAN 10 SEC AT CABO DA ROCA

T (sec)	Day	Time
12.24	23 Feb. 1978	1200
10.00	23	1800
12.24	26	0600
12.63	26	1200
11.21	26	1630
10.43	26	1800
10.86	1 Mar.	0600

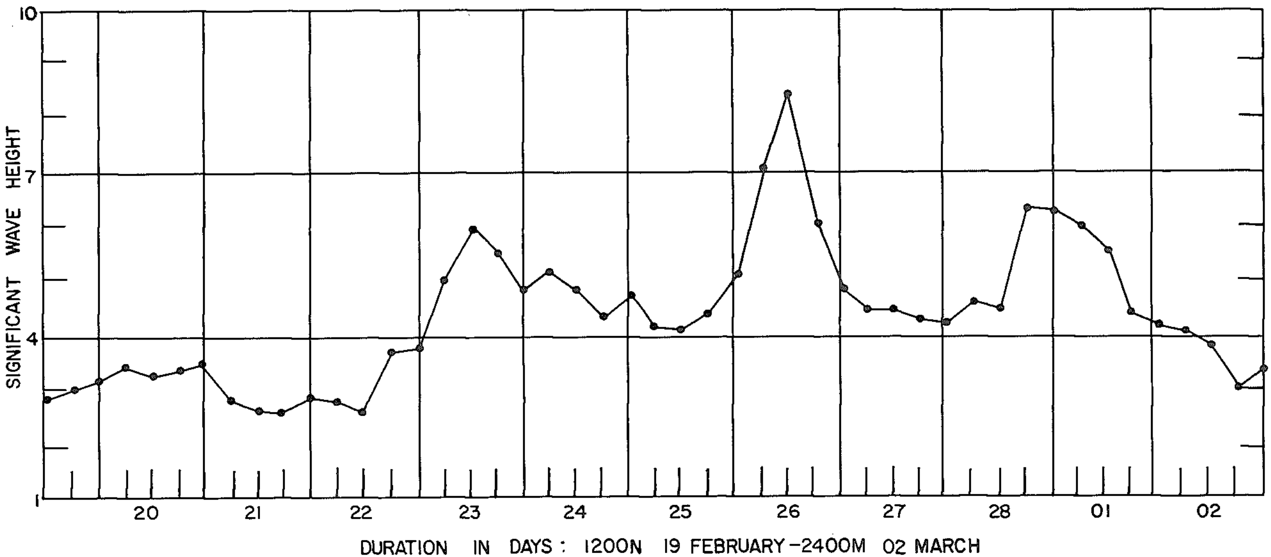

FIGURE 6.2 SYNTHESIS OF WAVE RECORD AT SINES FROM RECORDED DATA AT CABO DA ROCA (MANSARD AND PLOEG, 1978)

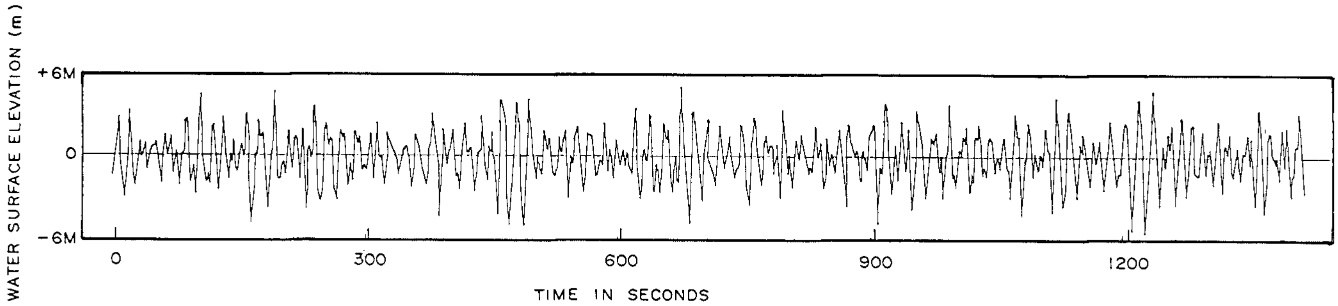

FIGURE 6.3 SAMPLE OF WAVE RECORD FROM CABO DA ROCA ON FEBRUARY 26, 1978

CHAPTER 7

Status of Breakwater After Storm

Because of the magnitude of the storm on February 26, 1978 and the storms that followed immediately, no damage assessment could be taken for several days. It is possible that some additional damage may have occurred during the following storms, since these had significant wave heights of up to 5.5 m. Assessment of the damage on the Sines breakwater began the first week in March. A summary of the observations made at that time are included in this section.

Cross-Section of the Breakwater

Perhaps the most revealing data of the failure is contained in the cross-section profiles of the structure made in mid-March 1978. The profiles made after the storm have been compared with the design section and the profile measured before the storm. These measurements were made by divers and taglines. These profiles are shown in Appendix G. A summary or typical section is presented in Figure 7.1. Unfortunately no profiles were available of the final section before the storm; only the profiles made before the dolos were placed are available. Only in a few cases did the underwater surveys extend down to the base of the structure or up to the superstructure. The location of each section is keyed to the corresponding caisson.

Additionally, longitudinal profiles were made on the structure. These profiles were made at three points on the superstructure: the top of the caisson supporting the superstructure, the top of the lower roadway, and the top of the wave wall. The profiles are shown in Figure 7.2.

Underwater Investigations

The surveys made by the Portuguese Navy are presented in Appendix G. Their surveys consisted primarily of visual observations of bottom material and depth soundings. No official underwater photographs were made. However, an employee of the contractor did make several dives and filmed on Super 8 mm the damage to the structure below water. These films showed numerous broken dolos at all depths. Two members of the panel, Orville T. Magoon and Billy L. Edge, also visited the site in August 1978 and dived on the structure with the Navy divers. The divers made a descent along the

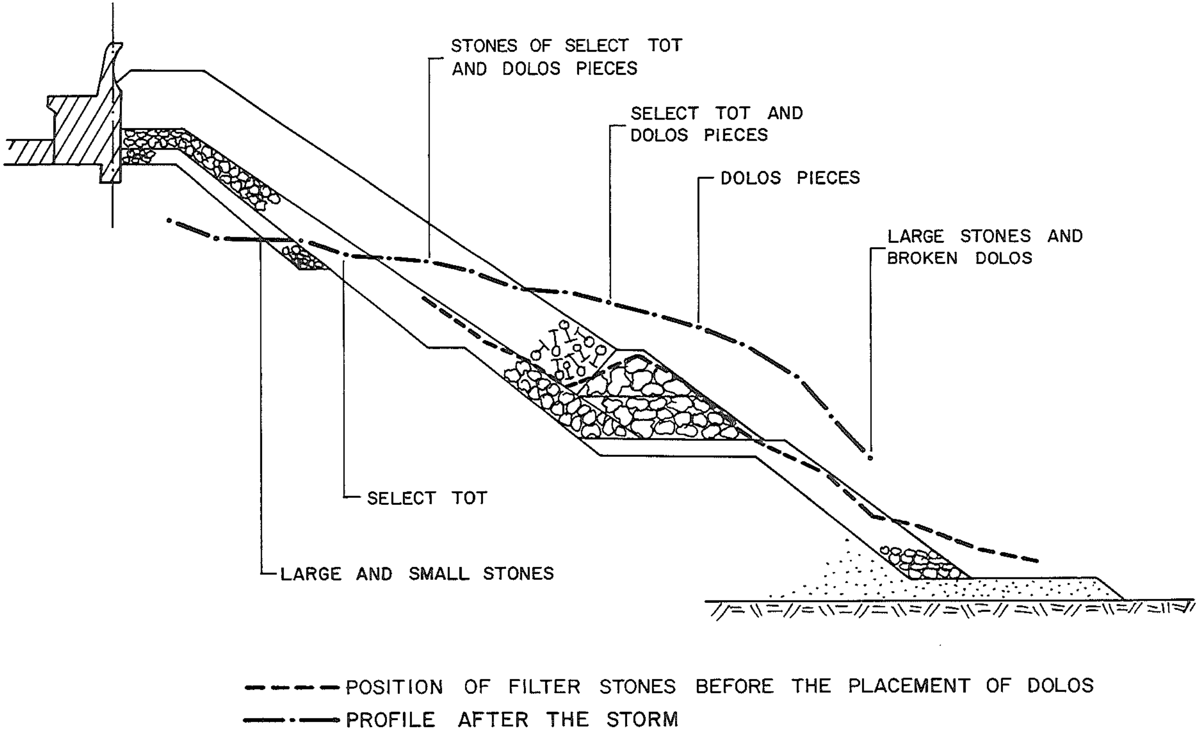

FIGURE 7.1 A TYPICAL CROSS-SECTION AS OBSERVED BY DIVERS

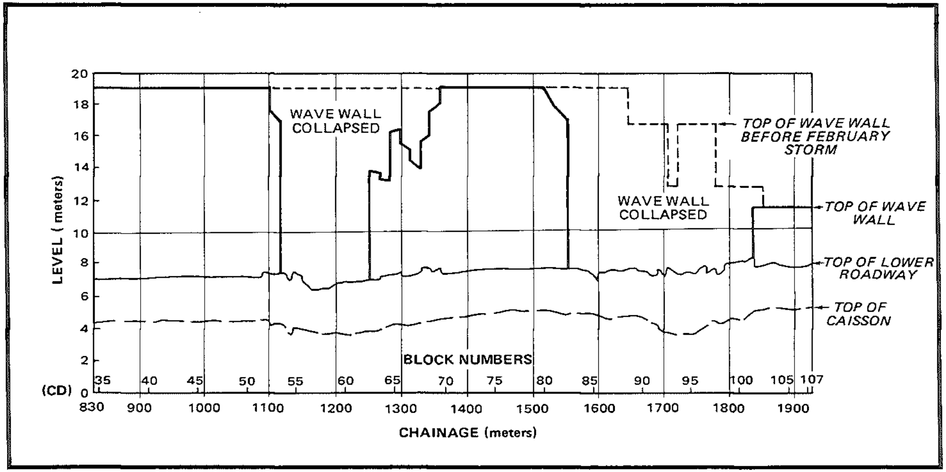

FIGURE 7.2 LONGITUDINAL PROFILES OF THE SUPERSTRUCTURE AFTER THE STORM (GAS)

face of the breakwater near caisson number 81 (these numbers are located in Figure 5.2) to a depth of -30 m where the bed rock is at a depth of -45 m. From there they swam along at that depth to a point near caisson 83 and ascended over the face of the breakwater. On their dive no whole or intact dolos were observed. Moreover, very little other material was lying with the fragments of dolos except at the maximum depths reached. Essentially the inspection by the panel members corroborated the data reports referenced in Appendix I by the Navy divers that virtually all of the underwater units were broken, irrespective of the state of the dolos above the water level. It also revealed recent movements of the broken dolos parts.

Dolos Movement at the Head of the Breakwater

Several dolos around the head were marked before the storm to measure settlement of the structure. Figure 7.3 gives the location of these individual armor units. Horizontal and vertical control was run on these armor units on 6 January 1978. After the storm, the units were located horizontally and vertically once again on 10 March 1978. However, only units d.1 to d.7 could be found. A summary of the movement of the units is given in Table 7.1. The largest observed movements were 0.46 m horizontally in one unit and 0.4 m vertically of another unit. Of course, the movements of d.8 - d.11 were significantly more. (Ordinate X is north and abscissa Y is east.)

TABLE 7.1

NET MOVEMENT OF INDIVIDUAL DOLOS AT THE HEAD
AS DETERMINED BY PRE- AND POST-STORM SURVEYS (METERS)

Dolos	X	Y	Vertical
d.1	-0.12	0.09	0.20
d.2	-0.21	-0.03	0.16
d.3	-0.19	-0.01	0.14
d.4	-0.13	-0.10	0.19
d.5	0.20	-0.42	0.04
d.6	-0.21	-0.05	0.16
d.7	0.01	-0.11	0.40
d.8	-	-	-
d.9	-	-	-
d.10	-	-	-
d.11	-	-	-

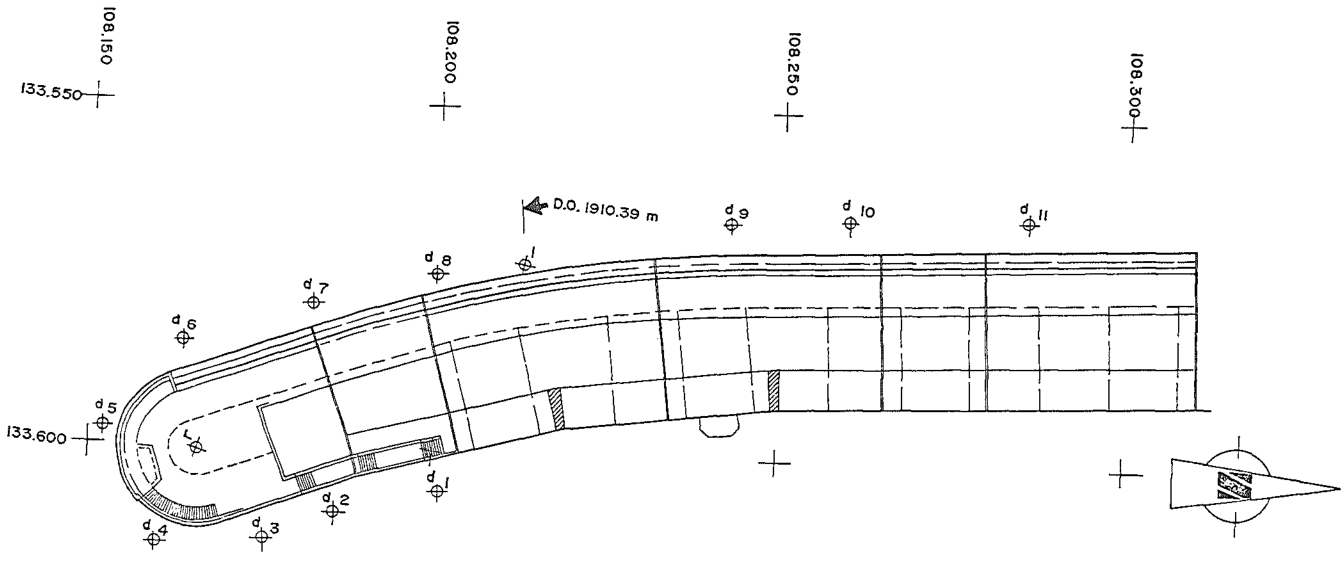

FIGURE 7.3 LOCATION OF INDIVIDUAL DOLOS AT HEAD FOR WHICH PRE AND POST STORM POSITIONS WERE OBTAINED

Photographic Record

The most impressive documentation of the results of the storm is that which was recorded photographically. Selected photographs of the damage on the superstructure and on the ocean face of the breakwater are included in Appendix H.

Visual Observations of Armor Layer

In many sections of the breakwater the dolos armor above the water level was completely removed, leaving the primary filter layer, and in some cases the core material, exposed. Only near berth number 3 and shoreward did the armor layer remain intact. Near the head, as indicated previously, some settlement and a few broken units were observed. Between the head and berth number 3 the heavily damaged sections were distributed as shown in Figure 7.4. It must again be emphasized that although the armor units were intact above the water level in some locations, the armor units were by-and-large damaged below the water level.

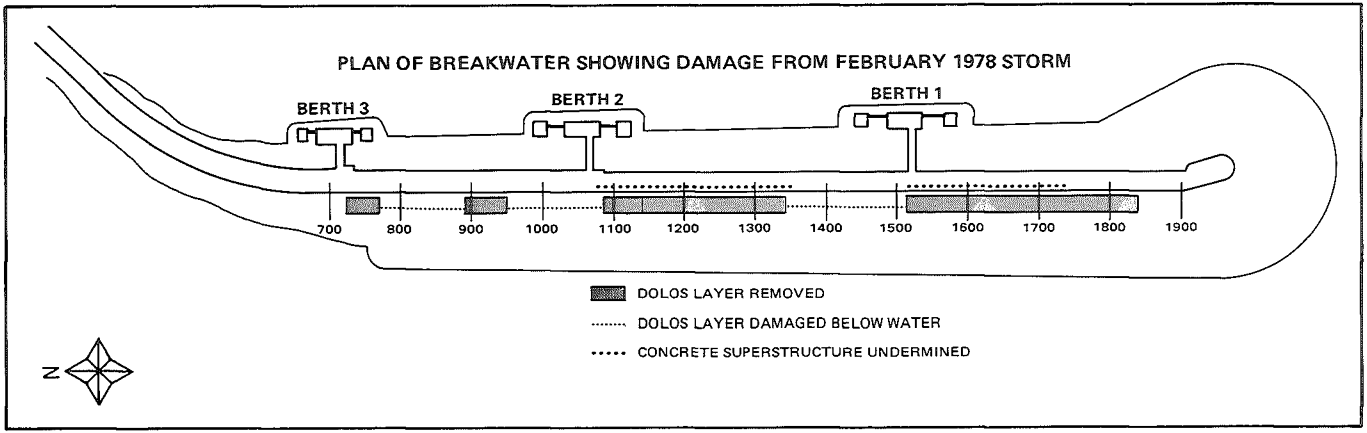

FIGURE 7.4 PLAN OF BREAKWATER SHOWING DAMAGE LOCATIONS FROM FEBRUARY 1978 STORM

CHAPTER 8

Studies Subsequent to Failure

Studies have been made by several hydraulic laboratories since the February storm of 1978. These studies have been directed towards the hydraulic design and performance of the main breakwater. Studies have been performed by the National Research Council of Canada with support of the Canadian Department of Public Works to evaluate the performance of the dolos armor layer under the storm conditions that occurred at Sines. Likewise, the Laboratorio Nacional de Engenharia Civil, LNEC, the civil engineering laboratory of the Portuguese government, has made several tests to simulate the performance of the dolos under the storm conditions and design conditions. In addition, the Laboratoire Central d'Hydraulique de France has made studies on the effects of refraction and shoaling of long period waves as well as the stability of dolos to very large waves. In this chapter these studies will be discussed to the extent that results are currently available.

LNEC Tests

As a part of the initial design procedures the LNEC in Lisbon performed a series of hydraulic model tests of the Sines breakwater, at a scale of 1:62, related to the initial and final design of the structure. Some of these tests are described in Mettam (1976), Morais (1974) and Castanho, Carvalho, and Vera-Cruz (1972). After the failure, LNEC did several model tests to simulate the damage observed on the breakwater. The first series of tests was based on the hypothesis that broken dolos in the armor were responsible for the failure. The tests began with a horizontal band of dolos in the cross-section of which 10% were broken. The cross-section was then modified to 20% broken units and finally to 50% broken units. Only when 50% of the armor units were already broken did the breakwater fail under the assumed storm conditions of February 26. A second series of tests was made to simulate individual dolos units breaking when they were moved by the storm waves; whenever a dolos unit moved, it was replaced with the two halves of a broken dolos. The results of these tests showed failure occurring in a manner similar to that observed in the field for waves somewhat larger than those estimated for Sines. Some tests were also conducted using a random wave input. LNEC has also conducted model tests for determining a new design profile for reconstruction. [The reader is referred to the LNEC reports referenced in Appendix I for further details on these tests.]

Tests at National Research Council of Canada

The National Research Council in cooperation with the Canadian Department of Public Works has conducted two sets of hydraulic model tests of the Sines breakwater. These model tests were designed primarily to simulate the damage to the breakwater during the storm of February 1978. A detailed description of these tests are given by Mansard and Ploeg (1978).

A model of the Sines breakwater was built to a geometric scale of 1:52 in the wave flume of the NRC. The dolos units were of slightly low specific weight (2.46) relative to those at Sines (2.55) but of correct relative dimensions. The dolos units were made of a mixture of sand and sulphur silicon capping compound. The Danish Hydraulic Laboratory suggests that the difference in the coefficient of friction between these "plastic" models and those made of mortar may cause some difference in the results. All stone sizes and the concrete cap were modeled according to the final design specifications. The breakwater was built in the dry and particular attention was given to the placement of dolos units. The first series of tests was, therefore, designed to investigate the performance of this structure with all model construction done according to prototype specifications.

For the first series of tests the storm in February was simulated for conditions when the significant wave height had reached above 6.0 meters. A shakedown sequence of waves was run before any actual storm conditions. Irregular waves (JONSWAP spectra) were used with some wave grouping. The JONSWAP spectra were used to produce a desired significant wave height and peak period; Mansard and Ploeg (1978) do not give the specific values for the parameters δ, σ, σ_2, and f_m used. The beginning of oscillation was noted with a significant wave height of 4.5 m. At 6.0 m a number of units were rocking just below the water level. Occasionally a unit was displaced to another location further down the slope.

During the simulation of the storm of February 26 the beginning of oscillation in the model occurred at 5.5 m (probably as a result of the settling-in process). However, at a wave height of around 7.9 m many units were rocking just below the water level and with the larger waves most units below the water level rocked a little. With 8.5-9.5 m waves (estimated peak of storm) most units below the water level moved substantially.

The units immediately below the mean water level were moved to another location further down the slope. The significant wave height was then increased in stages to 12.5 m. At this wave height dolos units were continuously removed from the armor layer. Essentially, the results showed the beginning of oscillation and displacement of dolos from the armor layer at wave heights substantially lower than those reported by LNEC and referred to by Mettam (1976).

The oscillations (or rocking) that occurred were sudden movements where the dolos unit was rapidly accelerated to the velocity of the water mass uprushing on the breakwater and then decèlerated as it impacted the adjacent dolos units. It was concluded from these tests that, during the February 26 storm at least 30% of the units in the area below the water level were "rocking" to such an extent that breakage was possible. If breakage occurred then it is possible that adjacent units would be free to move and up to 50% of the units would have broken. As previously noted, LNEC in their tests found that if 50% of the units are broken failure occurs with a profile similar to that obtained after the storm. They also found that replacement of rocking dolos by two half pieces resulted in prototype failure before 50% of the units were replaced.

The second series of tests was designed to simulate the tensile strength of concrete. The dolos were modified by means of a weak plane through the trunk. The dolos units were cut through the trunk and a slice of material was inserted with a tensile strength which properly simulated at model scale the tensile strength of the concrete in the prototype dolos (Mansard and Ploeg, 1978). The breakwater cross-section was then rebuilt with one complete part (top to bottom and one-third the width) made up of modified dolos units.

The test program was similar to the first set. The storms of February 23 and then of February 26 were simulated. In the tests, great care was used to avoid generating a significant wave height greater than that inferred for Sines (i.e. not greater than 5.9 m for the February 23 storm and 8.5 m for the February 26 storm).

The results of the second series of tests duplicated the nature of the failure very closely. The failed cross-section was almost identical to those obtained by survey after the storm. The distribution of broken dolos, filter stones, and core material agreed closely with the reports of the Portuguese divers and the findings of the Panel members who also dived on the site. Undermining of the cap also occurred as was observed at Sines.

It was concluded from the model tests that the failure of the model breakwater was primarily a consequence of the failure of individual armor units. Thus, through the artificial creation of tensile properties like those found in unreinforced concrete, individual armor units as a consequence of oscillation under wave action, were seen to break in the tests, leading ultimately to the failure of the entire armor layer. (Mansard and Ploeg, 1978)

Laboratoire Central d'Hydraulique de France

In 1973 this Laboratory undertook model studies of a breakwater cross-section for Sines that included 30 t dolosse. These studies were completed for the Societa Italiana per Condotte d'Aqua to assist with the preparation of the successful tender submission.

Since the first damage to the breakwater in February 1978, the staff of this Laboratory has studied for the contractor a number of aspects of the damage as well as developing some concepts for remedial works. These studies are reported in (Langlais and Orgeron, 1978, reports 1-5).

Analysis of Storm of February 26, 1978. This study described a detailed analysis of available meteorological data, ship observations, and recorded wave data. It was concluded that offshore from Sines the significant wave height reached a maximum value of between 8.5 and 9.5 m, which occurred from the W to WNW between 0900 and 1200 hours on February 26, 1978. The average wave period was between 12 to 14 s and it was noted that some waves had a period in the range of 20 to 22 s (Langlais and Orgeron, 1978, No. 1). The contractor had movies taken which showed very long wave periods.

Mathematical Model of Wave Refraction. This study found that refraction effects caused by the offshore topography could increase the wave height during the February storm by 10 to 40%. It was concluded that the significant wave height of monochromatic waves incident on the breakwater was of the order of 13 m with an associated maximum wave height exceeding 20 m (Langlais and Orgeron, 1978, No. 2).

Model Studies of the Breakwater Cross-Section as Built. The model tests were undertaken at a scale of 1 to 67. A wave generator produced identical groups of 10 irregular waves by amplitude modulation. The conclusions noted that maximum wave heights of the order of 13 to 15 m would, in most cases, destroy the armor layer. The incident wave period did not significantly affect these results. When broken dolos were placed in the armor layer no significant increase in the failure process was observed (LCHF, 1978, No. 3).

Model Studies of Dolos Movements. The objective of the study was to analyze the behavior of the dolos during wave uprush and downrush. Three particular hydrodynamic analyses were made: speed of fall of dolos in calm water, velocity of water within the armor layer during wave uprush, and velocity of oscillation of the dolos. A result of the tests showed that for the conditions of the 26 February 1978 storm the wave uprush could have caused vertical forces sufficient to lift a dolos from the armor layer (Langlais and Orgeron, 1978, No.4).

Model Study of Wave Refraction. This model study with regular long crested waves supported the conclusions of the mathematical analysis by demonstrating increases in wave heights of 30 to 40% and maximum recorded increase of 70% (Langlais and Orgeron, 1978, No. 4).

The results of the above noted studies were extensively reviewed by Messrs. Larras, Couprie, and Dubois (undated report). In their report, they discuss the causes of damage to the Sines breakwater during the period February 23-28, 1978. This report proposed that failure consisted of a mass displacement of the armor layer caused by large waves. The cause was, therefore, individual large waves with heights of the order of 15 m and with periods of 18 to 20 seconds. These waves were a result of a severe storm and significant wave refraction effects.

It is again emphasized that both the mathematical and hydraulic models used by Laboratoire Central d' Hydraulique de France employed deterministic waves and not a random wave field. This may have had an effect on their results.

CHAPTER 9

Considerations in Evaluation of Failure

In a project this complex, there are many possibilities for irregularities and uncertainties in design and construction. What follows is a list of what might be termed potential contributory factors mentioned to or observed by the panel. Conclusions of some other investigating groups which have been published are included. Some appear to be difficulties inherent in any large construction project; others are more specifically related to the Sines Project. They are all a legitimate part of the Sines data and as such should be recorded here and taken into account when additional study of the Sines failure is in order and when future projects of this sort are considered. It is again stressed that the material in the report is not to be construed as identifying the cause(s) of failure and the person(s) responsible; rather, this report is intended only to serve as a summary of the facts concerning Sines and to provide ideas for consideration in the design of future projects of such a grand scale. The areas of concern are briefly summarized below by categories. Lastly, a possible failure scenario is proposed based on the observations and studies of the Panel.

Owner-Designer Contractor Relationships

The designer felt that the primary area of concern was that he had no control over the execution of his plans during construction. In many cases (European in particular) the designer is retained for inspection purposes or is the technical agent dealing with the contractor for the client. In the case of Sines, the Gabinete da Area de Sines (GAS) retained all inspection and technical supervision of the project. It is also important to note, although a minor consideration, that all communication was in Portuguese. Moreover, throughout the project all communication between the firms and agencies involved had to pass through GAS; this slowed the communication process and restricted discussion. Underlining this problem of owner-designer-contractor relationships is the request made by Bertlin and Partners to see copies of the results of the irregular wave tests on the head of the breakwater as late as December 1976. (See Figure 5.1 for status of the breakwater at that time.) Bertlin noted in their review to a draft of this report that "...the lack of direct communication between the designers and the LNEC laboratories led to a number of communication problems. Two of these were particularly serious:

(a) In spite of frequent requests from the designers, the reports on the tests were not made available. They were merely told whether or not the proposed cross section had withstood design conditions satisfactorily.

(b) The designers requested GAS to arrange for testing model breakwater sections to total failure as it was known that, unlike rock slopes, dolos slopes lose their stability suddenly. GAS did not respond and these tests were not carried out." (Personal communication from Bertlin and Partners, 1980.)

Selection of Design Waves

Wave data available for the initial design of the Sines Breakwater were sparse. Even by the end of the design process only a minimal amount of wave data were available. Thus it was difficult to estimate the design wave conditions with a high level of statistical reliability. The 100-year storm design wave belongs to a different statistical population than the non-storm design wave commonly measured at Sines or at Figueria da Foz. Moreover, it is commonly accepted and the opinion of the Panel that extrapolation of extreme value events should not extend more than two or three times the total length of record. Nevertheless, demands do dictate at times that whatever data is available must be used. Under these conditions, the designer's recourse is to perform a detailed statistical error or confidence analysis of the data. The effects of refraction and wave grouping were not considered in the selection of the design wave.

Three particular items that apparently escaped consideration in the design wave selection were: a) climatic analysis, an overall review that would show protection of the Portuguese coast by the Azores high and show the dependency of the results of a climatic analysis on the phenomenon; b) statistical error analysis or identification of the accuracy of the design wave; and c) shallow water effects on the available recorded wave data upon which the design was based and on the waves approaching the Sines area.

It is important to note that the selection of the offshore design wave conditions did not influence the failure of the structure; the conditions that existed offshore were less severe than those specified in the design.

Storm of February 26, 1978

As indicated previously in this report, it is extremely difficult to identify the size of the waves that occurred on February 26 and those that preceded that day. The data presented herein, based on that measured at Cabo da Roca, indicate waves with a significant height of 8 to 10 m. Refraction analyses by the Laboratoire Central d'Hydraulique de France indicate larger waves with significant wave heights up to 13 m may have occurred at the structure. Because the wave recorder was not working, an indisputable definition of the wave field will never be available. (A comprehensive wave hindcasting model was used by Dr. Don Resio to provide an independent estimate of the storm characteristics. The results of these tests also give significant wave heights at about 10 m [personal communication with Dr. Don Resio, 15 May 1981].)

Breakwater Design

Although the breakwater was designed according to accepted practices (Shore Protection Manual, coastal engineering conference proceedings, etc.), the design still may not have been sufficient to resist the force exerted by the February storm; this is not a criticism of the design but of the available references for design. The design for this very deep site which is exposed to deep ocean waves was predicated on an extension of the procedures which were well-understood for small breakwaters in shallow water conditions. Another aspect of the design that causes some concern is the complexity of the breakwater cross-section; the section would be difficult to inspect, as well as to construct, in these extreme water depths albeit a similar cross-section is found in many design references.

The seaward slope of the breakwater was quite steep 1:1.5. This has been a cause of concern among those who have reviewed the design. Although the model studies showed the structure to be stable with this slope, it is so steep as to allow no tolerance for error. The head which was placed at 1:2 slope was much less damaged during the storm than the trunk.

Structural Strength of Dolos

It is unfortunate that little information has been published about the stress distributions that occur in dolos under dynamic loadings. An approach to the static problem is presented by Desai (1976).

The static stress in a dolos unit due to its own weight will be proportional to the length of the unit. The larger the unit, the greater will be the self-induced stress. There is a limit where a unit is so large that it cannot support its own weight. There is also a point where a unit located on the bottom layer of a breakwater armor section cannot support its own weight plus those units resting above.

Although it is possible to analyse, model and test the strength of a dolos unit, it is not possible to determine the loads to which a unit will be subjected. The dynamic loads applied to dolos depend upon their individual position and range of possible movement as well as the characteristics of the waves acting on them. The need for secure placement and interlock of dolos is very important. Individual units are not necessarily stable because of their own weight and therefore, they must rely on mechanical and frictional interlock with surrounding and underlying armor units. In an effort to avoid any violent movement or displacement of the dolos the design firm stated in their review of a draft of this report that (personal communication from Bertlin and Partners, 1980):

(a) "they included stringent oscillation criteria in their model testing specification

(b) they specified and recommended dolos placing and inspection procedures which were intended to ensure careful and secure positioning of the dolosse."

It was also understood that increase in the size of the unit and of the incident wave height could result in increased stresses in the units although, as already pointed out, it is not possible to carry out a precise analysis. The design technique adopted was "to carry out static and dynamic analyses of a wide range of hypothetical loading cases and compare the maximum stress levels in the proposed Sines dolos with those which had been used at East London, South Africa which were the largest dolos in service at the time." (It is not clear what type of static and dynamic analyses was to be used.)

Model Studies

The official model studies which formed a part of the design were done by Laboratorio Nacional de Engenharia Civil (LNEC) for GAS and not the designer, and the communication between the designer and the modeler was always via GAS. The results of the irregular wave tests were not available to the designer before the design was complete. The model tests did not account for the effect of wave grouping that occurs in the wave data recorded at Sines. Refraction also was not analysed or considered in the model studies. The model tests were not designed to simulate structural properties of the dolos armor units; this is not meant as a reflection on the laboratory, but rather the state-of-the-art in model testing.

With regard to the model tests Zwamborn (1979) says that:

> "it appears that no tests were done on the final design...using irregular waves of sufficient duration (say 6 to 12 hours prototype) with a significant wave height equal to or exceeding the design wave height of 11 m."

Furthermore Zwamborn cites studies done by the LCHP after the failure:

> "recent tests carried out by the LCHP showed that for wave periods between 16 to 18 s and wave heights in excess of 11 m, the vertical upward water velocity near the still water line could be the same order of magnitude as the terminal settling velocity of 42-t dolosse in still water. As a result, groups of dolosse could be lifted from the slope by these long and high waves, a phenomenon frequently observed in model tests."

Wave and Static Forces

An important contribution to the analysis of the internal stresses of dolos under both wave and static load conditions was given by Dean Morrough P. O'Brien in his review of an early draft of this report. His comments follow:

> "Hydraulic models have been employed to determine the stability of breakwater armor units exposed to ocean waves. These model results have been employed in the design of full-scale structures by applying the same linear scale ratio both to the wave dimensions and to the overall configuration of the breakwater as well as to the individual armor units. There is a basic relationship regarding the application of the results of model tests to fullscale structures which appears not to have been recognized or, at least, not to have been explicitly stated in the literature, namely, that the unit stresses, due to both static and dynamic loads increase in direct proportion to the absolute size, throughout geometrically similar breakwater units, arranged in geometrically similar configuration in the structure.
>
> "Although the forces applied by waves have usually not been measured in model tests, whatever the forces and pressures may have been in the model at the limit of stability, they will be increased in the prototype under dynamically similar conditions in accordance with the Froude model laws. Pressure (unit hydraulic stress) at corresponding locations and times is proportional to V^2 which in turn is proportional to the linear scale.

"The unit stresses at corresponding locations due to the static load of the nesting units plus the dynamic force applied by waves, corresponding to the stability conditions found in the model tests, will increase in proportion to the linear scale. The hydraulic model tests give no measure of magnitude of these stresses which may exceed the ultimate strength of the material in the prototype.

"The dolos is relatively slender as compared with natural rock or with the early cast-concrete armor. The weight of supported units plus the thrust of interlocking units may result in static stresses at some points which are near the limiting value, thus leaving small margin for wave forces. Whether or not failure will occur depends on the strength of the concrete, but it is also true that, no matter what material is used, there is a critical size at which failure would occur under static loads alone."

Dolos Placement

One item of concern is the ability of the floating cranes to place the dolos satisfactorily above and below the water in characteristically rough conditions. The design firm had recommended that land-based equipment be used. The dolos were not placed by location but by "density" (a predetermined number of dolos per m^3). (Instead of a coordinate for a particular dolos, a certain number were to be placed in a given area.) Underwater inspection of dolos placement was infrequent because of the heavy seas during most of the year. Neither the GAS nor the contractor maintained underwater inspection programs. Zwamborn (1979) reports that between 3.5 and 16.8% of the dolos in a sampled section were broken by August 1977 as a result of placement, settlement of the structure, and/or previous storms. Arrangement of the dolos was generally random (not with 60 percent having the vertical leg seaward as specified in the design, although the significance of this criterion has been discounted).

Dolos Reinforcement

The dolos were not reinforced. Arguments for this design decision are given in Appendix E. The issue of reinforcement of armor units is further discussed in Appendix J and Desai (1976). Subsequent to the February 1978 storm, some reinforced units were used as a test for the outer layer of dolos, to prepare the breakwater for the coming winter; although the performance of the reinforced and unreinforced units was not the same, both units sustained extreme damage in the 1978-1979 winter. This application of reinforced armor units should not be considered a true test of the response of the original structure to storm waves, had it been built with reinforced armor units.

Permeability of the Core

The permeability of the quarry run material (TOT) in the core may have been significantly less in some sectors than that used in the model tests and specified in the contract. This difference may have resulted in added wave run-up on the face of the breakwater; and this, in turn, may have had an adverse effect on the stability of the armor layer. It is important to recognize that the stability of the armor layer might be jeopardized by the increased wave run-up that would result from a lack of permeability of the core. The volume of both wave uprush and the return flow down the face of the breakwater would impose a greater force on the individual breakwater armor units and could cause their dislodgment from the protective layer.

Other Possible Causes of Failure

Additional possible causes of failure include those concerns expressed by the designer at the end of Chapter 5 and those presented below:

- deficiencies in the underlayer
- excessive fines in the filter or core material
- excessive pore pressure in the core caused by wave action
- failure of the 16 to 20 t toe stone
- wave reflection from wave wall

Zwamborn (1978) has presented an evaluation of possible causes of damage to the breakwater which are summarized in Appendix M. Perhaps a combination of some or all of the factors described in this Chapter could be responsible for the failure of the breakwater. It is practically impossible to assess properly these explanations because of the imprecise information describing the storm, as-built conditions and after-failure conditions. A possible scenario of the failure process is given. This scenario is presented by the Panel with the knowlege that there is insufficient data to propose it as the actual events of February 1978.

Published Statements of Cause

Several investigations have been conducted, at various levels of effort, to understand and identify the causes of failure. One of the first assessments was made by a team of Dutch engineers for the Delft Hydraulics Laboratory. One of their primary conclusions was that the removal of the 16 to 20-t toe stone by the larger waves was a likely cause of failure. Subsequent to the removal of these stones the dolos layer shifted and was damaged.

Zwamborn (1978) concluded from his research at the National Research Institute for Oceanography, South Africa, in a report to Condotte d'Acqua, "that the main causes of the major damage to Sines main breakwater during the February 1978 storm are as follows:

(i) The particularly damaging effect of the large waves in the spectrum which, because of the great water depth in front of the breakwater, could reach the structure without being reduced due to breaking. The effect of these large waves cannot be described by the significant wave height alone; model tests at an appropriate scale reproducing realistic prototype waves in deep water are essential. With a probable incident wave height (H_s) during the peak of the storm of 9 to 10 m, maximum incident wave heights of 14 to 17 m must have occurred.

(ii) Local incident wave heights in excess of the design wave heights due to refraction of the longer period waves (the period of the waves, at the height of the storm, containing most of the energy was 18 to 20s). Both the significant and maximum wave heights are affected and a reasonable correlation was found between the high wave areas and the failure areas. Average increases in wave height of up to 20 per cent (with maximum values of up to about 50 per cent) were found, thus increasing the probable local average significant wave height to 11 to 12 m and the corresponding maximum wave height to 17 to 20 m.

(iii) Dolosse breakages during the storm due to excessive movements caused by the very long and large waves discussed under times (i) and (ii) above, which resulted in vertical water velocitites in the armour of the same order as the terminal settling velocity of single and also groups of Dolosse." (Zwamborn, 1978)

Comments from the design team on the cause of failure have been included at the end of Chapter 5. In these comments the problem is focused on project management, supervision, and construction. The comments were summarized by Mr. Peter Mornement of Bertlin and Partners, in an interview with the New Civil Engineer, as "We are sure inaccurate placing of the dolos and the supporting toe were the main causes of failure." (Anon, 1979)

The contractor responded to this allegation as "contrary to what the Consultants state, the photographic documents demonstrate that the dolosse are well placed and that they have the necessary degree of interlocking." (See Appendix B)

Official Portuguese Commission Report

The Official Portuguese Investigation Team filed their report with the Government in April, 1979. The report of the official investigation is given in Appendix C. Some of their conclusions are as follows:

- Structural fragility of the dolos was the primary cause of failure.
- There were serious shortcomings in the design wave selection.
- The design of the breakwater was "theoretical" and difficult to build.
- Consideration was not given to refraction of wave energy.
- The LNEC was not exhaustive enough in its testing program.
- The reliability of the dolos should have been questioned.
- Gabinete da Area de Sines did not have the capability to plan and execute a marine project of such magnitude.
- There were shortcomings in the management and supervision of the project by GAS.
- LNEC should have had a more active role in the design phase.

Failure Scenario

The causes of failure and the sequence of events leading to the failed armor layer during the storm of February 26, 1978 will never be completely defined. This is because of the paucity of information concerning the exact state of the structure prior to the storm, the wave conditions at the breakwater during the storm and the condition of the breakwater during the storm. Many explanations of the cause of failure have been proposed, and the state of the breakwater following the storm was such that any number of events might have caused or contributed to the failure.

The Panel presents here a description of a probable pattern of events leading to the destruction of the armor layer. The critical observations leading to the failure scenario are those on the extensive breakage of the dolos, the profile of the damaged breakwater, (Figure 8.1) the model tests with irregular waves which showed rocking of dolos, the model tests with the artificially weakened dolos, and personal experience with other breakwaters. It is the authors' opinion that this description is plausible in that it fits with the available evidence albeit other failure scenarios can be proposed from the available data.

In the early stages of the storm, when the significant wave height reached 6 m, some dolos units began to move in the vicinity of the mean water level. These units were the ones which had been placed in a relatively unstable position and had little support from the adjacent units. (It is practically impossible to place dolos so that every unit is in a stable position.) The initial movements occurred when a larger wave ran up through the armor layer.

As the wave height and period increased, the movement became more severe and the unit itself accelerated to the velocity of the uprushing wave. The following impact with an adjacent unit produced tensile stresses in one or both units that exceeded the tensile strength of concrete. This resulted in breakage, and frequently the pieces were carried away by the uprushing or downrushing wave. During this process, the pieces themselves collided with other units, in some cases causing additional breakage.

At the peak of the storm, when the significant wave height exceeded 8 m, a large number of units located just below the water level broke. As the pieces were carried away, adjacent units were free to move and a rapid disintegration of the armor layer occurred. Initially the greatest damage occurred immediately below the mean water level. The broken pieces were moved by the water motion and by gravity from the armor layer and to the lower part of the mound. The slope of the seaward side of the breakwater below the water level then became flatter.

During the final stages of the storm the armor layer was completely removed at some locations. The broken dolos pieces were displaced to the base of the armor layer and the underlying stone layers were exposed. Wave action then moved these exposed stones over the broken dolos pieces. Continuing wave action eroded the core material and began to undermine the superstructure.

As the concrete superstructure was undermined it tilted forward. In some instances the structure broke in the base, where the pipelines carrying petroleum from the berths to shore would be located, and at some of these locations the wave wall at the top of the structure was snapped off and thrown back as a result of wave impact.

The reasoning leading to the above scenario of the failure of the Sines Breakwater suggests the following:

(1) The dolos units rocked, broke and moved in the armor layer under the wave conditions that existed during the storm of February 26, 1978.

(2) The dolos units were of sufficient strength to withstand the forces of wave action except when they moved. In movement, they were unable to resist the stresses produced.

Support for the argument that the damage occurred as a result of breakage of dolos units is provided by model studies, prototype testing of dolos strength and observation of damage to the Sines and other breakwaters.

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APPENDIX A

Official Reply of Bertlin-Consulmar-Lusotechna to Inquiry of Portuguese Investigating Committee

May 3, 1978

(This translation of the original reply which was in Portuguese is by R.P. Yates. Only one of the appendices mentioned in the document has been included; however, a notation of the contents of each is at the end of this document.)

7210/G/2/JGB/CAB 3rd May 1978

REPLY TO THE COMMISSION OF INQUIRY ON
THE ACCIDENT TO THE WEST MOLE AT SINES
SUBMITTED BY CONSULMAR ON 14th APRIL 1978

Translated by R.P. Yates

Question 1 Differences between the project profile of Consulmar and the profile approved for construction. Who proposed them? Who approved them? What justification was given?

Are they merely a matter of adjustments or are there differences which can be considered as being relevant?

Answers

(1) The profile proposed by the BERTLIN-CONSULMAR-LUSOTECHNA Planning Consortium for international tenders, and developed to the stage of preliminary project, was destined to serve as a basis of comparison for the tenders. It was assumed that the profile would later undergo changes as a consequence of the proposals made by competing tenderers and of laboratory tests.

(2) The firm known as CONDOTTE taking part in the competition, besides their proposal based on the preliminary project for tenders, also submitted a variant proposal (Variant B1) which includes some changes in the profile of the breakwater (appendix 1).

(3) This variant having been preferred, the Planning Consortium proceeded in the development of the project taking into consideration fundamentally the tests which meanwhile were being conducted by LNEC, the results of which were not available at the time the preliminary project for tenders was being prepared (appendix 2).

(4) The changes proposed by CONDOTTE and destined to facilitate construction by the means which the Contractors claimed to

possess, and to facilitate placing of the network of pipes on the breakwater consisted mainly of increasing the width of the breakwater and of its concrete superstructure.

(5) The adaptations introduced into the project later, as a consequence of the results of the tests, consisted fundamentally of adjustments to the concrete superstructure and to the layers of dolos and rubble which can be summarised in the following main items:

(6) - SUPERSTRUCTURE: Increase in the height above water of the wave wall from +16m (ZH) to +19m (ZH); separation of the slab supporting the pipes from the small inside caissons; transference of curved facing to the outside of the wave wall; introduction of ventilation holes in the slab supporting the pipes.

(7) - DOLOS COVER: Increase of weight of dolos from 30 to 40 t. Increase in the height above water of the berm from +13m (ZH) to +16m (ZH); increase in width of the berm from 2 to 3 blocks.

(8) - OTHER LAYERS OF RUBBLE: Increase in weight of rubble from 6 -9 tonnes to 16 - 20 tonnes in the support footing of the cover of dolos, and change in its configuration from parallelogram to trapezoidal; introduction of selected TOT above the -2.50m (ZH) level and the protection filter at the base of the mole in the zones of the sandy bottom; introduction of a second layer of secondary rubble between the TOT and the layer of 3 - 6 tonnes.

(9) The changes proposed in the Variant B1 by CONDOTTE were justified for reasons of economy and construction.

(10) The adaptations introduced in the development of the project were suggested by the test results and/or by recommendation of technicians with experience in this type of works, namely, Engineer Zwamborn.

Among these adaptations the following are noteworthy:

(11) - Increase in above-water dimensions of the breakwater and the dolos, and in the curve of the facing of the breakwater with the object of reducing wave over-topping which the tests had shown to be excessive and a possible cause of damage to pipes.

(12) - Increase in weight of dolos with the object of reducing the risk of their displacement from the positions in which they were laid and of oscillations observed in the tests with dolos of 30 tonnes (Appendices 1, 3, and 4);

(13) - Separation of the piping-support slab with the object of increasing the stability of the superstructure given that the tests showed that the break-up of the breakwater was due to its slipping towards the interior of the port (see figures 12 and 13 of Appendix 1);

(14) - Increase in weight of rubble in the footing of the cover of dolos, as a result of the tests having revealed that blocks of barely 6 - 9 tonnes had been displaced;

(15) - Change in form of the support base of the dolos cover suggested by Engineer Zwamborn of South Africa, one of the inventors of this new type of block, with the object of increasing security against slipping of the cover and of facilitating the placing of the first dolos (Appendix 5).

(16) All of the changes and adaptations referred to were submitted for approval to GAS, either by CONDOTTE (Variant B1) or by the Planning Consortium (Appendix 4).

(17) In our opinion all the changes effected contributed to improving the project and increasing the security of the Works. Where some of these changes are extensive from over wave-topping certain points of view (e.g., reduction of constructional facilities, etc.) they should not be considered as relevant changes in relation to the type and general concept of the Works.

Question 2 TESTS

Tests completed. Tests which could have been made, but were not made.

What tests should be made to clarify the accident?

Which laboratories other than LNEC are in a position to carry out such tests?

Answers TESTS COMPLETED

(1) From the point of view of hydraulics, four types of tests were made as follows:

- Studies of wave action in the port;
- Studies of channel stability;
- Studies of resonance in the port;
- Studies of stresses caused in the piping by wave over-topping.

(2) During the course of these tests several visits were made to the Offices of LNEC by technicians of GAS and of BCL. These visits were followed by meetings to analyze the results meanwhile obtained and to establish the general lines of conduct of tests in future (Appendices 2 and 5). Technical notes were prepared by LNEC during the tests on the temporary results of the tests. From these notes the following reports were prepared:

(3) STUDY OF TURBULENCE IN PORT

"Study of turbulence in a reduced model of the Port of Sines - Provisional Report, Oct. '72."

"Port of Sines - Study of turbulence in a reduced model - Provisional Report (II) - Feb. '73."

"Port of Sines - Study of turbulence in a reduced model - Provisional Report (III) - June '73."

(4) STUDIES OF CHANNEL STABILITY

"West Mole of the Port of Sines - Tests on a channel of typical section (breakwater of curved face) - Jan. '73."

"Port of Sines - Tests of stability of the West Mole Head - July '76."

"Tests in section of irregular channel waves" (not received but it is assumed to be in course of preparation).

(5) STUDIES OF WAVE RESONANCE

"Port of Sines - Study of resonance in reduced model -Mar. '74."

"Port of Sines - Study of resonance in reduced model -II Report - Apr. '75."

(6) STUDIES OF STRESSES CAUSED IN PIPING BY WAVE OVER-TOPPING.

"Studies of stresses caused in piping of West Mole-Port of Sines by wave-overtopping - Feb. '77."

(7) HYDRAULIC TESTS WHICH COULD HAVE BEEN MADE

Besides the tests made others could have been made in accordance with the programmes recommended by the Planning Consortium as indicated in Appendices 5, 7 and 8. The tests not made would have been of the same type as those made, covering other characteristics of waves (heights, periods and obliqueness of wave attack) or some adaptations of section. In general, they were not carried out because of time and/or equipment limitations.

(8) TESTS FOR CLARIFICATION OF THE ACCIDENT

For clarification of the accident in order to reproduce the damage verified, it is required to carry out tests of the present section in subjection to irregular waves, with wave conditions equivalent to those of the storms of 21.2.78 and 2.3.78. In these tests eventual differences introduced between the projected section and that actually executed should be introduced successively (for example defective interlocking of the dolos, faulty placing of rubble of 16 to 20 tonnes at the base of the dolos, excessive content of fines in the TOT, presence of high numbers of displaced dolos, presence of excessively large or small blocks in the various rubble covers, thicknesses and positions of covers not adhered to in the placing of dolos and stone blocks against the breakwater wave wall, etc.

Besides LNEC carrying out such tests, equipment is available at the following laboratories at least:

Trondheim (Norway)
Delft (Holland)
Wallingford (England)
Chatou (France)
Stellenbosch (South Africa)

Question 3 Did Consulmar collaborate with the Inspection Engineers?

In the terms of the contract what was Consulmar's function in this field?

Were the cross-sections actually executed presented to Consulmar as the works progressed?

Is it admitted that, in accordance with the construction methods adopted, appreciable differences between the projected section and the executed section could have been revealed?

Does there exist sufficiently efficient and rapid processes for checking the submerged works which were actually carried out?

Answers

(1) The firms of the Planning Consortium did not collaborate with the Inspection Engineers on the works site.

(2) Some of the visits made sporadically to the site by some of the Consortium technicians did enable them to observe how the mole was to be constructed. Comments on the construction procedures and the control techniques used are contained in the letters of Appendices 9, 10, 11, 12 and 13. The letters were always accompanied by verbal commentaries, but the commentaries were not always accompanied by letters.

(3) In terms of the contract it fell to the Planning Consortium to present all the clarifications necessary for the correct interpretation of the project.

(4) The Planning Consortium offered to contract separately to undertake the inspection functions, which offer was not accepted. Later, systematic technical assistance services were proposed more than once to the Inspection Office of GAS which had been set up meantime (Appendices 14 and 15).

(5) In this respect, about the middle of 1977 limited technical assistance was proposed and accepted by Consulmar independently of the Planning Consortium - B.C.L. which was intended particularly to assist the Inspection Office in the evaluation of

quantities to be paid and entailing barely two visits per month to the Works by two of Consulmar's technicians. This assistance was extended at the end of 1977 to include the detachment of two technicians on a permanent basis to Sines site to participate in the preparation of measurements charts and the criteria for the application of unit prices, in view of the need to prepare the documents for the approaching provisional acceptance formalities.

(6) Considering that the Consortium was not assisting in the Inspection function, the profiles of the works actually executed which the Inspection Department should have recorded, were never presented to the Consortium as the works progressed.

(7) The available information on the construction methods adopted, and particularly the control procedures used by the Contractor or by the Inspection Department, leads us to suspect that in certain locations differences between the projected profile and the profile executed might have been established, both in the configuration (positions and thicknesses of different layers) and in the characteristics and distribution of the material used (interlocking of dolos, dimensioning of the elements in each layer, percentage of fines in the TOT, etc.)

(8) The underwater processes available should not be considered as efficient and fast. Because of the inherent difficulties of underwater inspection, the truth is that at the present time an appreciable experience in the inspection of such works is already available, such experience being derived from the execution and inspection of delicate underwater operations such as pipeline and off-shore constructions for oil exploration and exploitation on the continental shelves. Many firms of various countries devote all their activities exclusively to the execution and inspection of underwater operations, for which they deploy very diverse and complex equipment.

Question 4 Experience on the performance of the dolos in works. Performance of 40 tonne non-reinforced dolos. Influence of placing by floating equipment in conditions of considerable turbulence. Recommended slopes. Does the support footing at the base of the dolos cover appear to be well dimensioned?

Answers

(1) Experience of the behaviour of the dolos in works is at the present time already reasonable (sic)/appreciable. Attached is a drawing indicating the locations and main characteristics of the works executed or projected with this type of block (Appendix 16).

(2) The dolo is indisputably effective in the field of hydraulic (marine) functions. In fact, considering its high potential for interlocking in works subject to high project wave values, it offers minimisation of weight as compared with other types of blocks.

(3) During the development of the project, the use of reinforcement of the dolos was considered, the conclusion arrived at being that reinforcement was not necessary, as was made explicit in the paper made by J.D. Mettam for the 15th Conference of Coastal Engineering in 1976 (Appendix 1). The Planning Consortium, besides several contacts made with specialists in this technology, including visits to South Africa, also took into account the paper by Lillevang entitled "Experimental Studies of Stresses Within Breakwater Armour Piece Dolos" given at the same Conference.

(4) In the case of Sines, the dolos were over-dimensioned (42 t instead of 33 t theoretically) in order to retard the beginning of movement, and their shaped form made more robust (introduction of chamfers in the leg-to-body joints and the thickening of the body) to reduce the risk of fracture. This preoccupation led to the inclusion in the bills of quantities of the contract dolos for testing and later the attention of GAS was drawn to this matter (Appendices 17 and 18).

(5) The use of floating equipment in conditions of considerable wave action, such as is suspected to have occurred at Sines during some periods of the construction of the mole, may have entailed difficulty in placing and interlocking the dolos as well as causing breakage and fractures in them. The larger the dolos, the greater the precautions that should be taken in placing them. However, it is not known what were the criteria used for the establishment of limits of marine conditions for placing dolos. During the evaluation of various proposals submitted in the Competition of 1973, the attention of GAS was drawn to the problems inherent in the use of floating equipment in placing dolos (Appendices 19 and 20).

(6) In the Tender Specifications (page 319) it was stipulated that the Contractor should demonstrate to the Inspection Department "on site by means of models that the process proposed for placing the blocks at underwater levels ensured adequate interlocking of the protection cover."

(7) The slopes and the section of the backing of the dolos cover were dimensioned in accordance with the results of tests with irregular waves which took place at LNEC, and which within the inherent limitations of simulation were considered sufficiently representative of natural conditions. The slopes adopted, as well as the classes and thicknesses of rubble are in accordance with experience gained in other works and with constant reference to the literature of the technology available at the project development level. It is not believed that they presented difficulties of execution beyond those inherent to this type of works. Despite the exposure given to the profile adopted not only for the purpose of the tender competition as in the brochure distributed by GAS and already published articles, no criticism of the general dimensioning of the profile was ever heard. Engineer Zwamborn, Director of the Laboratory where dolos were first studied, classified the profile as "adequate for the design conditions" (Appendix 5, page 4).

Question 5 Experience of the behaviour/performance of rubble foundations of great depth.

Magnitude and evolution during settling period.

Importance of the fact that some portions of the foundations were not left to pass a winter before the construction of the superstructure.

Is there any correlation between the damaged portions and those on which the superstructure was built before the passing of one winter?

Answers

(1) Particularly in the construction of dams, there have been executed in many parts of the world embankments of rubble of great height. The settlement of such embankments depends on the nature of the constituent elements, on granulometry, on the existence, or non-existence of sharp serrations in the elements of the embankment, of the height of the layers in which the foundations are executed, and the interval of time between the tipping of the various layers. Rubble Works using blocks of stone with very sharp serrations and of almost uniform size such as used generally for supporting blocks and concrete caissons in coastal works, sometimes causes considerable settlement.

Foundations consisting of rounded elements and those of a properly selected granulometry present in general, as a whole, settle barely 0.5 to 1.5%.

(2) This was taken into consideration in the particular case of Sines. The action of the sea during the period of 3 months stipulated in the specifications as a minimum between the tipping of the rubble and the execution of the concrete superstructure, as also the size limits of the fines destined to fill the spaces between stones of greater dimensions, contributed to limiting the settling to small values. The inclusion in the tender specifications of measures to reduce settlement was

intended mainly to avoid cracking of the superstructure in the event of differential settling.

(3) The Consortium does not have systematic information on the dates of placing the rubble nor does it have any knowledge of the results of observations of settlement during the placing of TOT and rubble.

(4) The Consortium does not, for those reasons, consider itself qualified to determine whether there is any correlation between the damaged portions and those on which the superstructure was constructed before the passing of one winter, nor whether with one winter passed, complete withdrawal of temporary materials and repairs to the zone (normally designated as "Winter Head") were always carried out.

Question 6 What was the height of waves allowed for in the project? And at what value was it found to be in the case of the accident? In normal conditions, is it possible to quantify the safety of a sloped breakwater? In the case of Sines what would the coefficient of safety be?

Answers

(1) The criteria adopted for the selection of the project wave are explained, for example, in the paper given by Engineer Mettam in 1976 at the Conference of Coastal Engineering in Honolulu (Appendix 1).

(2) They can be summarised as follows:

beginning of the oscillation of the blocks in storms (of a violence) with a frequence of 1 in 10 years.....8.5m

beginning of displacement and fall of some isolated blocks in storms (of a violence) with a frequency of 1 in 30 years.....9.5m

beginning of destruction (more than 1% of damage in the dolos cover in storms) (of a violence) with a frequency of 1 in 100 years.....11.0m

(3) The characteristics of the storm, including the height of the maximum significant wave, are not yet completely known, a work group having been formed by technicians of INMG, LNEC, and CONSULMAR to study them. At present it can be said that the significant wave can not have exceeded waves with a frequency of 1 in 30 years. The difficulties encountered in the characterisation of the storm must be attributed fundamentally to the non-existence of data on the conditions in the outer bay of Sines relating to 25th and 26th February.

(4) Considering the characteristics of this type of work, it is normal procedure to quantify the safety of a sloping mole on the basis at least of the adoption of a requirement with a definite probability of occurrence.

In the case of Sines, the period of frequency considered was 100 years. The criterion of damage adopted for this requirement depends on the magnitude of the Works and the difficulty of eventual repairs.

For practical reasons 1% of damage is often regarded as "absence of damage."

(5) Merrifield to whom the credit for the initial development of dolos is owed, states in an article published in "Dock and Harbour Authority" issue of April 1970 that in South Africa it is admitted that 2% of the dolos can be displaced when subjected to the project wave. Also, it is current practice to test works in reduced scale model to the point of destruction, the wave conditions in which destruction is attained, and the form of damage, being recorded (Appendix 8).

(6) In the case of Sines breakwater, the destruction tests were meantime carried out only with regular waves, and in an intermediate phase of the project it was observed that the destruction point was reached with the slipping of the superstructure toward the interior of the port when subjected to waves of 13 m.

Question 7 Is there any explanation for the accident?

In your judgement what causes could have originated the accident?

Which among the following hypotheses seems to you to have caused the accident?

1. Gross error in the project.
2. Gross error in construction.
3. Marine conditions much greater than those allowed for in the project.
4. One, or interaction of more than one, of the following hypotheses:
    - 4.1 Erosion of the foundation.
    - 4.2 Settling of the core of the base.
    - 4.3 Loss of fines from the core.
    - 4.4 Displacement of the dolos cover due to deficiency of support footing.
    - 4.5 Deficiency of dolos fixing.
    - 4.6 High percentage of displaced dolos.
    - 4.7 Excessive value of Slope-angle projected.
    - 4.8 Construction with slopes more pronounced than in the project.
    - 4.9 Specific discovery of a locally high number of displaced dolos starting off a process of destruction of the zone under protection.
5. Other hypotheses.

Answers

(1) While information on the characteristics of the storm and the true state of the Works before and after the storm is not available, in our judgement it is impossible to ascertain the cause(s) of the accident.

(2) In regard to the hypotheses presented above, and keeping in mind all that has been revealed, we believe there was no error - much less any error qualifiable as gross - in the preparation of the initial project or in the introduction of changes and

adaptations which were made afterwards. The project was prepared on the basis of and in accordance with the information available at that time, and on the basis of data in existence, and later changes were introduced when indicated to be advisable by the laboratory tests.

(3) In regard to eventual errors in construction, they could only be ascertained after details of the state of the works before and after the accident become known.

(4) Careful inspection of the Works in the zone where the superstructure held up is of particular interest as it could eventually reveal the deficiencies in construction which are difficult to detect in the damaged zones.

(5) The details of the characteristics of the storm so far obtained, are of a nature conducive to the belief that despite its exceptional duration the Works were not subjected to seas with waves greater than allowed for in the project.

(6) For the above reasons, it is difficult and premature to pinpoint the causes of the accident. The explanation might be found in a combination of some of the hypotheses enumerated in the question.

(7) On the basis of available information, and without seeming to favour any of the hypotheses mentioned, we take it upon ourselves nevertheless to point out the deficient placing of the dolos in some zones, perhaps resulting from the use of floating equipment in limiting conditions of roughness of sea. Another aspect which might be found to be relevant is the settling of the core of the breakwater resulting from a combination of various factors, namely the presence of fines in the TOT, underwater breakouts in adjacent zones, excessively large elements in the secondary layers, etc. This settling, if in fact it took place, would cause directly or indirectly the breakdown of the dolos cover and/or the superstructure of the breakwater.

(8) It would be advisable to ascertain whether the effects of the settling of the breakwater core on the dolos cover, combined

with deficient fixing of the dolos, could have been the root cause of the process of destruction.

(9) In addition to the hypotheses presented in the question, we suggest the presence of excessively large rubble in the secondary layers and empty spaces having been left in some zones between the facing of the breakwater and the layers of dolos and rubble, thus exposing the TOT as can be observed in the appended photograph (Appendix 21).

Question 8 Assuming that it is not possible to define the causes of the accident in time to plan the reconstruction works for completion before next winter, what temporary works are required to be undertaken at once to protect existing structures?

Answers

(1) In accordance with the opinion recently expressed to the Inspection Engineer at the Works, it is a matter of urgent priority to protect all zones affected from the root of the mole to Berth 2 inclusive, as this Berth is necessary for supplying the refinery.

(2) Beyond Berth 2 also the mole does not appear to be in a condition to survive a normal winter, but it is feared that the short favourable period for repairs which are now beginning cannot be utilized for lack of adequate equipment.

(3) In these conditions, it is imperative to construct dolos of 40 tonnes at an accelerated rate to ensure the possibility of their placing with adequate equipment and in good time.

(4) To ensure the proper placing of dolos, it is recommended that the Contractor ascertains whether this summer there could be brought to Sines fixed platforms such as are used in various places for oil prospecting.

(5) The final solutions for repairing each of the damaged portions of the mole can be formulated only after the survey results, the results of inspection by divers, etc., are known.

(6) The reconstruction works must be properly supported by laboratory tests or by tests carried out by technicians of GAS and of the Planning Consortium, so that while the Works are in progress measures recommended on the basis of direct inspection can be undertaken without delay.

LIST OF ATTACHMENTS TO THE ORIGINAL REPLY*

1 - DESIGN OF BREAKWATER AT SINES HARBOUR by J.D. Mettam, presented to the "15th Coastal Engineering Conference," in Honolulu in 1976.

The criteria for determining project waves and the evolution undergone by the breakwater's cross-section. Contains some considerations on the internal tensions in the concrete of dolos, and a comparison with the tensions established in 20-t dolos.

2 - LETTER 534 OF 5-7-74, FROM BCL TO GAS

Describes the main changes introduced up to that date in the project. Mentions various intermediate phases of the project, the main deficiencies detected in the tests, and the measures taken to improve the profile.

3 - LETTER 196 OF 31-8-73 FROM BCL TO GAS, AND REPORT ON THE DIMENSIONS OF DOLOS TO BE USED IN THE MAIN BREAKWATER, ATTACHED.

Contains an analysis of the criteria which undermine the project waves and the dolos weight. Also considers the rising cost of labor and repercussion on eventual damages of the dolos cover.

4 - LETTER 4969 OF 7-9-73 FROM GAS TO BCL

Communicates the decision to change the weight of the dolos from 30-t dolos to 40-t.

5 - LETTER OF 24-9-74 FROM J.A. ZWAMBORN TO GAS

Contains an affirmation of agreement to the project solution and calls attention to the inadequate quality control of the works. Refers to the importance of the quality of concrete in dolos and its placing in works.

6 - NOTE ON THE CONTINUATION OF REDUCED MODEL TESTS, DATED 18-2-74, FROM BCL TO GAS

Refers to the criteria for determining the project wave and requests that LNEC introduce the corrections to the profile which are seen to be necessary to meet those criteria. Also offers a suggestion on the sequence of future tests.

7 - FIVE NOTES FROM BCL TO GAS ON THE PROGRAMME OF MODEL TESTS, SENT BETWEEN MARCH '72 AND JULY '73.

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* These attachments are in the files of the Port Sines Investigating Panel

Describes the project waves and their determining criteria, and suggest successively varied programmes for the different types of laboratory tests.

8 - <u>LETTER 283 OF 8-11-73 FROM BCL TO GAS</u>

Requesting that irregular wave tests should be conducted with progressively higher significant waves up to destruction.

9 - <u>LETTER 471 OF 4-4-74 FROM BCL TO GAS</u>

Calling attention to differences detected between the project and the execution of the protection covers and to the need for drawings defining the works actually executed. Attention also called to the fragile quality and deficient control of the TOT.

10 - <u>LETTER 530 OF 2-7-74 FROM BCL TO GAS</u>

Expressing doubts about the quality of materials placed and workmanship of the works particularly the quality of TOT, dimensions of secondary stone-work, curing of concrete in dolos, and method of placing various types of blocks. Makes recommendations on the organization of the inspection function. (This document is included in this Appendix and a response from Condotte d'Acqua is included in Appendix L.)

11 - <u>LETTER 796 OF 8-7-75 FROM BCL TO GAS</u>

Recommending additional field studies and observations.

12 - <u>LETTER FROM J.D. METTAM TO GAS OF 23-12-76</u>

Commenting on the visual observation of 3 to 4% of dolos which have split in place on the breakwater.

13 - <u>LETTER FROM J.D. METTAM TO GAS OF 4-3-77</u>

Commenting on visual observations made on the breakwater and enclosing annotated photographs of the upper covering with particular emphasis on the positioning of the dolos.

14 - <u>LETTER 466 OF 1-4-74 FROM BCL TO GAS ON PROPOSAL OF TECHNICAL ASISTANCE TO THE INSPECTION OFFICE</u>

Stating that the technical team proposed for assistance to the inspection office of the works would probably not be adequate. The proposal describes the assistnce recommended in accordance with the guidelines received from GAS and indicates the technical team which should be constituted. Attached are the minutes of the meeting in which this matter was discussed with GAS, including the fact that BCL considers that the technical team should consist of 10 BCL technicians appointed to the site, and not just the two proposed.

15 - LETTER 690 OF 28-2-75 FROM BCL TO GAS

Summarizing the evolution of the matter of "Technical Assistance to the Inspection Office."

16 - TABLE INDICATING THE ZONES IN WHICH DOLOS ARE USED

This chart contains mention of 56-t dolos to be used for the protection of a Nuclear Station (in course of projection), besides a resume of works already completed.

17 - LETTER 557 FROM BCL TO GAS OF 14-8-74

Suggesting tests with dolos to check their capacity of resistance and offering collaboration in the preparation of a test programme.

18 - TECHNICAL NOTE REGARDING THE PLACING OF DOLOS OF 14-8-74

In reference to some norms/standards and constructive recommendations for placing dolos, particularly in regard to the position of each dolos, the use of floating cranes, the construction of secondary layers and the use of models.

19 - TECHNICAL NOTE FROM BCL ON PROPOSALS (PAGES 12 TO 15) OF 25-5-73

Contains an analysis of construction procedures proposed by the various competitors, including statements on the placing of dolos and the risks of their splitting owing to the use of floating cranes.

20 - LETTER 157 OF 19-7-73 FROM BCL TO GAS AND ATTACHMENT

Commenting on Condotte's obligation to guarantee the possibility of placing all the dolos from the capping of the breakwater.

21 - PHOTOGRAPH TAKEN FEBRUARY 1978

Showing the separation between the concrete face of the wave wall and the layers of 3 to 6-t rocks and dolos. It establishes the fact that the TOT elements are not covered by the 3 to 6-t rocks nor by the dolos.

APPENDIX A (continued): Attachment 10

BERTLIN . CONSULMAR . LUSOTECNA

Letter to

The Director of
The Gabinete da Area de Sines
R. Artilharia Um, No. 33
LISBON

Our ref.
12.20.0 Date:2nd July 1974 Letter 000530

Re: Execution of Marine Works

Dear Sir,

Since you invited us to come to Sines to attend a meeting to discuss the problems experienced by the fishermen as a result of the work in progress, we took the opportunity to visit the site planned by the Group.

In the course of this visit, although we had neither the equipment nor the time to take measurements, we found that the work is not being carried out in accordance with the provisions of the Contract.

Although we are not responsible for the way in which the work is executed, we would like to draw to your attention certain aspects of the construction which may affect the future performance of the works.

Placing of TOT in the core of the West Breakwater

The specification states (page 266) that the rock to be used in the breakwater may not contain more than 5% of material weighing less than 1kg. During the visit, the TOT in the core was found to contain fines in an amount exceeding that specified. Fines located inside the breakwater will eventually be swept away by the waves, sooner or later, through the interstices in the rock which will result in voids being created within the body of the breakwater and, consequently, to a settling of the superstructure and the protective armour layers. When this happens, the breakwater is liable to be damaged in stormy weather. Repairs to the breakwater to avoid this risk would entail rebuilding the layers of rock beneath the dolos which will be extremely expensive.

One of the precautions which should be taken to avoid the presence of excessive fines involves removing overburden before quarrying. This has not always been checked as we discovered in the course of our visit.

Secondary armour layers in the West Breakwater

The thicknesses of the armour layers in the West Breakwater, whose function is to prevent the passage of adjacent materials, were selected in accordance with the sizes of their constituent elements (page 266 of the Specification). We discovered that in the secondary layers there was an appreciable quantity of inordinately large boulders almost the thickness of the layer, which, by leaving large gaps between boulders, prejudice the function of the layer.

For this reason, we feel it is vital for the Fiscalisation constantly to check the maximum and minimum boulder sizes for use in secondary armour layers.

Grooves in the central stem of dolos units

Dolos in the yard and those already placed have grooves in the middle of the central stem. Some of these grooves have extremely sharp angles where a concentration of stresses may lead to breakage of the dolos when these are exposed to wave impact.

We feel that immediate steps should be taken to avoid these grooves.

Curing of the concrete used for dolos units

The strength of the dolos depends upon the conditions under which the concrete was cured, hence the reference on page 287 of the specification 7.3.7 to the care which should be exercised in curing.

The cracks which we noted on the surface of certain dolos prompt us to believe that the provisions given in the Specification as to such care has not always been heeded.

Dolos identification

The dolos which we examined bore no markings as to the dates of concreting. It is stated in the Specification (page 303) that all units should be marked with a reference number plus the date of manufacture so that in the event of the concrete cube tests indicating the need for rejection, the offending units could be identified.

Placing of dolos and quarry stone on site

The dolos provide an interlocking structure which contributes greatly towards the stability of the covering and which is very difficult to obtain in components of other shapes. To ensure that they are interlocking, especial care has to be taken in assembling them on site as a result of which the Specification stipulates (end of page 268) that both the primary layer and the secondary armour layer should be made to the complete thickness from top to bottom.

Tests performed at LNEC have confirmed that the performance of dolos layers placed as a complete thickness was better than that of dolos layers in two superimposed layers. Consequently, neither the rocks nor the dolos in the same course should be placed in stacked lines without a progression from the bottom to the top of the course and throughout its entire thickness.

We found that, on site, a line of dolos was being positioned and on top of it a second line.

The specification gives the required number of dolos per unit surface area (page 416) and makes it incumbent on the contractor to test, with the aid of models, the methods of placing (page 319). It further states that 60% of the dolos should be placed with the vertical leg turned towards the outside of the slope. In so far as we were able to tell, these provisions have not been respected.

The placing of dolos and quarry stone on site is an operation which has to be carefully followed by the Fiscalisation, since the behaviour of the breakwater under storm conditions will depend on this operation.

Breakwater for the Construction Harbour

We found, as in the West Breakwater, that the specifications to TOT (page 266) were not being respected in regard to the breakwater for Construction Harbour. Precautions must be taken to avoid an excess of fines in the TOT and to ensure removal of earth placed on the breakwater to facilitate movement of equipment.

It was further discovered that the rock layers had not been positioned with the accuracy necessary for ensuring perfect performance of the breakwater. It would be as well to check systematically the sizes of the elements and thicknesses and positions of the rock layers.

Blocks for the construction harbour quay

It was discovered that in some of the blocks already completed, pipes connected to the underside had been included to facilitate fitting of hanging straps. These pipes break up the continuity of the underside of the block, thus reducing its section of resistence and, hence, its capacity to withstand flexural forces. If the pipes had been fitted half way up the blocks, close to its neutral fibre, not only would there have been no loss of strength but also an improvement in the state of equilibrium during transportation and placing.

Recommendations

Concerned about the short-comings of the work under reference and fearing that others may have escaped our notice during the visit to the works, we should like to draw your attention to the gravity of the situation and to recommend the organization of local supervision backed up by personnel and equipment whose function it would be to ensure constant compliance with the provisions of the project and of the Specification.

We would emphasize that, in view of the fact that virtually the entire structure will be below water, the materials and elements of which the structure is made, it will be extremely difficult later on, in the event of an accident, to establish the standard and accuracy of placing of these materials and elements. For this reason, there has to be extremely careful, uninterrupted supervision, right from the start of the work.

Yours faithfully,
For the Group,
(Signed)
F. Vasco Costa

APPENDIX B

Official Replies of Societa Italiana per Condotte d'Acqua to Inquiry of Portuguese Investigation Committee

(This is an official response to the designer's concerns expressed at ASCE Coastal Structure Conference March 1979 about deficiencies in instructions. Presented to the Official Investigation Committee by Condotte d'Acqua.[See Mettam, 1979])

1. INCOMPLETE PARAPET WALL

As we have already had the opportunity to mention, the Consultants classified as "a known deficiency" the circumstance that the parapet wall is incomplete, the fact being known that the storm hit the breakwater when it was still under construction.

The works proceeded in accordance with the construction phases (see Constructive Process in the annex to our letter No. 307/78 of April 15, 1978) set forth and approved by the Inspectorate of the Sines Area Bureau and according to the Program of Works also approved by the Inspectorate; therefore, the situation at the time of the storm was the following:

- the parapet wall was incomplete from the 1656 distance of origin up to the head (see annex 1).

On the other hand, the Consultants do not indicate how such pseudo "known deficiency" could constitute a "cause" of the accident which occurred in the breakwater, when it is undeniable that that part of the works, even though incomplete, suffered much less damage than other zones totally completed.

2. UNFINISHED DOLOSSE ARMOR UNIT ALONGSIDE THE PARAPET WALL

Also in this case, the circumstance that the work was not complete is classified as a "known" deficiency.

As can be verified through the construction phases (see Constructive Process already mentioned), only after the completion of the parapet wall from the level +16(ZH) to the level +19(ZH) was the emplacement of the last dolosse of the upper part carried out.

In the Occurrence Proceedings written by the Sines Area Bureau on March 18, 1978, the situation of the dolosse armor unit at the time of the storm is described as follows:

- at trunk 51, 15 dolosse had not as yet been placed;
- from trunk 52 to trunk 56, 45 dolosse were missing;
- at trunk 72, 15 dolosse were missing;
- from trunk 79 to trunk 81, 45 dolosse were missing;
- from trunk 82 to trunk 83, 28 dolosse were missing;
- at trunk 84, 15 dolosse were missing;
- from trunk 85 to trunk 87, 26 dolosse were missing;
- from trunk 88 to the head, approximately 1 dolos per lin. mt. was missing.

As we already had the opportunity to mention to the Sines Area Bureau in the Request concerning the Occurrence Proceedings sent to the G.A.S. with our letter No. 406/78 of June 14, 1978, between trunks 51 and 72 only 34 dolosse were missing, and not 75.

On the other hand, the Consultants do not indicate how such pseudo "known deficiency" could constitute one "cause" of the accident which occurred at the breakwater.

3. SECOND LAYER OMMITED EXPOSING THE 0.25 - 1 TONN. LAYER

4. VERY LOW SECOND LAYER

Also in point 3, the fact that the work was not completed at the time of the storm is, for the last part, considered a "probable deficiency".

The situation of the works at the time of the storm (according to the Program and the Constructive Process) also referred to in the Occurrence Proceedings and confirmed by us, was the following:

- From trunk 100 and up to the head inclusive, the enrockment of 3 to 6 tonn., as a protection to the parapet wall, was missing.

This phase of the work was in the process of execution when the storm occurred (see annex 1).

With regard to the rest of the breakwater, the Consultants indicate in point 3 the omission of the second layer as a "possible deficiency". In point 4 they state that it was possible that a second layer, at full extension of the breakwater, was placed at a very low level. In these two observations - point 3 and 4 - there exists an evident contradiction: in point 4 they state that the 3 to 6 tonn. layer existed but it was possible that it was placed at a lower level, whereas in point 3 they state that it is possible and probable that such layer did not exist, exposing the enrockment of 0.25 to 1 tonn. If the second observation is sound (point 4), the 3 to 6 tonn. layer existed and would replace the enrockment of 0.25 to 1 tonn. which obviously could not stay exposed, as stated in point 3.

In any of the cases we reject such possible and probable deficiencies, since the execution of the work has complied with the project.

Finally, it is not indicated how such possible and probable deficiencies can constitute "causes" of the accident which occurred at the breakwater.

5. INCORRECTLY PLACED DOLLOSE

The Consultants consider as a "known deficiency" dolosse incorrectly placed up to approximately the 1650 distance of origin, and as a "probable" deficiency dolosse placed from such distance up to the head.

As we mentioned in the introductory part of this reply, contrary to what the Consultants state, the photographic documents demonstrate that the dolosse are well placed and that they have the necessary degree of interlocking.

As we have already stated, this is also the opinion of the technicians to whom we showed the series of photographs taken on November 4, 1977.

Since, up to this moment, we have not had the opportunity to submit such photographic document for your consideration, allow us now to add it to our reply.

As previously stated, the document is dated November 4, 1977, prior, therefore, to the intervention for the purpose of completion requested by G.A.S. and carried out in the month of January 1978 between the 828 and 870 distances to the origin (annex 2).

We would again recall that the dolosse were placed according to the instructions received - in the presence of the representative of the G.A.S. Inspectorate and the Consultants - of Eng. Merrifield and of Eng. Zwamborn, during the visit which took place from November 4 to 12, 1974 to Cape Town, Gaansbaai, Port Elizabeth, East London, Durban, Richard Bay and to the Stellenbosch Laboratory.

6. SECOND LAYER WITH EXAGGERATED THICKNESS OR WEIGHT OF BLOCKS

The Consultants, in their letters of July 2, 1974 and August 14, 1974, sent to the Inspectorate and transmitted to us for our cognizance, stressed, among other things that "the thickness of the covering layers of the west breakwater, the function of which consists in preventing the vanishing of the underlying materials, was chosen in accordance with the dimensions of their constituting elements (page 266 of the Specification Form). We verified that in the secondary layers there had been placed a substantial amount of blocks with exaggerated dimensions, near the thickness of the layers which, by originating great spaces among the blocks, compromises the function of the layer. For this reason, it is considered indispensable that the Inspectorate ensures permanent control of the upper and lower limits of the dimensions of the blocks to be applied in the secondary covering layers".

Now they define as "known deficiencies" those indicated in due course in the trunk of "enraizamento" (rooting), and they arbitrarily extend the possibility that those deficiencies are repeated in the entire breakwater.

With regard to the part concerning the rooting, we already had the occasion of replying to the aforementioned letters of the Consultants in our letter No. 718/74 of August 27, 1974, in which we stated among other things: "At the time of the Consultants' visit, the part of the breakwater concerning the rooting was under execution. For this part, some uncertainties arose about the type of covering to be placed under the dolosse. As a matter of fact, drawing 150A indicates the enrockment from 0.5 to 3 tonn., whereas drawing 173, from 3 to 6 tonn. Therefore, it is very probable that the Consultants had seen the emplacement of those with larger dimensions, since drawing 173 was under execution".

Regarding the dimensions and weight of the enrockments to be placed, we emphasize that in accordance with the request of G.A.S. in their letter No. 11/74 of September 16, 1974, in which it was asked:

> "1 - Before September 30, 1974 you must order the placing in the Quarry, in the West Breakwater or in the Construction Harbor of significant samples, conveniently identified and protected, of all the granulometries of the enrockments which are being utilized in the works, having, for the effect, agreed with our official Mr. Fonseca upon the process of measuring the limits and percentage of each granulometry.
>
> "2 - The nonfulfillment of such instructions will compel this Inspectorate to order the interruption of the works until the said instructions are complied with."

Condotte replied by letter No. 822/74 of September 25, 1974, in the following terms: "With reference to your letter No. 11, although we would remind you that the blocks of sample rock had already been, in due course, placed in the quarry and in the dike, as was verified by your geologist, we hereby confirm that we will issue an order that other blocks be placed in evidence, in conformity with the manner and in the zones to be arranged with Mr. Fonseca".

We hereby declare, therefore, that the Inspectorate as well as Condotte were well aware of the importance conferred by the projectors on the dimensions of the blocks and that, since the beginning of the works, the necessary steps were taken for identification of the weight of the blocks, for their classification and for the orientation of the personnel in charge of them.

Finally, concerning the "thickness of the layer", defined as "exaggerated" by the Consultants, we call to mind the Constructive Process approved by the Inspectorate of G.A.S. and the methods of control always executed in conjuction with G.A.S., which we have already had the occasion to mention to you in our letter No. 425/78 of July 18, 1978, which we enclose (annex 3).

We stress that, while the Consultants attribute to such layer the only function as being a filter of fine material, Eng. Marrifield, regarding the primary layers of stone under the dolosse armor unit, thinks that:

> It is also essential that the primary layer of stone be thick enough to allow good and quick drainage of water that has passed through the armour layer. I can see no good purpose from the drainage point of view in narrowing the primary layer of stone as it reaches the upper section of the wave zone".

There is, hence, a different concept towards the elements of the project concerning the primary layer: Merrifield considers it a drainage of water and not a layer of material which prevents the exit of fine material of the core, as, in turn, the Consultants consider it.

On the other hand, the Consultants do not indicate how such pseudo "known deficiency" can constitute a "cause" of the accident which occurred at the breakwater.

7. LAYER OF 0.25/1.0 TONN. WITH EXAGGERATED THICKNESS/WEIGHT OF THE BLOCKS (PHOTOGRAPHS SHOW THAT THE 0.25/1.0 TONN. LAYER DOES NOT EXIST)

The Consultants state that the 0.25-1.0 tonn. layer of the enrockment is exaggerated in thickness and in the dimensions of the elements (blocks); this deficiency is evidenced by photographs which "show that no layer of 0.25-1.0 tonn. exists".

As the photographs at issue are not in our possession, we are not in a position to be able to comment on them.

That deficiency, however, is indicated in the drawing as "possible" for the entire extension of the breakwater, whereas for a little trunk corresponding to Post 2 it is classified as "known".

Regarding the point at which the drawing indicates one "known deficiency", neither we nor G.A.S. have knowledge of the fact that at that point the 0.25-1.0 tonn. enrockment is missing. As to the "possible" deficiencies, we also reject such statment and declare that we are ready to execute all the verifications you deem opportune and which are still possible.

We have already mentioned many times the ways and systems of choice of each class of enrockment (visual comparison with sample elements placed in evidence at several points in the yard) as well as those of emplacement and control thereof (depth gauges, topographic surveys, etc., executed in conjunction with the Inspectorate). It is left to us only to add that all the phases of work respected the Constructive Process approved by the Inspectorate and were executed according to the project, and that the Inspectorate (see exchange of correspondence) was extremely sensitive to this problem.

We also recall the different interpretation of the layers of enrockment underlying the layer of dolosse given by the Consultants and by the inventor of the dolosse, Eng. Merrifield.

The remark which is in the drawing produced by the Consultants - "Settlement observed in December 77" - is not clear because, in addition to not indicating the value of such settlement, the type of material concerning the settlement at issue is not specified. Concerning this, we can only recall what we have already mentioned when answering question No. 10, point 4, of the questionnaire you sent us on April 5, 1978.

On the other hand, the Consultants do not indicate how such pseudo "known deficiencies" and "possible deficiencies" can constitute "causes" of the accident which occurred at the breakwater.

8. SELECTED T.O.T. WITH MANY FINES

In the letter of the Consultants - quoted in the drawing - dated July 2, 1974, the following is written:

> "Placement of T.O.T. in the core of the west breakwater
>
> The Specifications Form stipulates (page 266 that the enrockments to be utilized in the breakwater will not contain more than 5% of materials with less than 1 kg. During the visit the presence of fines was observed in the T.O.T. of the core in a quantity superior to that which was indicated. The fine materials which will be placed inside the breakwater will sooner or later end up being carried away by the waters, through the crevices of the enrockment, which will originate voids in the body of the breakwater and, consequently, settlements of the superstructure and of the protection layers. When that happens the breakwater will be subjected to damages during storms. The repairing of the breakwater to prevent such a risk will involve the reconstruction of the layers of enrockment under the dolosse, a reason why it will be extremely expensive.
>
> One of the cautions to be taken in order to prevent the presence of fines in excess consists of the removal of the soil which covers the quarry before carrying out the work therein, which has not always been observed, as we discovered during our visit."

On August 27, 1974, Condotte answered as follows:

> "Emplacement of the T.O.T. in the cores
>
> The Specifications Form sets forth that the T.O.T., both the selected and the normal, may contain pieces of rock under 1 kg. which do not exceed 5% of the weight. Since, in the breakwater,

an average of 250 daily trips are made by 60-ton dumpers and the total amount transported is 15,000 tons, 750 tons of material below 1 kg. could be transported. Since the specific weight of that material is 2.00 tons/m^3, 375 m^3/day of fine material could be transported.

As a matter of fact, dumpers have a capacity of 65 tons; hence, T.O.T. below 1 kg. should be 3.22 tons per trip, corresponding to 1.60 cu. meters of fine. This quantity was never placed.
In addition, a test was carried out in the presence of your geologist on August 2 last, and the percentage of fine material reached about 2/3%, and on August 6 last, with a dumper chosen at random, we unloaded on land in the presence of your consultants, who admitted that the percentage was on sight much less than 5%.

On the other hand, under the supervision of the Works Inspector, we took the following steps in order that the material carried to the breakwater would be the best possible:

1) elimination of the sterile covering material

   This elimination is effected under the surveillance of the Inspectorate and in such a way that, as we already had the opportunity to communicate to you several times, after the cleaning access to the higher part of the quarry is not possible most of the time, not even with small drilling equipment, which even forces us to carry out explosions of rectification to give room to a surface passable for our means;

2) removal of nonsuitable material from the front of the quarry

   This material, which is in the crevices of the rock, is sent to the dumping ground. For example, in the period from August 1 to August 25 about 17% of the material was sent to the dumping ground;

3) high capacity pumps were placed on the advancement of the breakwater and the material was vigorously dampened for the purpose of eliminating the finer part;

4) by means of points of reference placed on the breakwater, the settlements of the very breakwater are measured. Up to now, in the zone initially executed the settlements are of a few centimeters.

The measurements mentioned in items 2) and 3) were taken although the contract did not envisage this to be the responsibility of the Company.

From the aforementioned, we think the material which is transported to the breakwater is fully acceptable. If, however, the consultants have an opinion to the contrary, obviously the question no longer concerns the exploration of the quarry but rather the nature and conditions of the material which constitute the quarry itself."

Neither the Consultants nor the Inspectorate of the Sines Area Bureau made at any time any remark to this letter of ours, so that for us the matter was closed.

Up to December 31, 1977, the quantities of unserviceable material removed and transported to a dumping ground had reached 5,580,000 cu. meters in approximately 7,000,000 cu. meters of material utilized for the works, that is, 44% of the material removed from the quarry had been discarded. It seems to us that this fact constitutes an obvious indication of how rigorously the Inspectorate made the selection and of the criterium of the same selection, which Condotte never contested.

On the other hand, an excess of fine material would, as a consequence, have had important settlements of the breakwater, settlements which did not occur.

Insofar as the photographic document produced by the consultants is concerned, a document relating to the "beach" of fine material in the zones of trunks 100 - 101 - 102 - 103, although we do not have such document, we wish to call attention to the fact that the fractionization of the material at the moment of placement respected what was set forth in the project. In the mentioned zones and after the storm, a considerable and gradual reduction of the granulometry occurred as well as a roundness of the elements due to erosion (sample 1), caused essentially by the dynamic action of the sea. We enclose a photograph taken about ten days after the storm and which shows a fractionization of the material very different from the one registered some time afterwards (beach of fine material) by the Consultants (annex 4).

Let us once again bear in mind that, with the authorization of the Inspectorate, a layer of sand or tiny material was spread over the breakwater at level +5.50 in order to permit the transit of heavy vehicles which effected the transportation and unloading of the material. This layer was removed when the construction of the superstructure was initiated. If the

Consultants, in their observations, mention this layer, it is obvious that such material was not part of the selected T.O.T., as it constituted only an indispensable precaution to facilitate the transit of heavy vehicles.

The statement according to which there are photographs "showing holes in excess" in the selected T.O.T. during the construction is not clear, since for the same material the consultants declare that there is an excess of fine material. If the holes (voids) in the selected T.O.T. are in excess, there is a lack of fine material. If the fine material is in excess, there can be no "holes".

The Consultants do not point out how such pseudo "known deficiencies" and "probable deficiencies" could constitute "causes" of the accident which occurred at the breakwater.

9. POSITION OF THE WINTER HEAD WHICH MAY NOT HAVE BEEN COMPLETELY REMOVED

Since 1975, at the beginning of each winter period, a provisional protection of the breakwater was constructed which was to be eliminated before recommencement of the works in periods favorable to construction (see vol. IV attached to the contract).

That protection was built in the winter of 1974/75 with the use of enrockment, whereas in 1975/76 and 1976/77, 40 ton dolosse were also utilized.

The removal of the protections was executed with the use of maritime and land means and with the help of divers who sometimes, as agreed with the Inspectorate, resorted to explosive charges applied on the dolosse in order to prevent the retention of elements of great dimensions in the core.

Following is the number of dolosse used in the heads during the years 1975/76 and 1976/77 and of those which we recuperated, as set forth in the Inspectorate Reports:

Provisional Head	Distance from origin	Placed	DOLLOSE Recuperated	Destroyed
Winter 75/76	1075	193	171	22
Winter 76/77	1680	172	144	28

Before recommencing the unloading of the T.O.T. and the emplacement of the enrockments, underwater reconnaissance by the divers of G.A.S. and Condotte were carried out in order to make sure that the material placed for protection had been completely removed.

Only after such verification was the reinitiation of the T.O.T. placement authorized.

Therefore, the possibility acknowledged by the Consultants that the winter protections had not been "completely removed" is excluded.

The Consultants, however, also did not indicate how such "deficiency", admitting that it existed, could have originated the accident which occurred.

10. INCORRECTLY PLACED DOLOSSE

In this item, a probable deficiency, a pretended incorrect placement of the dolosse, is considered. In a footnote it is afterwards added that "the position and placement of the dolosse are incorrect."

At the same time, however, it is said in these same notes that surveys do not exist. We would like to know how, without surveys, the consultants can evaluate whether or not the position of the dolosse is correct.

With regard to the sentence: "dolosse placed by a floating crane", it is not clear whether the consultants consider that to be a deficiency or whether they simply wanted to make a distinction between dolosse placed by land and those placed by sea.

Actually, the systems of dolosse emplacement and the forms set forth by the Constructive Process approved by the Inspectorate, as well as the surveys previously carried out during and after the emplacement, gave the guaranty that the dolosse layer was being executed as envisaged in the project.

When dolosse were breaking in the course of emplacement, they were taken away and replaced.

During the dolosse emplacement from 1974 to 1978 alone, 186 dolosse out of 20,113 were broken and replaced, that is, 0.9%, as is recorded in the minutes opportunely sent to you.

The same survey made by G.A.S. and attached to the Occurrence Proceedings, which we rejected as it was not carried out in conjunction with us and was not communicated in due time, shows a limited number of broken dolosse; the justification of those fractures, as we informed you, must be attributed to sundry causes but, in any case, never to placement by sea.

On the transportation and placement of the dolosse, we comunicated the following matters to you:

In answer to the first questionnaire (see our letter No. 307/78 of April 15, 1978):

- point No. 2: the Constructive Process approved by the Inspectorate of G.A.S. was sent;

- point No. 6: the equipment utilized for the transportation and the gripper for the picking up and placement of the dolosse were described;

- points 9.1 and 9.2: the equipment utilized in the placement of the dolosse was described;

- point 9.6: the direct system for the control of eventual breakings during the placement by pontoon was pictured;

- point 9.7: we mentioned the control systems and the inspections during the emplacement of the dolosse;

- point 9.8: the rules and the criteria followed during emplacement were pictured.

In answer to the second questionnaire (see our letter No. 377/78 of June 7, 1978):

- point No. 2: we talked afterwards about the placement by sea and particularly about the influence thereon of the meteomaritime conditions;

- point No. 3: we also described the control systems of the emplacement by sea;

- point No. 4: we mentioned the systems of dolosse emplacement following the rules suggested by the consultants;

- point No. 7: we talked about the test made for the emplacement of the dolosse.

We summarize below the considerations which determined the choice of the equipment utilized:

The organization of the work year and the choice of the equipment necessary for the execution of the works are, obviously, related to the geometry of the work to be executed, to the envisaged completion schedule and to the conditions imposed by the Specifications form for the execution.

In the case of the west breakwater at Sines, the following circumstances:

1 - the need of having to place dolosse up to a distance of 55 m. from the axis of the breakwater, that is, of having a "system" available which at 55 m. has a capacity of about 50 tons (42 for the dolosse and 6 for the gripper);

2 - the impossibility for the support of a crane to make use of the superstructure in concrete at level (+8,00), as this could be built,

according to the rules of the Specifications form, only three months after completion of the breakwater, after verifications of settlements had been completed. Thus, in case a crane was installed in the superstructure, about 250 m. of the breakwater under construction would be without protection of dolosse;

3 - the need of having to carry out "as soon as possible" the protection with dolosse of the constructed core;

4 - the need of separating the emplacement of T.O.T. and of the enrockments from that of the dolosse, that is, not having only one means for their emplacement;

5 - the lack of space in the advancement at level (+ 6.00);

6 - the need, because of times of execution, of having available, for each phase of the work, means which can be interchangeable according to necessities so that no operation had to be suspended;

7 - the need to avoid that the advancement depended on only one equipment which, due to its dimensions and position, would be extremely vulnerable

led to the choice of the following system:

- the dolosse are placed by sections of 20 lin. meters forming all the thickness of the project layer, from the bottom upwards in three successive phases.

  The first phase, by sea, by means of a pontoon equipped with an M25 American Hoist crane, envisages the emplacement of dolosse from level - 15,00 Z.H. to level + 4,50 Z.H.

  The second phase, by land, with an American Hoist 11250 caterpillar crane, from level + 4,50 to level + 8,00 Z.H.

The third and last phase also from land with the same caterpillar crane, from level + 8,00 Z.H. to level + 16,00 Z.H.

This system was approved by the Owner of the work and by the Consultants before the adjudication of the contract work.

The system has the following characteristics:

- before carrying out the emplacement of the dolosse, the slope of the enrockment below the dolosse layer and the support banquette was verified with manual depth gauges and inspections by divers. Once it was verified that the levels obtained through the depth gauge respected those of the project, the emplacement of the dolosse was carried out.

For the emplacement, in the construction of the main layer, the following rules were followed:

a) satisfactory degree of interlocking between one element and another;

b) stable position of each one of the elements;

c) formation of the armoring in all its thickness from the base;

d) placement of the elements in a manner so that 60% remained with the vertical leg turned to the outside of the slope;

e) replacement of the dolosse which were broken at the time of emplacement;

f) execution of the armoring with the utilization of such a number of elements as to comply with the project cross section. The project density is 0.16 dolosse/sq. m. and the density achieved is 0.18 dolosse/sq. m.

Zwamborn, in the mentioned report, justifies in the following way the difference between the envisaged density and the density achieved:

"<u>Dolos packing density</u>

The prescribed packing density was based on that used originally, in South Africa, viz. (double layer):

$$N = 1.04\ V^{-2/3} = 0,160/m2$$

$$N = 2$$

(for $V = 16,58\ m^3$)[N,12] where N is the number of blocks per unit area. This constitutes 8,56 40 T. Dollose per m length of breakwater and is 4 per cent above the "mean" packing[3] density defined by Zwamborn

A layer thickness of 6,5 m was accepted for the design which corresponds with a shape factor <u>C =</u> 1,275. This value agrees with the original roughly cast cement mortar models and the value of C corresponding to the above value of N is C = 1,08 which results in a double layer thickness of $T_n = nC_nV^{1/3} = \underline{5,5}$ m (n = 2). The 6,5 m aimed for in the project is thus 18 per cent thicker and it is no wonder that considerably more Dolosse had to be placed to achieve the prescribed cross-section.

Records provided by Condotte show an overall coverage packing density of 0,1816 Dolosse/m^2. This packing density is obtained by increasing the value of the constant in the above formula for N from 1,04 to 1,18, representing an increase of 14 per cent in the number of Dolosse, N. The expected layer thickness then follows from the corresponding shape factor value[13], i.e. $C_n = 1,23$, namely $t_{n=2} = 6,27$ m which is fairly close to the required 6,5 m."

g) emplacement in such a way as to respect the project cross section, regarding inclinations and thickness.

These rules are in conformity with those suggested by the Inspectorate.

The emplacement by sea was executed the following way:

- positioning the pontoon through fixed points on land and bouys on the sea, performed by a topographic team; the pontoon was anchored with two bottom anchors and at least two cables on land to prevent its movement during the emplacements;

- lifting of the dolosse with an M25 American Hoist crane equipped with special grippers molded according to the shape of the element and having a guiding device for orientation of the dolosse (tak-line) in their emplacement;

- control by the diver after completion of the emplacement in order to verify eventual fracture of the elements.

The placement of the dolosse by pontoon was suspended as soon as the meteomaritime conditions might involve risks of fracture of the elements.

The emplacement by land with an American Hoist 11250 crane equipped with special grippers and with a guiding device (tak-line) was executed with the support and the control of the topographic team.

In the course of the breakwater repair works commenced on August 29, 1978, from August 23, 1978 until September 26, 1978, 742 dolosse were placed by pontoon, 188 of which were armed and 554 unarmed. Only two were broken.

This emplacement was carried out in the same sea conditions, with the same means, with the same systems and the same men utilized in construction of the breakwater, but now in worse conditions because of the irregularity of the slope as a consequence of the storm, which afterwards demonstrates that the placement with a floating crane is not a cause for fracture of the dolosse.

If, in spite of everything, the Consultants still have some doubt in that respect, they have only to go and watch the works still under way.

With regard to the model which, according to the Consultants, was not adopted, we must bear in mind that following the conversations which took place in South Africa among Engineers Merrifield and Zwamborn, and Eng. Biscaia of the Inspectorate of G.A.S., Eng. Silveira Ramos, of the Consultants and Engineers Paolella and Agostini from Condotte, there arose the determining necessity of executing the armoring in only one layer and of interlocking the dolosse as much as possible. These rules were absolutely respected during the construction without any comments whatsoever from anyone.

In time and after a trip to South Africa, the execution of a model on a reduced scale turned out to be unfeasible for meeting the requirements and preference was given to a model on actual scale. As we stated when answering question No. 7 of the second questionnaire, it was in this way that it was decided to place the dolosse, in the presence of the Inspectorate of G.A.S. and of its Consultants, all along a sample trunk of the breakwater of 20 lin. meters.

This test was considered fully satisfactory and therefore, the dolosse utilized for that purpose were not withdrawn and the subsequent emplacements were caried out following the same scheme. We are not aware of any letter from the Consultants commenting on this test.

11. SECONDARY LAYER WITH INADEQUATE WEIGHT/EMPLACEMENT

The Consultants define as "possible deficiencies" the weight of the second layer blocks and the system of emplacement in the entire breakwater.

It is also stated, and in this case as well, that: "there is no information of surveys, there are no verifications made by G.A.S.". We reject, as false, these arbitrary statements.

Contrary to what the Consultants write on this matter (which repeats what is written concerning point 10), the "surveys" and the "verifications" were made (see group of surveys carried out by Condotte together with the Inspectorate and which were forwarded to you with our letter No. 425/78 of July 18, 1978). We would call to mind that the surveys, reproduced graphically in this group, represent only one part of the verifications carried out during the construction of the breakwater.

As we already had the opportunity of informing you, the selection of the various classes of enrockment was made by visual comparison, taking as a reference sample elements previously weighed, cubed and exposed in well visible places of the yard. In addition, the enrockment was controlled by the Inspectorate, both at the exit of the quarry as well as before being placed, and even during its emplacement.

11.1 INADEQUATE EMPLACEMENT

The emplacement of the enrockments was carried out according to the form prescribed in the Constructive Process approved by the Inspectorate and always in the presence of one representative of the latter.

The Consultants do not explain why the emplacement system is inadequate nor do they indicate "adequate" alternatives for the adopted system.

What are the criteria of evaluation and on what experience of construction of deep water harbors are the Consultants based to define as inadequate a system of execution of one phase of work, the selection of which was made in terms of dimension of the works and of their technical-constructive requirements?

On the other hand, it is also not indicated how that "inadequate placement" could constitute the "cause" of the damage which occurred in the breakwater.

11.2 METHOD DESCRIBED BY THE ENGINEERS OF THE WORK (FROM CONDOTTE? FROM G.A.S.?) DOES NOT FACILITATE IN AN ACCURATE EMPLACEMENT

We do not understand the meaning of this sentence. According to the provisions of the Specification Form, Condotte presented to the Inspectorate the Constructive Process for the west breakwater. The Inspectorate approved this Process and, pursuant to the conditions set forth therein, carried out the work. We are not aware that the Consultants have indicated other systems different from those which were envisaged and approved.

12. DOLOSSE SUPPORT FOOTING WITH INCORRECT PLACEMENT AND DIMENSIONS

The placement of the enrockment of 16/20 tonn. to build up the support banquette of the dolosse armor unit was being carried out in conformity with the Constructive Process, approved by the Inspectorate.

We may also add that the enrockment of 0.5/3.0 tonn. is, as the project sets forth, placed in the outside slope of the breakwater at more than 60 lin. meters of distance from the axis. In these conditions, we do not understand what equally sound system of emplacement could be adopted, in addition to that envisaging the direct unloading of the materials through floating methods. We wish to stress also that in this phase of the works the verifications and surveys were carried out as set forth in the Constructive Process.

On the other hand, it is not indicated how such "possible deficiencies" could constitute a "cause" of the damage which occurred at the breakwater.

15. TOE OF THE TOT WITH INADEQUATE DIMENSIONS AND EMPLACEMENT

The TOT was placed, as planned, through the unloading of barges which open at the bottom.

The verification of the material loaded on board each barge was made by the Inspectorate.

After the emplacement of the TOT, the dimensions of the toe were verified through surveys.

It is not understood on what ground the hypothesis is based that in some trunks there were possible deficiencies and in others there were not.

We do not know the elements of evaluation which the Consultants used as a basis for considering that the emplacement effected by us was inadequate.

At the same time, they never indicated any alternative to the system adopted.

On the other hand, it is not pointed out how such "possible deficiency" could constitute one "cause" of the accident which occurred at the breakwater.

16. INEXISTENT LAYER OF FILTER

In the Map of Measurements attached to the contract, the execution of the filter was an item considered "conditional".

The Sines Area Bureau had the option of asking or not asking its execution, at a time considered opportune.

Without this order, Condotte was not authorized to execute such filter.

By its letter No. 88/76 of March 9, 1976, the Sines Area Bureau requested that the filter be executed.

After this request, Condotte carried out the work which had been ordered.

The progressive of the breakwater advancement at the date of the Bureau's letter was approximately 1,400.

The filter was executed with the dimensions set forth in the project and in agreement with the Inspectorate, beginning, obviously, at the above mentioned distance from origin.

On the other hand, the Consultants do not indicate how such pseudo "known deficiencies" and "probable deficiencies" could constitute "causes" of the accident which occurred at the breakwater.

CONCLUSION

Globally examining the document of the Consultants we may conclude that the remarks therein are extremely oversimplified, not to use a stronger word.

They consist in ascribing to the execution some defects that not even the Consultants themselves know if they exist - if they knew, they would not call them "probable" or "possible" - and, although not putting it in writing, they want to make believe that all those pseudo defects of execution were the cause of the accident. However, they did not even attempt to demonstrate, defect by defect, what was their possible influence in the accident as it occurred.

And, on the other hand, they completely ignored all the circumstances - in addition to such pseudo defects - which could have caused the accident. Thus, they were not interested in knowing what had actually been the condition of the sea during the storm period; they did not take into consideration the tests in model performed at the laboratories in Lisbon (L.N.E.C.), of Canada (National Research Center) and of Paris (L.C.H.F.); they did not take into consideration whatsoever the qualified opinions of experts recognizedly experienced in maritime works in deep waters. For them everything comes down to throwing doubts on the Inspectorate and the execution of the works in an arbitrary and oversimplified manner.

It is not incumbent upon Condotte to fight for the Inspectorate of the G.A.S.; however, you have in your possession the complete documentation and the correspondence exchanged between G.A.S. and Condotte during the four years the construction of the breakwater lasted. In this way, you will be able to verify that the Inspectorate performed its function rigorously.

Once the "drawing" of the Consultants is studied, we may legitimately ask: what do they seek to attain with that drawing, after all?

If they aspire to show the existence of defects of execution - had they or had they not contributed or could they have contributed to the accident - the drawing serves only to cast confusion.

If they seek to insinuate that all the pseudo defects appearing in the drawing were causes of the accident, they should have gone to the trouble to prove that causality and if they did not do so it was because it was manifestly impossible and, therefore, they did not dare to do so.

And, in conclusion, we would emphasize an interesting point.

When the Consultants in their report talk about incorrect or wrong emplacement, we may similarly ask on what grounds of experience in constructing harbors in deep waters can they justify taking such a categoric position. And if, on the other hand, the project was so sensitive to the point that "possible" or "probable deficiencies" pointed out by them could cause such a serious crumbling, by the same token the validity of that same project, subject to such consequences, is placed in doubt.

We think, however, that the best way to proceed is to study the actual meteomaritime conditions observed in February 1978 and in this field we are convinced we have given a sound contribution.

SOCIETA ITALIANA PER CONDOTTE D'ACQUA
Branch in Portugal - Lisbon
The Director

Dr. Eng. Giuseppe Paolella

APPENDIX B - continued:

ANSWER OF CONDOTTE TO LETTER NO. 000530 (see Appendix A, Attachment #10)

OF "BERTLIN. CONSULMAR. LUSOTECNA",

WHICH WAS DELIVERED TO CONDOTTE

BY THE "GABINETE DA AREA DE SINES"

EMPLACEMENT OF THE TOT IN THE CORES

The Specifications Form sets forth that the TOT, both the selected and the normal, may contain pieces of rock under 1 kg. which do not exceed 5% of the weight. Since, in the breakwater, an average of 250 daily trips are made by 60-ton dumpers and the total amount transported is 15,000 tons, 750 tons of material below 1 kg. could be transported. Since the specific weight of that material is 2.00 tons/m^3, 375 m^3/day of fine material could be transported.

As a matter of fact, dumpers have a capacity of 65 tons; hence, TOT below 1 kg. should be 3.22 tons per trip, corresponding to 1.60 cu. meters of fine. This quantity was never placed.

In addition, a test was carried out in the presence of your geologist on August 2 last, and the percentage of fine material reached about 2/3%, and on August 6 last, with a dumper chosen at random, we unloaded on land in the presence of your consultants, who admitted that the percentage was on sight much less than 5%.

On the other hand, under the supervision of the Works Inspector, we took the following steps in order that the material carried to the breakwater would be the best possible:

1) elimination of the sterile covering material

   This elimination is effected under the surveillance of the Inspectorate and in such a way that, as we already had the opportunity to communicate to you several times, after the cleaning access to the higher part of the quarry is not possible most of the time, not even with small drilling equipment, which even forces us to carry out explosions of rectification to give room to a surface passable for our means;

2) removal of nonsuitable material from the front of the quarry

   This material, which is in the crevices of the rock, is sent to the dumping ground. For example, in the period from August 1 to August 25 about 17% of the material was sent to the dumping ground;

3) high capacity pumps were placed on the advancement of the breakwater and the material was vigorously dampened for the purpose of eliminating the finer part;

4) by means of points of reference placed on the breakwater, the settlements of the very breakwater are measured. Up to now, in the zone initially executed the settlements are of a few centimeters.

The measurements mentioned in items 2) and 3) were taken although the contract did not envisage this to be the responsibility of the Company.

From the aforementioned, we think the material which is transported to the breakwater is fully acceptable. If, however, the consultants have an opinion to the contrary, obviously the question no longer concerns the exploration of the quarry but rather the nature and conditions of the material which constitute the quarry itself.

SECONDARY LAYERS OF THE WEST BREAKWATER COVERING

At the time of the Consultants' visit, the part of the breakwater concerning the rooting was under execution.

For this part, some uncertainties arose about the type of covering to be placed under the dolosse. As a matter of fact, drawing 15 A indicates the enrockment from 0.5 to 3 tonn., whereas drawing 173, from 3 to 6 tonn.

Therefore, it is very probable that the Consultants had seen the emplacement of those with larger dimensions, since drawing 173 was under execution.

RECESSES IN THE CENTRAL ELEMENTS OF THE DOLOSSE

Some dolosse of 15 tons have, actually, a recess in the central part of 2,35 mm. in a 1160 mm. thickness, corresponding to 0.0021, and some of 40 tons have a recess of 3 mm. in a 1589 mm. thickness, corresponding to 0.00133.

That is due to the fact that in some casings the rubber plate separating the walls to prevent the transmission of vibrations from the upper part to the lower part went back inside when the assembly took place.

This insignificant inconvenience was promptly eliminated.

CONTROL OF THE CONCRETE OF THE DOLOSSE

Under the supervision of the Inspectorate, the concrete was continuously controlled.

Before placing the dolosse, both the Works Inspector and Condotte exercise control in order that construction defects do not appear. Should some defects be detected, the causes of such defects would be examined and those dolosse would be destroyed and sent to the dumping ground.

Up to now, neither we nor the Inspectorate have noticed any defect in the manufacture of the concrete for the dolosse; therefore, we regret that the Consultants had not pointed out to us the dolosse which they did not consider suitable.

IDENTIFICATION OF DOLOSSE

Actually, reference numbers were not put on the first 15-ton dolosse, but only the date of manufacture.

We proceeded to put numbers on all of them including the first ones.

PLACEMENT OF DOLOSSE AND STONE BLOCKS

The dolosse are placed according to the instructions in the Specifications Form.

We are obviously at your disposal to assist and collaborate in the execution of all the tests with the model, in order to establish, if the consultants deem it necessary, the instructions on the geometric form which the dolosse layers must assume and on the consequent ways of emplacement.

The two layers of dollose are placed simultaneously. When the consultants came, the emplacement of the part of the dolosse from 126 to 139 was being carried out. In this zone, which is the rooting of the breakwater, only one layer of dolosse is envisaged in the project. It is probable that the consultants had thought it was intended to put another layer over it.

As to rocks, they are also placed in two simultaneous layers. However, once in a while it may happen that we just carry out one layer of rocks, when the roughness of the sea compels us to take shelter quickly. In an instance such as this, the rocks of the armoring are displaced afterwards in order to constitute two simultaneous layers.

BLOCKS FOR THE PIER OF THE CONSTRUCTION HARBOR

In agreement with the Works Inspector, we reduced the dimension of the tubes of the lower part in order to permit the placement of the suspension straps.

Lisbon, August 27, 1974

(Appendix B Continued: Replies to Official Investigation Committee and related correspondance).

QUESTION:

7. Control of the quality of the concrete and of interruptions in the concreting

ANSWER:

7. The quality of the concrete destined to dolosse was studied and tested before production was started. Afterwards, it was systematically controlled in the course of the work, together with the Inspectorate, through tests of mechanical resistence to compression at two days, at 28 days, at 90 days and at one year, with cubic samples with a 20 cm. edge. The collecting of the concrete to be submitted for testing was done in agreement with the Inspectorate, in the following ways: for each 100 cu. meters of concrete produced, a quantity of concrete sufficient to make six cubic samples was collected at the outlet of the concrete mixer and with approximately half of the unloading done; in this manner, two collections of three samples were obtained, each one destined to compression tests at two and at 28 days, respectively.

The control of the quality of the concrete is done based on the amount of its resistance to compression at 28 days of hardening. The method followed is the one rquired by the Specification Form and in "R.B.L.H.", by Decree No. 404/71 of September 23.

Through the controls of the quality of concrete destined to the dolosse produced in the period between October 1974 and November 1977, upon 2558 samples, the following data can be extracted:

- medium stress $\bar{\sigma}$ = 536 kg./sq. cm.
- characteristic stress σ_{bk} = 454 kg./sq. cm.
- medium quadratic deflection = 50
- variation coefficient = 9

With regard to interruptions in the concreting of dolosse, we have already explained in point 6 that there were practically no interruptions during the concreting of the dolosse.

QUESTION:

7.1 Did any rejection of the dolosse take place for noncompliance with the quality rules?

ANSWER:

7.1 Dolosse rejected by the Inspectorate for noncompliance with the quality rules set forth in the Specification Form were 32 out of 20,864 constructed, which is equivalent to a percentage of 0.15% of total production.

QUESTION:

7.2 Were tensile tests carried out? How often?

ANSWER:

7.2 Condotte did not carry out tensile tests.

In July 1975, at the time of the study of new compositions of concrete, bend tests were carried out at the "Laboratorio Nacional de Engenharia Civil" (National Laboratory of Civil Engineering) which indicate the results of the tensil test. These results are transcribed in annex 7.a).

QUESTION:

4. In practice how was the specification controlled, according to which 60% of the vertical legs of the dolosse must be turned to the sea? The fact is that visual observation does not show this.

ANSWER:

4. The emplacement of the dolosse by land was done with a crane equipped with a device (tak-line) which, in practice, consists of a cable

connected to the grippers and which, maneuvered manually (see photograph No. 4.b. attached), orients the emplacement in any desired direction.

We emphasize that after visits effected to shipyards in South Africa, this specification began to be considered more like a recommendation to follow, if possible, since the most important thing is the interlocking of the dolosse (see answer to question No. 7) and to achieve the best interlocking - which was strongly recommended by the specialist of South Africa - it may happen that dolosse superimposed on others do not stay with the vertical legs rigorously turned to the sea.

QUESTION:

7. No. 5 - Emplacement, of item 2.11 - Concrete blocks in protections, of No. 2 - Works of the general Technical Clauses provides that:

Before the emplacement of any blocks, the contractor will demonstrate to the satisfaction of the Inspectorate, on location, with models, that the process proposed for emplacement of the blocks underwater ensures a suitable interlocking of the protective armoring. The cost of preparation of the model and of the tests should be included in the unitary prices of the units which appear in the quantities map".

7.1. Was this demonstration made?

7.2. What were the results?

ANSWER:

7., 7.1., 7.2. In the mentioned letter No. 718 of August 27, 1974, we said textually: "We are, obviously, at your disposal to assist and collaborate in the execution of all tests with the model, in order to define, if the consultants find it necessary, the instructions on the geometric form which the layers of dolosse must assume and on the consequent forms of emplacement".

Such tests were afterwards replaced by the visit to South Africa.

With the objective, in fact, of helping the "Gabinete da Area de Sines" and the consultants in solving the problem of the emplacement of the dolosse, our Company, by letter No. 821 of September 25, 1974, which we enclose, thought it useful to invite representatives of the Inspectorate and of the Consultants on a trip to South Africa, visiting works where, up to that date, dolosse had been used (annex 7.a.), (annex 7.b.).

The Gabinete accepted the invitation and the Consultants endeavored to arrange meetings with Eng. E. Merrifield, inventor of the dolosse, and with Eng. Zwamborn of the Stellenbosch Laboratory, which had already performed numerous laboratory tests with dolosse, for the purpose of an exchange of ideas.

This visit took place from November 4 to 12, 1974, with trips to places of works in Cape Town, Gaansbaai, Port Elisabeth, East London, Durban, Richard Bay and also to the Stellenbosh Laboratory, according to the program we enclose (annex 7.c.).

The Gabinete was represented by Eng. Ismael Biscaia de Carvalho; the Consultants were represented by Eng. Silveira Ramos and Condotte by Eng. Giuseppe Paolella and Romano Agostini.

In addition to the observations made during that trip, a first emplacement of blocks (dolosse) was carried out, on location, for verification of their adequate - or not - interlocking. It was attended by the Inspectorate of the Gabinete da Area de Sines and by their Consultants. As no deficiency of interlocking was observed, the dolosse utilized for such purpose were not withdrawn, and the operations of emplacement were carried on in the same pattern.

QUESTION:

1. When answering question No. 5 of the first questionnaire, Condotte mentions that the percentage of dolosse which were broken, both in manufacture and in emplacement, may be considered normal when compared to the amount registered in other works.

It is important to indicate:

1.1 Which were the works and the respective percentages of broken dolosse in which such situations took place?

ANSWER:

1.1 As we have already had an opportunity to mention (see answer to point 9.5 of the first questionnaire), the dolosse broken during transportation were 21 out of 20,864 constructed, which is equivalent to a percentage of 0.1%.

Dolosse broken in the course of emplacement were 186 out of 20,113, which corresponds to a percentage of 0.9%.

Omar J. Lillevang, consulting engineer in Los Angeles, California, in his communication presented at The Fifteenth International Conference on Coastal Engineering, Honolulu, July 1976, made public a schedule which we enclose, through which we observe that the number of dolosse broken during the sundry phases of the Sines works may be considered normal (Annex 1.a.)

QUESTION:

1.2 What were the percentages of broken dolosse in Sines, verified after emplacement, in accordance with Condotte's assessment?

ANSWER:

1.2 In the verifications effected by divers before the storms of the winter of 1977/78, both out of water and underwater, a percentage of approximately 2% of broken dolosse was discovered in the dolosse armor unit of the entire breakwater. We consider "broken dolosse" those completely sectioned at the central part or with legs missing.

The cause for finding such a percentage of broken dolosse can be storms or the placing of the slope of the dolosse, or both phenomenons simultaneously.

QUESTION:

1.3 In January 1978, after the storms occurred during the winter months of 1977, an inspection along the west breakwater took place, by boat, of the condition of the dolosse armor unit; in that joint surveillance participated representatives of the G.A.S. Inspectorate, of G.A.S. consultants and of Condotte. In two places one or another broken dolos was found. In the zone between the progressive 828 and the progressive 870 there were also some broken or displaced dolosse and at water level, in a certain place of that zone, rocks of the enrockment underlying the dolosse armor unit could be seen, an indication that, in that place, dolosse were broken or displaced in a larger quantity.

The Inspectorate recommended to Condotte, orally, to carry out the reloading of this zone between the progressives referred to above, which Condotte did, placing via sea on January 26, 1978, 14 dolosse, on January 27, 1978, 28 dolosse, and on February 4, 1978, 14 dolosse, in a total of 56; the remaining 20 dolosse calculated by the Inspector as necessary in order to make a complete reloading, could not be placed before the storm of February 26, 1978, because of sea conditions.

As to the other two places where slight alterations had been observed, no repairing was effected for the same reason.

LIST OF ANNEXES TO QUESTION NO. 1

1.a. Schedule with percentages of fractured dolosse, taken from the communication presented by Eng. Omar J. Lillevang, on the occasion of The Fifteenth International Conference on Coastal Engineering, Honolulu, July 1976.

Translation

MINISTRY OF PUBLIC WORKS

Higher Council of Public Works and Transportation
Rua das Pedras Negras, 16 - Lisbon

COMMISSION FOR ANALYSIS OF THE ACCIDENT

AT THE SINES WEST PIER (CAMOS)

CONFIDENTIAL

Director of the Branch Office
in Portugal of Societa Italiana
Per Condotte d'Acqua

Av. Conselheiro Fernando de
Sousa, No. 19-110.

L I S B O N

Official letter No. 99/78

File No. 7-CONDOTTE

Date: July 12, 1978

Dear Sir:

1. In the answer to point 1.3 of question No. 1 of the 2nd questionnaire sent to you, it is mentioned that "between the progressive 828 and the progressive 870 there were also some broken or displaced dolosse and, at water level, in a certain place of that zone, rocks from the enrockment underlying the dolosse armor unit could be seen, an indication that, in such place, a larger quantity of dolosse were broken or displaced".

Information from another origin disclosed that this was a well localized spot, about 60 m. wide between the elevations - 2.00 m. and +4.00 m., susceptible of creating alarming suspicions to an attentive and experienced observer.

Was Condotte conscious of that or did it underestimate the occurrence, as it seems can be inferred from the transcript above?

An atmosphere of apprehension for what had happened did not evolve - an atmosphere foretelling other similar phenomenons, the traces of which began appearing along the outside slope in zones where, as a consequence of the storms at the end of February, greater destruction of the dolosse armor unit took place.

Was there not felt a need for a minute observation of all the outside slope of the breakwater, both in the emerged and the underwater parts?

2. In the cross sections surveyed by Brigade No. 1 of the Hydrographic Institute between September/October 1975 and January 1976, substantial differences are detected between the project cross sections and the executed cross sections insofar as the slope of the outside breakwater is concerned.

These differences are observed in the cross sections corresponding to the following distances to the origin (m.): 1069.09; 929.09; 949.09; 969.09; 989.09; 1009.09; and 1029.09.

Unfortunately, the surveys performed by the divers of the H.I. were not exhaustive, but, judging from the sampling which covers a limited extension of the breakwater, doubt may arise as to the quality of execution, even taking into consideration allowances for error characteristic of a maritime work of such magnitude.

Did Condotte take cognizance of these deficiencies? In which terms? Was it notified to correct them? Did it fulfill what was imposed on it in those contingent notifications?

I look forward to your reply to these questions at your ealiest convenience.

Very truly yours,

THE PRESIDENT

(signed) Mario Fernandes

Mario Pinto Alves Fernandes

(Inspector General of Public Works and Transportation)

Translation

SOCIETA ITALIANA PER CONDOTTE D'ACQUA

No. 427/78

Lisbon, July 18, 1978

Eng. MARIO PINTO ALVES FERNANDES
Inspector General of Public Works and Transportation
Chairman of C.A.M.O.S.
Higher Council of Public Works and Transportation
Rua das Pedras Negras 16
LISBON

Dear Inspector General:

We answer in the following terms to the Commission's esteemed letter No. 99/78 of July 12, 1978.

I - Answer to point 1.

1. In point 1. of your above mentioned letter only one point is transcribed of the answer that Condotte gave in its letter No. 377/78 of June 7, 1978, to point 1.3. of question No. 1 of the 2nd Questionnaire sent by C.A.M.O.S.

   The complete reading of our answer to that point 1.3. sheds light at once upon some of the questions not put forward.

   Thus, Condotte started out saying: "In January 1978, after the storms occurred during the winter months of 1977, a verification, by boat, was carried out along the west breakwater of the condition of the dolosse armor unit; in that joint verification there participated representatives of the Inspectorate of the Sines Area Bureau, of the G.A.S. Consultants, and of Condotte".

   Therefore, when C.A.M.O.S. now asks whether there "was not felt a need for a minute observation of the entire outside slope of the breakwater, both in the emersed as well as in the underwater parts", the answer is:

a) A minute observation of the entire outside slope of the breakwater was made in the emersed zone, and it was precisely that minute observation which permitted the discovery of the existence between progressive 828 and progressive 870 of some broken or displaced dolosse and, at water level, some stone from the enrockment underlying the dolosse armor unit could be seen, an indication that in such spot dolosse were broken or displaced in a larger quantity.

b) A similar observation in the underwater zone has not taken place because it could be carried out only by divers and the weather conditions did not permit their work.

2. The "information of other origin" referred to in your letter is unknown to us. As to the "spot" observed, we have already said how it was discovered and if that "other origin" knew more about the subject than the Inspectorate and Condotte, it did not appear in due time to report on its information.

With regard to the "alarming suspicions" that the spot might create to an attentive and experienced observer, it is obvious that a "suspicion" - or more than suspicion, a certainty - was created that such zone needed to be reloaded. And because of that, as is also stated in our reply: "the Inspectorate recommended to Condotte, orally, to carry out the reloading of that zone between the mentioned progressives, which Condotte did, placing by sea on January 26, 1978, 14 dolosse, on January 27, 2978, 28 dolosse and on February 4, 1978, 14 dolosse, in a total of 56; the remaining 20 dolosse calculated by the Inspector for the reloading to be complete could not be placed before the storm of February 26, 1978 due to the conditions of the sea".

3. That the arrangements made by the Inspectorate and by Condotte regarding that zone were adequate and that, therefore, the "alarming suspicions" were groundless insofar as the same zone was concerned is demonstrated by the fact that between progressives 828 and 870 the effects of the February storms were not felt.

4. No climate of apprehension was created, nor could be created, in view of what happened between progressives 828 and 870, in addition to what was stated in No. 2.

On the one hand and as was also already stated, a minute verification had just been carried out along the entire breakwater and no similar pheonomenon was found in any other location.

On the other hand, the limitation of the zone which was hit (52 meters), the circumstance that some storms of a certain importance had already occurred, the fact that it was part of the very conception of the dolosse armor unit that, with the passing of the years and the occurrence of storms, elements settle down and reinforce their interlocking, and, finally, the consideration that the trunk at issue had been built in August 1975 and, therefore, if there was any kind of defect it would have manifested itself long ago, all contributed to the thought that the phenomenon was not to become generalized but rather to be locally remedied.

5. You refer to "similar phenomenon, the traces of which began appearing along the outside slope in zones where, in consequence of the storms at the end of February, there were the greatest destructions of the dolosse armor unit".

   We do not know what those traces had been and by what means they were detected and evaluated as "traces". We ask you the favor of indicating them to us so that we can consider them.

   The examinations which Condotte and the Inspectorate carried out and their results are already recorded in the answers given to that Commission.

II - <u>Answer to point No. 2</u>.

1. The survey of Brigade No. 1 of the Hydrographic Institute carried out between September/October 1975 and January 1976 were never brought to the cognizance of Condotte; therefore, we are not in a position to issue any opinion on it.

2. We cannot fail but once again to express our surprise and regret in seeing that the Area of Sines Bureau had in its possession documents which could, in principle, be of interest to the execution of the work and which it did not bring to the cognizance of the contractor.

The only plausible explanation for that omission is that the G.A.S. considered the data gathered and not exhibited to the contractor to be normal and without importance to the execution of the work.

But if that is the case, it also seems that, upon now presenting the data to C.A.M.O.S. or to any other entity, it should have explained the reasons for the omission which occurred.

3. It happens, however, that your letter says "one cannot fail but to raise the doubt as to the quality of execution, even taking into consideration allowances for error characteristic of a maritime work of such magnitude".

Condotte cannot let such a doubt subsist, not even for a moment, and thus, while waiting for that Commission - if it so sees fit - to furnish to Condotte the surveys of the Brigade referred to above, comes as of now to state the following regarding the seven cross sections mentioned in the said letter.

a) Condotte has in its possession and transmits herewith cross sections in places very close to the seven which were mentioned:

Distance to the origin	Date
925	Sept. 18, 1975
945	Sept. 22, 1975
965	Oct. 13, 1975
985	Nov. 8, 1975
1005	Nov. 21, 1975
1025	Nov. 3, 1975
1065	Dec. 17, 1975

b) These cross sections were surveyed together by Condotte and the Inspectorate.

c) The surveys were carried out with a manual depth gauge, which, for a work of this kind, constitutes the procedure with more accurate results.

We avail ourselves of this opportunity to present to you, Sir, our best regards.

Very truly yours,

SOCIETA ITALIANA PER CONDOTTE D'ACQUA

Branch Office in Portugal - Lisbon

The Director

Dr. Eng. Giuseppe Paolella

LIST OF ANNEXES TO LETTER NO. 427/78 OF JULY 18, 1978 *

2.a. - Cross sections of construction of the outside slope of the West Breakwater at the following distances to the origin: 925, 945, 965, 985, 1005, 1025, 1065.

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* These were not available to the Panel.

APPENDIX C

Report of the Portuguese Official Investigating Committee

MINISTÉRIO DAS FINANÇAS

E DO -- PLANO

GABINETE DE INFORMAÇAO E RELAÇÕES PÚBLICAS

NOTA

Por despachos conjuntos, de 28 de Fevereiro de 1978 e de 1978 e de 20 de Abril do mesmo ano foi criada, pelo Governo, uma Comissão de Análise do Acidente do Molhe Oeste de Sines, de tinada e efectuar um cuidadoso esame do acontecido, com averiguação de eventuais responsabilidades.

Foi essa Comissão integrada pelos seguintes membros:

- Engº. Mário Fernandex, Inspector Geral do Conselho Superior de Obras Públicas e Transportes, que presidiu;
- Engº. Armando Campos e Matos, professor catedrático de Pontes e Estruturas Especiais da Faculdade de Engenharia da Universidade do Porto;
- Engº. Eurico Carrondo Tomé, Subdirector Geral de Portos.

Após laboriosa e exaustiva actividade, deu a Comissão por findos os seus trabalhos, tendo entregue ao Governo o respectivo relatório, cujas conclusões são agora comunicadas à opinião pública. Apenas se alterous a numeração dos capítulos (que se integrava como é evidente no sistema geral do relatório) e se desenvolveram em nota de pé de págnia, para esclarecimento, algumas das siglas utilizadas.

1. Conclusões

1.1. Na generalidade

1.1.1. O molhe oeste de Sines constitui uma obra de engenharia de hidraúlica marítima, de características invulgares, em particular pelo que respeita à profundidade das águas em que foi implantado.

Pode dizer-se, como já se tem dito, que se trata duma das maiores obras do mundo do seu género, e que o respectivo projector punha problemas que não permitiam a sua condução e resolução em termos análogos aos de muitas obras correntes que por esse mundo se vão construindo.

O volume dos investimentos envolvidos na empreitada nº 8/72, adjudicada à Condotte (1), e de que o molhe oeste absorve cerca de 90% - perto de três melhões de contos, à partida, a custos de 1973 - atingiu em fins de 1977 cerca de seis milhões de contos (5 910 037 c), tendo o GAS (2) previsto pagamentos em 1978, no montante de 1 217 332 contos e em 1979 de 554 867 c ou seja, no total, o custo das obtras incluídas na empreitada sobe para perto de oito milhões de contos (7 682 236 c), que não inclui qualquer verba cobrindo os extragos provocados pelo acidente. Refira-se, ainda, que as importâncias indicadas não estão referidas a uma mesma base do valor da moeda.

A simples menção destes números, põe em evidência a grandeza da obra, grandeza que, se outras razões não houvesse, bastaria para provocar uma profunda relexão da parte daqueles a quem foi confiada a enorme responsabilidade de conduzir todo o grandioso empreendimento de Sines, no qual as obras portuárias se inserem.

Procurou-se ao longo de nove meses de investigações, cumprir o mandato que à Comissao de inquérito foi outorgado pelo Governo, tarefa que não foi nem fácil, nem pode ser rápida.

De facto, quer pela natureza da própria obra, da qual uma parte muito importante dos seus elementos se encontram submersos, a profundiades que vão de (-30,00) a (-50,00), tornando difíceis, senão impossíveis, algumas averiguacões, quer pela dependência da comissão de inquérito, no que se refere à prontidão e conteúdo qualitativo das respostas, em relação a numerosas entidades que forma inquiridas, produzirma pareceres, elaboraram informações e realizaram ensaios, só por imponderação ou desconhecimento das dificuldades, poderia esperar-se que a comissão de inquérito concluisse o seu trabalho em tempo mais curto do que o que efectivamente gastou.

A comissão tem a consciência de que, com as dificuldades referidas e as limitações decorrentes da afectação dos seus membros a tarefas absorventes dentro das suas funções normais, e da sua separação física, produziu o seu trabalho com toda a isenção e em tempo útil.

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(1) Societá Italiana Per Condotte d'Acgua.

(2) Gabinete da Área de Sines.

1.1.2. Um acidente do tipo do que sofreu o molhe oeste de Sines, pode ser devido a várias causas, conforme a comissaõ de inquérito já teve ocasião de dizer no relatório intercalar de 31 de Maio de 1978, e nesse pressuposto conduziu as pesquisas tentando perseguir várias pistas, em três grandes áreas: o projecto, a execução das obras, e a suq condução e fiscalização.

O desenvolvimento das averiguações acabou por fazer ressaltar uma causa de natureza técnica como preponderante - a fragilidade dos dolos face às forças que os solicitam, embora enquadrada com outras no âmbito mais geral da elaboração e organizaçao do projecto, por um lado, e da capacidade e organização da entidade a quem coube conduzir todo o processo, o Gabinete da Área de Sines, por outro.

Nas "conclusões na especialidade" põem-se em evidência os aspectos averiguados ao longo da investigação e que estão devidamente tratados no presente relatório, com pormenor que se julga adequado.

1.1.3. Em resumo, o acidente sofrido pelo molhe oeste de Sines, resultou de causas técnicas, relacionadas com a elaboração do próprio projecto, e poderia ter sido eventualmente evitado, se causas institucionais, respeitantes à organização e à capacidade do orgão condutor do projecto, tivessem sido eliminadas.

Quanto às primerira - causas técnicas - referem-se uma deficiente avaliação de solicitações em grande parte motivada por carência de dados estatísticos, a adopção duma estrutura com recurso a blocos artificiais de protecção, os dolos, muito controversos, sobre os quais não havia conceitos definitivos e nenhuma experiência em Portugal, uma geometria do perfil resistente muito académica e de difícil execução, económica mas sem reserva de resistência (talude exterior a 1,5:1), e uma pouco apurada apreciação dos ensaios em modelo reduzido que foram observados com "olhos muito hidraúlicos" e não sofreram a contraprova de ensaios noutro laboratório conceituado.

Quanto às segunda, que designámos por causas institucionais, queremos aludir à falta de capacidade técnica e organizativa do GAS para conduzir todo um processor que vai desde a fase preliminar dos estudos até à execução da obra. Nem a organização foi adequada, nem as pessoas, quer as que intervieram na fase de elaboração do projecto quer as que, depois, participaram na condução fiscalização das obras, possuiam a bagagem técnica - conhecimentos teóricos e experiência vivida - indispensável para obra de tal natureza e tamanha envergadura.

Esta incapacidade do GAS, conjugada com uma certa displicência, veio ao de cima, com clareza, nas próprias relações com a comissão de inquérito, facto para o qual teve esta oportunidade de chamar a atenção do Governo em devido tempo.

Finalmente, não é possível à comissão de inquérito tirar conclusões definitivas em termos de, com absoluta tranquilidade de consciência, poder afirmar que deficiências de execução da obra se poderão ter adicionado às outras causas mencionadas, contribuindo para o acidente. Ê lícito supor a ocorrência de deficiências de construção, mas não é possível prová-las.

1.2. Na especialidade

Ao longo do relatório, em particular no nº 4-Apreciação, vem analisados com o pormenor possível, face aos dados disponíveis, os aspectos relacionados com o projecto, com a construção e com a condução e fiscalização da obra.

A eles nos referimos, a seguir, em síntese.

1.2.1. Quanto ao projecto

Como norma geral o projecto deve ser elaborado recorrendo a toda a informação de base possível, por forma que os parâmetros que a condicionam sejan perfeitamente definidos e dignos de fé.

O tempo que se gasta no projecto poupa-se na execução da obra. O projecto consumirá o tempo que for necessario para desfazer todas as duvidas, por forma que a construção se possa fazer sem hesitações e sem soluções de descontinuidade.

Apontam-se a seguir os aspectos que, no entender da domi sã or de inquérito, se consideram como tendo tido influência no aci dente de 26 de Fevereiro de 1978.

1.2.1.1. O clima de agitação marítima não terá sido correctamente definido, principalmente em resultado da carência de dados estatísticos, em número e em qualidade dignos de confiança.

Baseram-se fundamentalmente, em dados colhidos na Figueira da Foz, durante 7 anos, e registos de dois invernos duma bóiondógrafo posicionada ao large de Sines.

1.2.1.2. Em consequência, as alturas da onda significativa nas proximidades da obra terão sido deficientementes avaliadas, com todas as probabilidades de o terem sido por defeito.

Por outro lado, e pelo que respeita aos períodos de retorno, também se pode suspeitar duma previsão estatística pouco correcta, pois que a altura da onda significativa, H_s = 8,5m corresponde, segundo os critérios de projecto, a um período de retorno da ordem dos 10 anos, e segundo a análise do INMG (3), esse período é estimado em cerca de 5 anos.

1.2.1.3. O projecto não contém planos de ondulação, como é norma em projectos desta natureza.

Remete-se para um estudo de agitação marítima no (então) futuro porto de Sines, realizado para o GAS, por três especialistas do LNEC (4), em regime de profissão liberal, que é nitidamente insuficiente e não permite tirar conclusões definitivas.

Assim, os problemas da concentração de energia, resultantes da ocorrência de fenómenos de refracção, não foram averiguados embora milite a favor dos projectistas a falta de elementos relativos á batimetria dos fundos, que deviam ter exigido, e a circunstância de, até então, não haver conhecimento da existência na costa portuguesa de ondas de grande período, únicas para as quais o fenómeno é relevante.

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(3) Instituto Nacional de Meterologia e Geofísica.

(4) Laboratório Nacional de Engenharia Civil.

Não obstante, a importância da obra e a sua localização tornavam perfeitamente indispensável que tal estudo tivesse sido incluído entre as peças do projecto.

1.2.1.4. A geometria do perfil resistente do quabre-mar é muito académica e arriscada.

"Académica" porque os seus refinamentos geométricos são imcompatíveis com os processos construtivos próprios das obras marítimas e practicamente impossíveis de controlar. Illustra-se a afirmação com o exemplo da berma do prisma de apoio do manto de dolos que, com 3,00 m de larguar, à cota (-15,00), e constituida por blocos naturais de 16 a 20 T, nenhum constructor conseguiria executar, ainda que se argumente com o facto, conhecido à posteriori, de que os ensaios em modelo reduzido, conduzidos em vários laboratórios, revelaram que a berma, com tal configuração e dimensões, não era determinante no processo de ruína.

"Arriscada" porque, com a preocupação da economia de primeiro investimento - os volumes da obra crescem substancialmente com o suavizar dox taludes - se adoptou um talude do manto de protecção a 1,5:1 que a prodencia e o bom senso desconselharia numa obra deste vulto, de mais com as incertezas e insuficiências de informação que caracterizavam o clima de agitação marítima, e portanto, a definição das solicitações.

1.2.1.5. Optou-se por um tipo de bloco artificial de protecção, o dolo, sem que tal decisão fosse apoiada em estudos comparativos com outros blocos artificiais hã muito consagrados, estudos esses que haveriam de prosseguir todas as fases dum projecto deste tipo, incluíndo os ensaios em modelo reduzido.

1.2.1.6. Adoptou-se, assim, na zona mais importante do molhe, um tipo de bloco extremamente controverso com virtudes muito enaltecidas e defeitos muito pouco conhecidos, estes agravados pela extrapolação de dimensões, e de que não havia qulquer experiência em Portugal.

Foi uma experiência ousada para um país pequeno, embora se compreenda que, historicamente, o dolo era a grande "vedeta" das obras maritímas dos princípios dos anos 70, 3 se deva, por imperativo de justiça, elucidar que só a partir de então, começaram a surgir, aqui e além, as críticas a um bloco de protecção de quebra-mares que se apresentara como quase milagroso, quando foi lançado.

Deve dizer-se também que é muito provável que, qualquer outro projectista, optasse pelo mesmo tipo de bloco que era, na altura, repete-se, a grande moda.

A falta de resistência estrutural dos dolos face às solicitações a que estão submetidos por via de acção dinâmica das vagas, explica a sua fractura individual, a rotura do imbricamento e, portanto, a perda da estabilidade global do manto dos dolos que, reduzidos a peças mais pequenas, se chocam uns contra os outros, provocando mais fracturas e a ruína total do manto.

O mecanismo teórico da rotura é o que resulta da aplicaçao de esforços dificilmente quantificáveis, no estado actual dos conhecimentos, que provocam tensões de tracção no betão superior às que este material pode suportar.

Os ensaios em modelo reduzido realizados no Canadá em que, nos dolos, foi simulada a resistencia do betão à tracção, demonstraram que a explicação para o acidente que os próprios ensaios evidenciaram, foi confirmada pela forma que, no modelo, tomou o perfil após a ruína, a qual era inteiramente análoga à do protótipo após o acidente.

Outrossim os ensaios realizados no estaleiro da obra, sob a orientação do LNEC, utilizando o próprio protótipo dos dolos, confirmaram a fragilidade destes blocos para acções estáticas da ordem de grandeza do próprio peso e, por maioria de razão, para acções dinâmicas, como as que terão de sofrer em serviço, que são, de certeza, de efeitos muito mais nefastos.

1.2.1.7. Os ensaios, em modelo reduzido, realizados no LNEC foram apreciados muito do "ângulo hidraúlico" e nada d o "ângulo estrutural", apesar do alerta lançado em tempo oportuno.

1.2.1,8. Embora nada tenha a ver com a acidente, a projecto não considerou os problemas de conservação do manto de dolos durante a vida da obra. De facto, há uma zona do mesmo manto, que não é acessível por qualquer equipamento corrente quer a partir da plataforma do quebra-mar, quer a partir do mar usando equipamento flutante. Aliás, subsisten dúvidas quanto à possibilidade de reparação do manto de dolos, em termos de ficar assegurado o imbricamento, essencial para a seu comportamento.

1.2.2. Quanto a construcao

Apesar de todos os esforços que desenvolveu para apurare provar - a existência de defeitos de construção, e de que dá conta o capítulo respectivo do presente relatório, à commissão de inquérito não foi possível demonstrar, sem margem para dúvidas, a existência de factos, imputáveis à construção, susceptíveis, de, conjuntamente com outras causas, terem contribuido para a ruína do quebra-mar.

Dum modo geral existe um consenso, a que não se associou o agrupamento projectista BCL (5), quanto à qualidade, em termos globais, da obra executada.

A comissão de inquérito, ainda que admita a possibilidade da ocorrência de defeitos de construção para além dos que decorrem da dificuldade de execução do perfil tal como foi concebido - seria estultícia negá-la - reitera a sua posição de se considerar impossibilitada de poder atribuir a defeitos de construção algumas responsabilidades no colapso da estratura, por manifesta fal ta de provas.

1.2.3. Quanto à condução e fiscalização da obra

Não tem a comissão de inquérito dúvidas de que a condução e a fiscalização da obra não estiveram à altura da sua importância e dos problemas que punha.

Ê certo que se no GAS, e na fase do projecto, tivesse havido técnicos e/ou dirigentes dotados de preparação adequada, em domínio tão especializado, talvez se tivesse perfilhado outra sorução.

Só nessa medida é que o organismo, dando uma prova positiva da sua capacidade, poderia ter contribuido, a montante, para evitar ou, pelo menos, minimizar as consequências do temporal de 28 de Fevereiro de 1978.

Apesar disso, julga-se de salientar que:

(5) Bertlin - Consulmar - Lusotecna.

1.2.3.1. O GAS não foi estruturado para conduzir um processo e fiscalizar uma obra de tanta envergadura.

1.2.3.2. Não se põe em dúvida a honestidade das pessoas e a sua vontade de acertar, mas põe-se em dúvida a sua competencia e aptidão para conduzir e orientar o projecto e fescalizar a obra.

Mostrou-se que, com uma unica excepção, nenhum dos técnicos tinha qualquer experiência de obras marítimas, quer a nível de preparação teorica quer a nível de execucão.

1.2.3.3. Nas fases de elaboração do projecto e de apreciação das propostas, a inexistencia de verdadeiros especialistas, um que fosse, impediu que documentos importantes como o relatório do Danish Hydraulic Institute e o manual encomendado pelo próprio GAS à Interplan Corporation, tivessem sido objecto de ponderada reflexão e de diálogo com o agrupamento projectista e com o LNEC.

Não se percebe meso para que é que o último serviu. E, por certo, não foi elaborado a título gracioso...

1.2.3.4. A falta de apreciação do projecto de quebra-mar por entidade técnica independente dos projectistas e do GAS, e acima de ambos, foi uma lacuna importante.

1.2.3.5. A falta de organização da fiscalização, a inexperiência dos seus agentes, com uma única excepção como se disse, mas mesmo essa irregular e menos consistente, se é certo que poderá não ter nada a ver com a explicação do acidente, impediu, porém, que se conhecessem, se de facto existiram, defeitos seriós de construção, que elementos dignos de fé, organizados e sistematizados, pudessem ser postos à disposição da comissão de inquérito, enfim impediu, de forma quase definitiva, que se conheça com algum rigor a história da obra.

1.2.3.6. O GAS cuidou mal da sua imagem externa. Não se aceita nem se compreende que um dos seus técnicos passasse para o serviço do empreiteiro e voltasse para o GAS, como se este não fosse o dono da obra e aquele o seu executante. Não basta que a mulher de César seja honesta, é preciso tambem que o pareça...

2. Responsabilidades

2.1. Considerações gerais

O mandato conferido à comissão pelo despacho ministerial de 28 de Fevereiro, explicita a "averiguação de eventuais responsabilidades". Trate-se do ponto mais delicado da tarefa da CAMO sobre o qual e antes de mais, importa tecer algumas consideraçõn.

Assim, e em primeiro lugar, é convicção da comissão de quérito que o agrupamento projectista - qualquer projectista - é cara a sua tarefa empenhadamente, pondo nela todas as suas capacidades, todo a seu saber.

Por uma razão, ou por outra - a urgência, a pressão de quem encomenda o projecto, a má definição das relações entre os intervenientes no processo, etc. - o projectista pode ser insensivtimente levado a adoptar soluções, a tomar decisões, não completamente amadurecidas, mas influenciadas pela necessidade de ser objectivo, dinâmico de responder ao que lhe pedem.

Mas não está, pensa-se, de má consciência, não está a ser negligente. Ê o caso do cirurgião a quem o paciente morre nas mãos não obstante todos os seus esforços, todo o investimento da sua ocupetência e do seu saber, para o solvar. Qua responsabilidades é que se ilhe pedem, ou melhor, que a sociedade ilhe pede?

Por outro lado, projectar é correr rescos. Por vezes, enormes. Há que ter este aspecto em consideração.

Quem executa as obras tambem corre riscos, os riscos inerentes à execução dos diferentes elementos da própria obra e às pessoas que nela trabalham. É lícito supor que, em grande parte dos casos, e sobretudo nas empresas grands, ao objectivo do lucro - e a assunção de riscos é um dos factores que legitima o lucro - se associa o da defesa da idoneidade da própria empresa, a sua imagem de marca. Não se crê que uma empresa de características internacionais, enverede deliberadamente pela fraude.

A função fiscalização é extremamente importante mas tambem aqui há limitações para a sua própria actuação, desde a falta de meios, à inconveniente organização da sua extrutura e à falta de capacidade técnica interna, por deficiente ou insuficiente compresão de quem superintende.

Em resumo, a comissão de inquérito é de parecer que, neste domínio das responsabilidades a imputar, numa análise racional, e desapaixonada, todas as circumstâncias atenuantes deverão ser ponderadas, na convicção em que está de que todos procuraram dar o seu melhor.

2.2. Imputação de responsabilidades

A comissão de inquérito reitera a sua intenção de não imputar responsabilidades a pessoas, mas simplesmente a entidades jã que todos os trabalhos forma, na generalidade, realizados por equipas, e a individualização dessa imputação, além de difícil, poderia conduzir a resultados injustos.

2.2.1. Agrupamento projectista BCL

Vimos que (1.2.1.) o agrupamento projectista elaborou um projecto com algumas deficiências, umas mais importantes do que outras.

A mais importante de todas e que foi a causa da ruina do quebra-mar, não é propriamente um erro de projecto mas de concepção, com a aceitação passiva dum elemento da obra - o dolo - em que, como era corrente na época, se acreditava quase sem reservas mas que sofre de muitos inconvenientes que, como se referiu, recomendam o uso da máxima prudência na sua aplicação.

Um outro aspecto em que a responsabilidade do grupo BCL é menos diluida é o que se refere à avaliação do clima de agitação marítima. Perante a insuficiencia de dados não foram adoptados os coeficientes de segurança que a prodência aconselha.

Estamos certos que a reformulação do projecto irá conduzir a alturas de onda significativas e a períodos de retorno muito diferentes dos considerados.

A não consideração do problema da concentração de energia em resultado dos fenómenos de refracção cuja ocorrência ficou provada nos estudos em modelo matematico do LCHF (6), do NRC (&) (Canadá) e do LNEC, e em modelo físico do primeiro, poderia ter constituido a causa preponderante do acidente se, segundo os ensaios de Canadá, o manto de dolos não tivesse ruído para uma altura de onda significativa muito inferior à onda do projecto (11,0 m).

(6) Laboratorie Central d'Hydraulique de France.

(7) National Reserach Council.

Se isso não tivesse acontecido a comissão de inquérito teria sido conduzida naturalmente a responsabilizar o agrupamento projectista pelo que aconteceu.

2.2.2. L.N.E.C.

Não obstante o prestígio da instituição, a actuação do Laboratório Nacional de Engenharia Civil não está isenta de críticas.

Com efeite, faltou na altura da observação dos ensaios em modelo reduzido, a supervisão de alguém com experiência capaz de reflectir sobre a chamada de atenção do Eng^{o} Vera Cruz, quando fez o ensaio com dolos de 30 T 3 alertou para os problemas que a sua resistência mecânica podia levantar.

Foi pena que tudo se tivesse confinado à "visão hidraúlica" dos ensaios, que se olvidasse a apreensão do Eng^{o} Vera Cruz, e que não tivesse sido solicitada, como se impunha, a colaboração dos especialistas do sector de estruturas do LNEC.

Há, portanto, no processo uma certa responsabilidade do LNEC.

Foi pena, ainda, que os Relatórios I e II respeitantes aos ensaios realizados em 1972/73, so viessem a ser publicados - mas não divulgados - com data de Fevereiro e Janeiro de 1977, respectivamente.

Foi pena, finalmente, que quem conduziu os ensaios, de que dão conta os relatórios I e II, de Fevereiro e Janeiro de 1977, acima mencionados, trabalhasse simultâneamente para o LNEC e para o BCL, o que, se não tem implicações no plano moral, poderá tê-las tido no plano dos compromissos intelectuais e técnicos, e no seguimento do processo.

2.2.3. G.A.S.

A comissão de inquérito julga ter dado do GAS a imagem que lhe corresponde.

A sua responsabilidade reporta-se à fase da elaboração do projecto e à fiscalização da obra.

No primeiro caso, poderia ter eventualmente contribuido para evitar que se tivesse adoptado uma solução que veio a revelar-se inseguar.

No segundo, pela condução da fiscalização em termos que não foram correctos e que teriam habilitado a comissão de inquérito a conhecer a história da obra, com toda a fidelidade, e a permitir detector, com clareza e fundamento, os erros de construção que, porventura, tivessem sido cometidos, aos quais pudesse ser imputada responsabilidade no acidente, erros que ninguém é capaz de provar, nem se julga que haja meio de o fazer.

Esta a grande responsabilidade do GAS, cuja acção e orientação é, no fim e no cabo, da responsabilidade de quem, no período mais relevante do empreendimento, exerceu as funções de seu director.

2.2.4. Societá Italian Per Condotte d'Acqua

As deficiências de execução que ihe foram apontadas não puderam ser provadas, nem por quem as denunciou, nem pela comissão de inquérito.

O projecto da obra não é da autoria do empreiteiro e a utilização dos dolos não foi por ele sugerida.

Aliás, demonstrou-se que o fabrico dos blocos seguiu as normas adequadas e o betão utilizaco atingiu qualidade e resistência que não é fácil ultrapassar.

Os métodos de execução, em particular a colocação dos dolos, foram aprovados pelo dono da obra, com a adjudicação. A contestação dos projectistas pelo que se refere à colocação com equipamento flutuante, não foi, na altura, considerada consistente pelo GAS que não vetou, portanto, a utilização do mesmo. Deve referir-se que durante a reparação provisória foram usados sem reparos, os mesmos metodos e equipamento que antes do acidente.

Sobre este problema da responsabilidade por erros de execução, existe legislação. Ê o Artº, do D.L. nº 48 871, de 19 de Fevereiro de 1969, que a seguir se transcreve:

"Artº 34º (Responsabilidades por erros de execução)

1. O empreiteiro é responsável por todas as deficiências e erros relativos à execução dos trabalhos, ou à qualidade, forma e dimensões dos materiais aplicados, quer quando o projecto não fixe as normas a observar, quer quando sejam diferentes dos aprovados.

2. A responsabilidade do empreiteiro cessa quando os erros e vícios de execução hajam resultado da obediência a ordens ou instruções escritas transmitidas pelo fiscal da obra ou que tenham obtido a concordância expressa deste".

Em conclusao, não é possível imputar quaisquer responsabilidades ao empreiteiro pelo acidente, derivadas de defeitos ou erros de execuçao.

2.3. Consequências da imputação de responsabilidades

O assessor juridico da comissão de inquérito, Dr. Deodato Nuno de Azevedo Coutinho, elaborou, com data de 22 de Janeiro de 1979, o parecer relativo a responsabilidades e de que se transcreve a parte final:

"Em conclusão:

A matéria carreada para or processo ---

a) Não aponta para que os autores do projecto ou a empreiteira devam ser responsabilizados pelo danos causados pelas vagas do mar no molhe oeste do porto de Sines, em 26 de Fevereiro de 1978;

b) Denuncia a existência de factos, imputáveis a dirigentes e técnicos do GAS, que se situam indiciariamente no domínio do ilícito disciplinar, mas que não são, em si, fonte de responsabilidade civil.

Este é o meu parecer".

3. Conclusão final

Ê parecer da comissão de inquérito que a causa determinante do acidente ocorrido em 26 de Fevereiro de 1978, no molhe oeste de Sines, foi a fragilidade estrutural dos blocos artificials utilizados no manto exterior de protecção do molhe (os dolos).

Entende tambem a comissão deixar aqui registado que se provou a possibilidade de ocorrência, no local da obra, de fenómenos de concentração de energia das ondas, que poderiam, por si só me mesmo com comportamento extrutural satisfactório dos dolos, ter causado a ruína da estrutura, embora para alturas de onda superior àquelas para a quais se descadeou o processo de fractura dos dolos.

Nestas condições, e de acordo com o que ficou dito ao longo do relatório, dá o seu acordo às conclusões do paracer do assessor jurídico mencionado no número anterior.

Lisboa, 2 de Maio de 1979

APPENDIX D

Evaluation of Wave Conditions at Sines:

1. Wave Conditions off Sines, 22 February to
   2 March 78, by Lawrence Draper

2. Port de Sines, Caracteristiques au Large de
   la Tempete tu 26 Feurier 1978, by
   Laboratoire Central D'Hydraulique de France

Evaluation of Wave Conditions at Sines

22 February to 2 March, 1979

Prepared by Laurence Draper

July 1978

WAVE CONDITIONS OFF SINES, PORTUGAL, 22 FEB. 78 - 02 MAR. 78

LAURENCE DRAPER

Wave conditions off Sines have been estimated from meteorological conditions, and visual observations of wave height have been compiled and analyzed. The UK Meteorological Office was asked to investigate the wind-fields over an area lying between 5° and 40°W and between 30° and 50°N. From the results of their analysis the wave conditions have been computed for every twelve hours (noon and midnight) with the exception of 0001 hr 23 February, for which the windfield data are not available.

The results are expressed in two formats: one is a wave-field map for the area considered for each twelve-hourly interval; the other is a plot of significant wave height against time for the sea area just west of Sines (Figure B-1).

The wave hindcasts have been made using the IOS oceanic wave prediction graphs (Darbyshire and Draper, 1963); and the swell component has been derived based on an analysis of swell decay undertaken by the London Weather Centre (Sykes 1975). It is pertinent here to point out that as the wind-field analysis does not cover the area west of 40°W, no swell from storms outside this area is taken into account. This is probably unimportant at times of high predicted waves; for example, even an additional 4-m high swell arriving at the time of the highest predicted wave conditions of 9.1 m would still yield a combined height of less than 10 m. However, the effect of swell arriving at times of lower waves could result in the actual wave conditions being slightly higher than those predicted. An inspection of the North Atlantic weather maps shows no significant source of swell, external to the area considered, capable of affecting the site at the times when high waves were predicted, and very little at other times. However, as it is necessary to undertake a detailed meteorological study to identify low to moderate wind-fields accurately, this simple type of assessment cannot be relied on to guarantee the absence of swell from elsewhere.

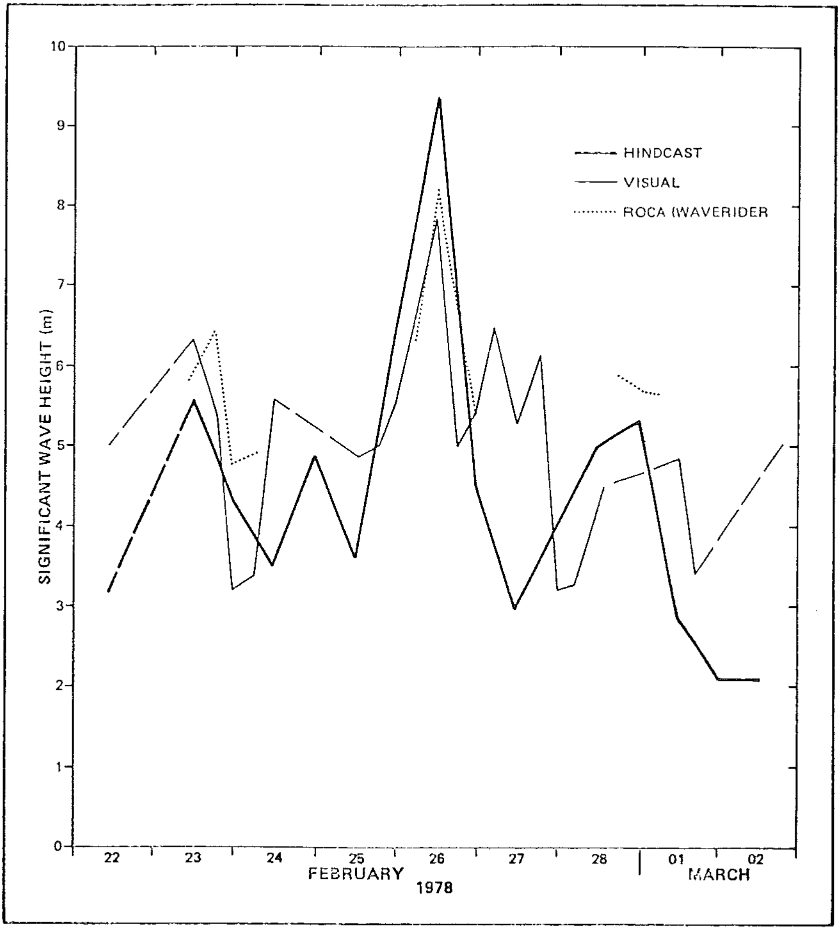


FIGURE D-1

COMPARISON OF WAVES HINDCASTED BY DRAPER
WITH THE VISUAL SHIPBOARD OBSERVATIONS AND THE
MEASURED WAVES AT CABO DA ROCA

The UK Meteorological Office has listed visual estimates of wave conditions. The heights of those within about 3^{o} of longitude of the Portuguese coast off Sines have been abstracted and an average taken; they have also been plotted on Figure 1. Whilst visual observations en masse are useful in comparing one area with another, individual observations are of little value and the average of a small number of them should be treated with considerable caution. There are four occasions when there are either one or no visual observations nearby (usually at night) so adjacent figures are joined by a dotted line. However, these observations do seem to follow the broad pattern of the predictions from wind and add a little confidence to the predictions. In addition, the few available significant heights from the waverider at Roca are plotted on Figure 1. These again, although intermittent, are compatible with the predictions and visual observations, although the highest measured value is a little lower than the highest predicted value.

The numerical values of significant wave height are listed in table 1, together with the predicted zero-crossing period.

Rarity of the Event

A simple analysis by the UK Meteorological Office suggests the following hourly-mean wind speed for the Portuguese coast off Sines.

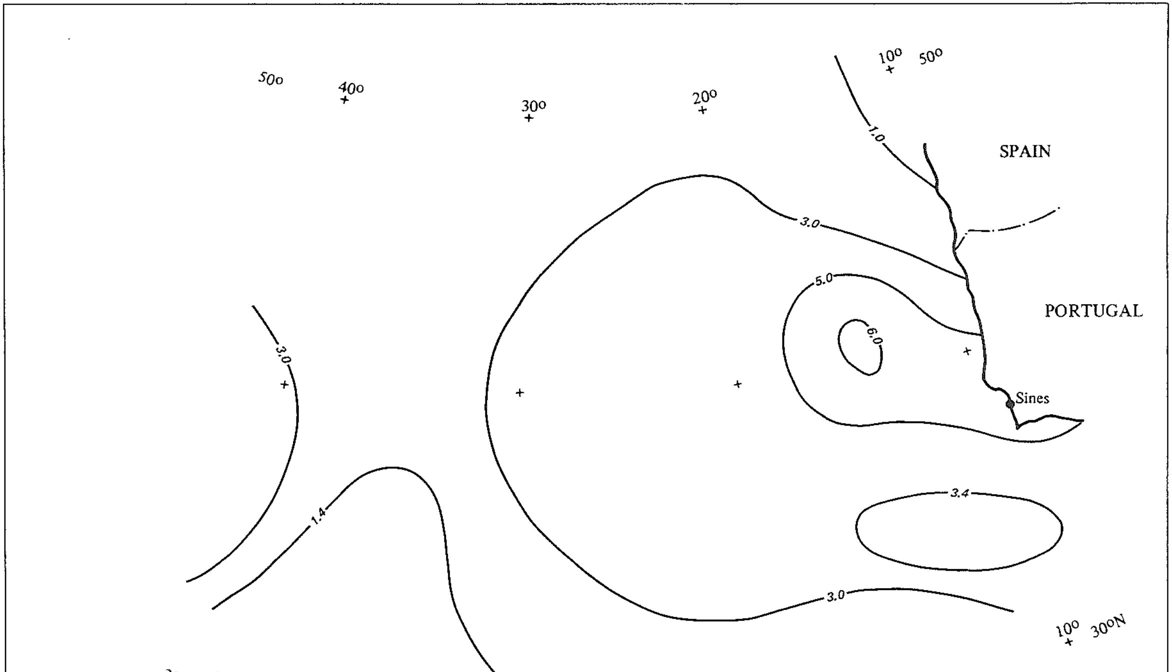
50 yr	62 kn
10 yr	56 kn
1 yr	48.5 kn

The Meteorological Office study (once every 12 hours) suggests that whilst hourly-mean winds in excess of 60 kn were present at about 40^{o}N and between 20^{o} and 30^{o}W at 1200 hr on 25 February, and possibly up to 70 kn, and that there were winds of over 50 kn at about 20^{o}W at 0000 hr on 26 February, these very severe winds did not persist long at Sines if they were achieved at all. Accordingly, it may be assumed that whilst the wind-field at or near Sines may not have been worse than, or may even not have achieved, the 10-year level, the swell arriving at the coast would probably have been more severe than the Sines winds indicated, but that there is no firm evidence on which to base an objective assessment of the rarity of the event. It is possible that the average return period of the waves was of the order of 10 years.

TABLE 1

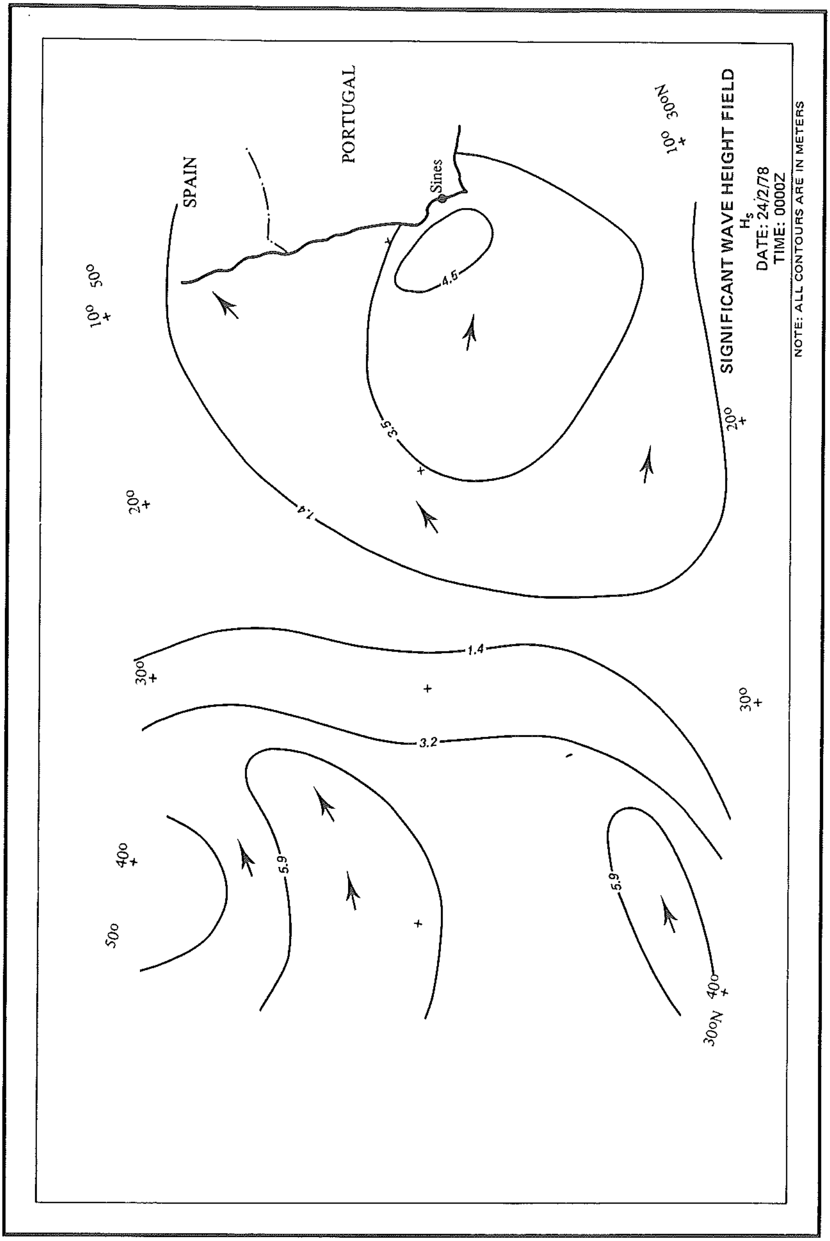
SIGNIFICANT WAVE HEIGHTS IN METERS, ZERO-CROSSING PERIOD IN SECONDS

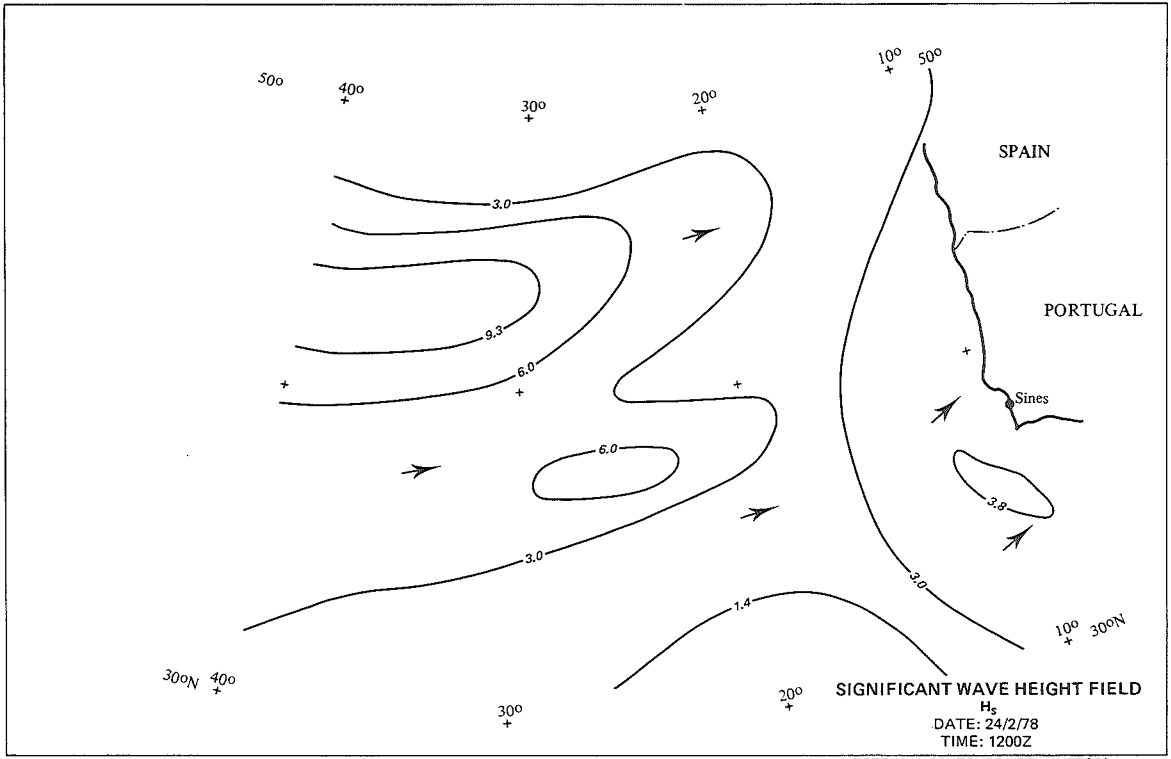
Date 1978	Time GMT	Prediction H_s	Prediction T_z	Visual H_s	ROCA H_s
22 Feb.	Noon	3.2	6	5	
	18				
23	00				
	06				
23	Noon	5.6	7	6.3	5.85
	18			5.4	6.4
24	00	4.3	8	3.2	4.8
	06			3.4	4.9
24	Noon	3.5	7	5.6	
	18				
25	00	4.9	8		
	06				
25	Noon	3.6	7	4.9	
	18			5	
26	00	5.9	13	5.6	
	06			6.5	6.4
26	Noon	9.1	14	7.9	8.2
	18			5	6.6
27	00	4.5	11	5.4	5.4
	06			6.5	
27	Noon	3.0	8	5.3	
	18			6.1	
28	00	4.0	9	3.2	
	06			3.4	
28	Noon	5.0	9	4.5	
	18				5.9
01 Mar.	00	5.3	10		5.7
	06				5.7
01	Noon	2.9	6	4.8	
	18			3.4	
02	00	2.1	8		
	06				
02	Noon	2.1	8		
	18			5.1	



SIGNIFICANT WAVE HEIGHT FIELD
H_s
DATE: 23/2/78
TIME: 1200Z

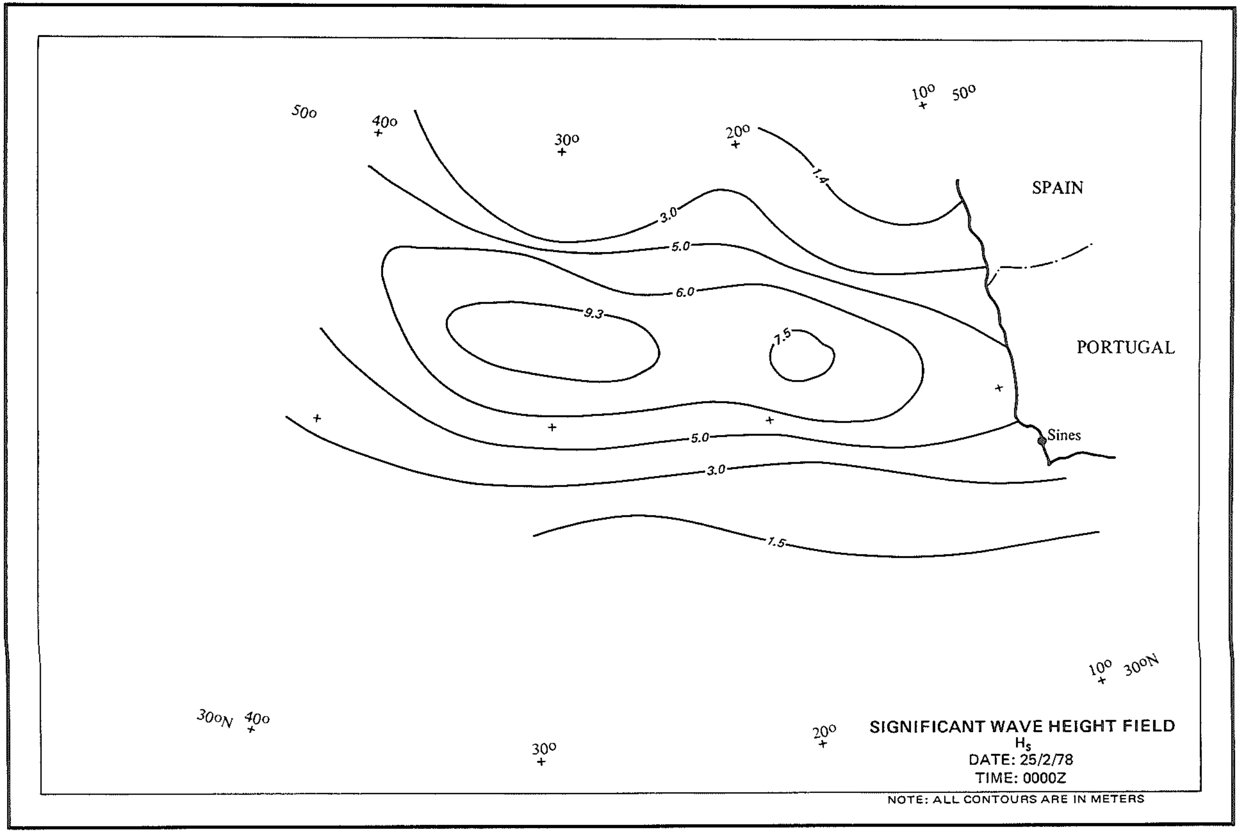
NOTE: ALL CONTOURS ARE IN METERS


SIGNIFICANT WAVE HEIGHT FIELD
H_s
DATE: 24/2/78
TIME: 0000Z
NOTE: ALL CONTOURS ARE IN METERS
SPAIN
PORTUGAL
Sines
4.5
3.5
1.4
3.2
5.9
10° 50°
10° 30°N
20°
30°
40°
50°
30°N

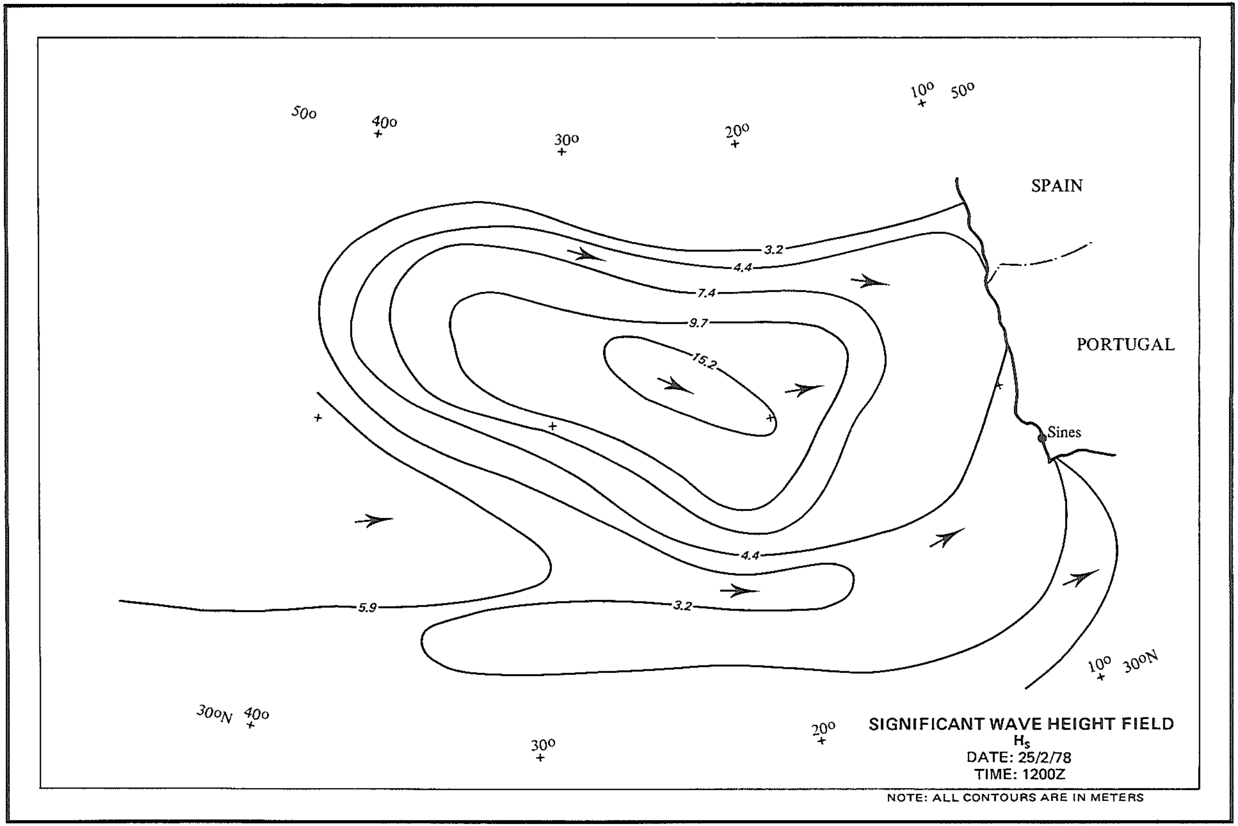


SIGNIFICANT WAVE HEIGHT FIELD
H_s
DATE: 24/2/78
TIME: 1200Z

NOTE: ALL CONTOURS ARE IN METERS

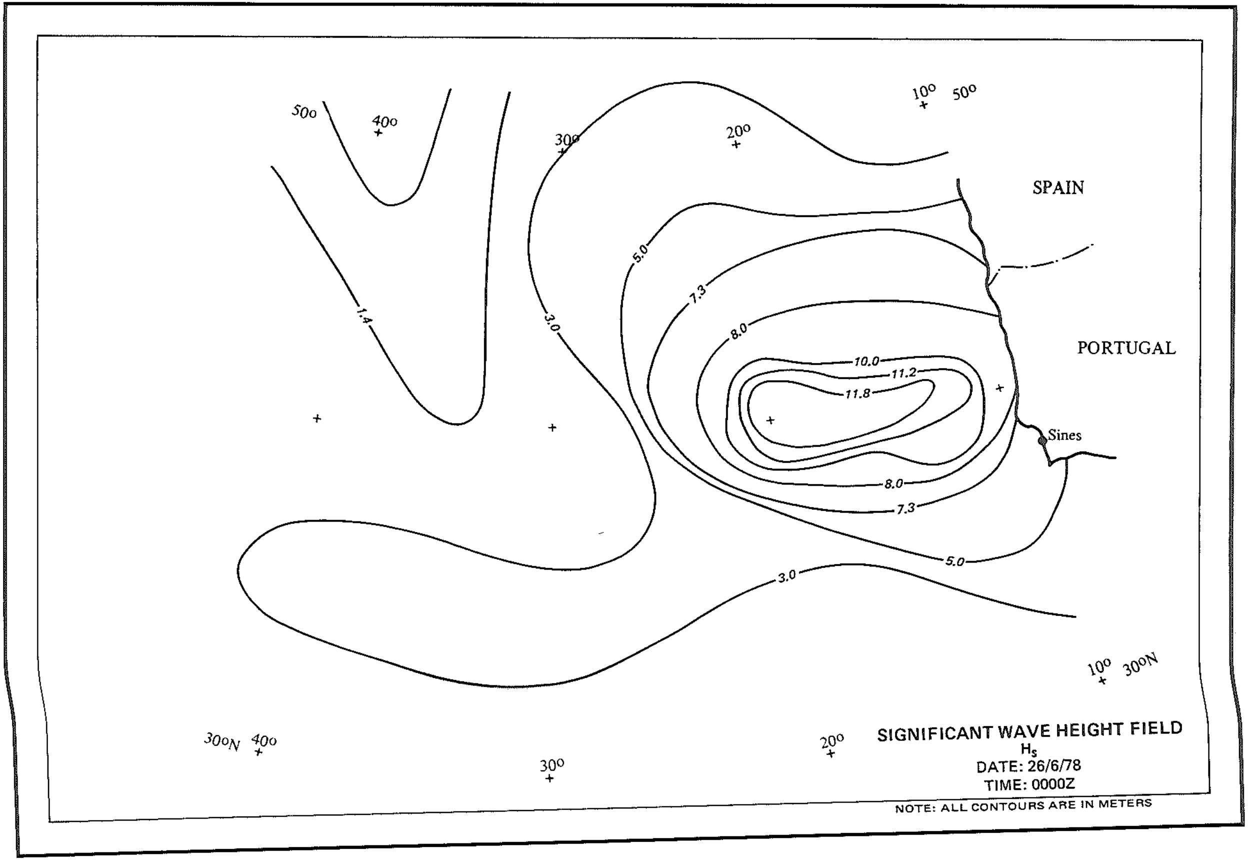


SIGNIFICANT WAVE HEIGHT FIELD
H_s
DATE: 25/2/78
TIME: 0000Z
NOTE: ALL CONTOURS ARE IN METERS

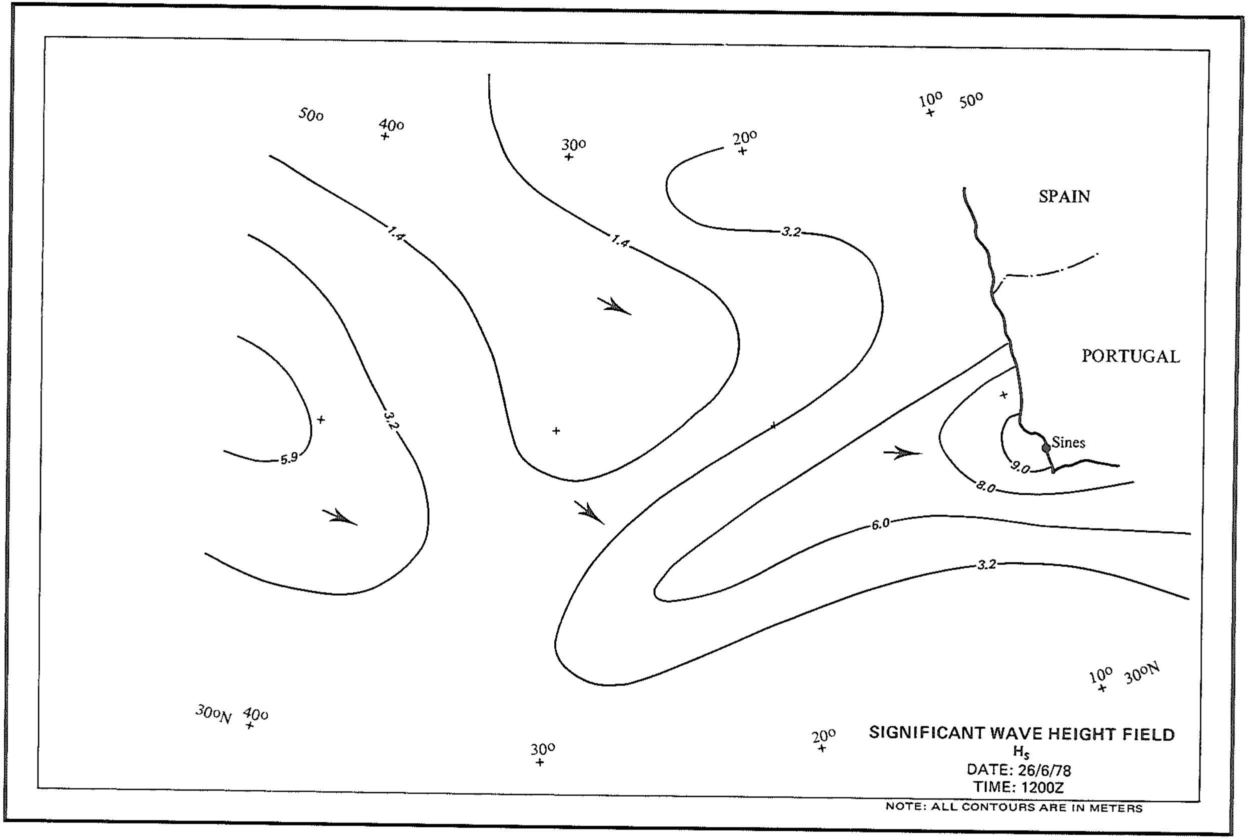


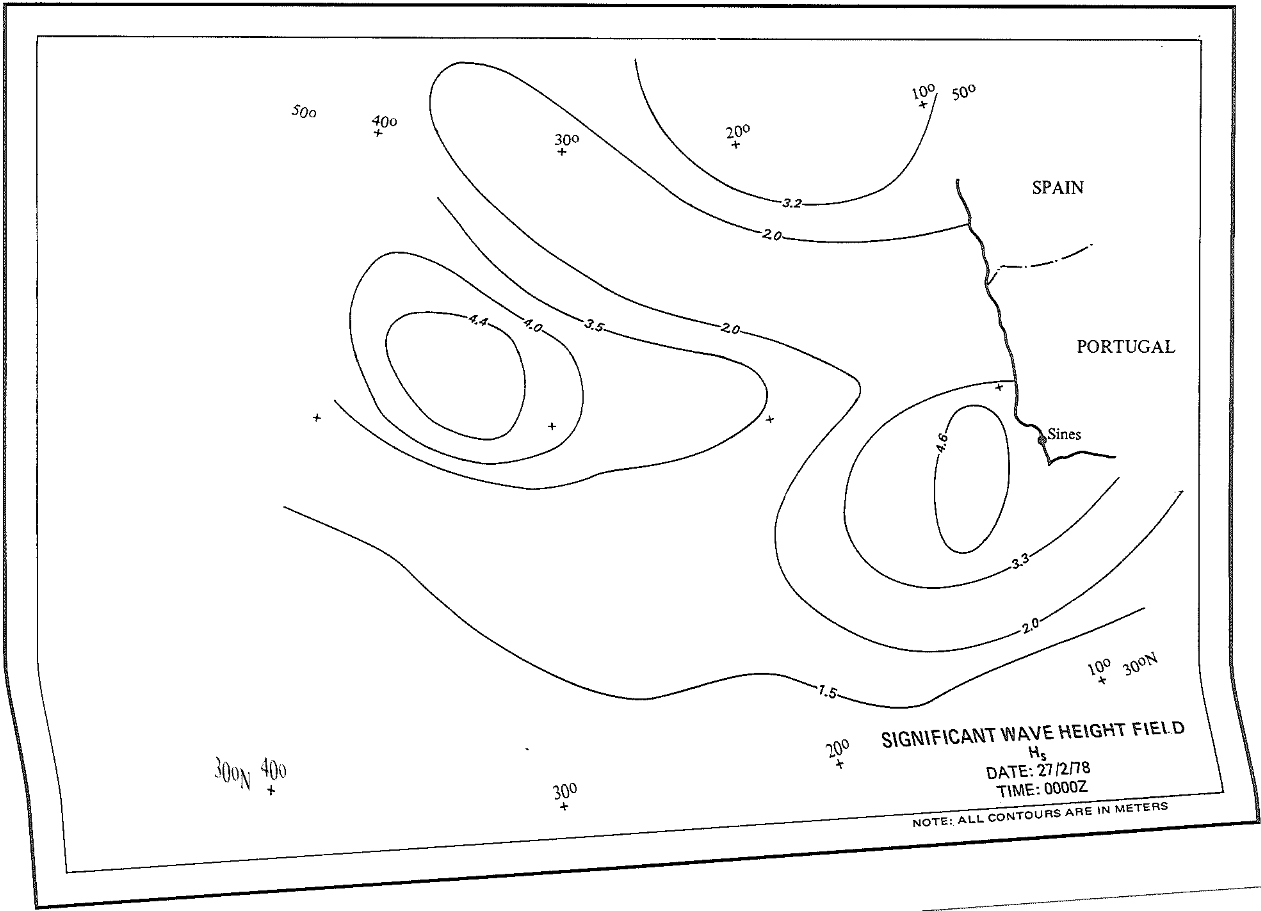
SIGNIFICANT WAVE HEIGHT FIELD
H_s
DATE: 25/2/78
TIME: 1200Z

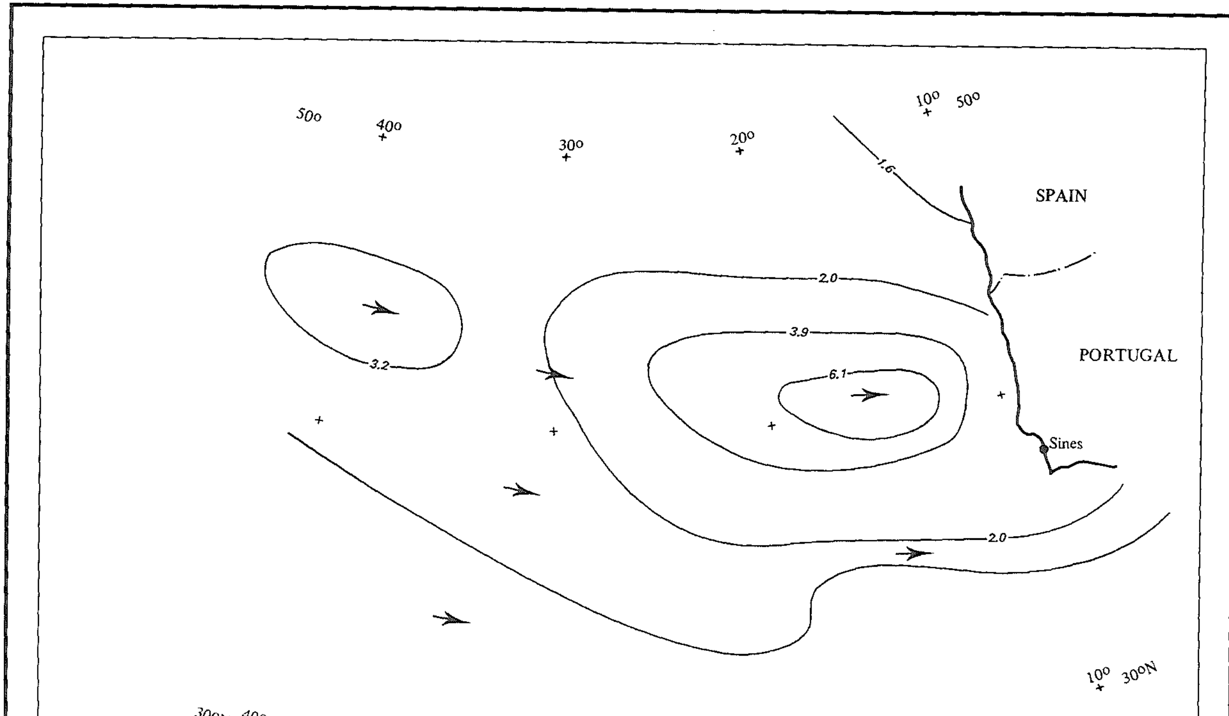
NOTE: ALL CONTOURS ARE IN METERS



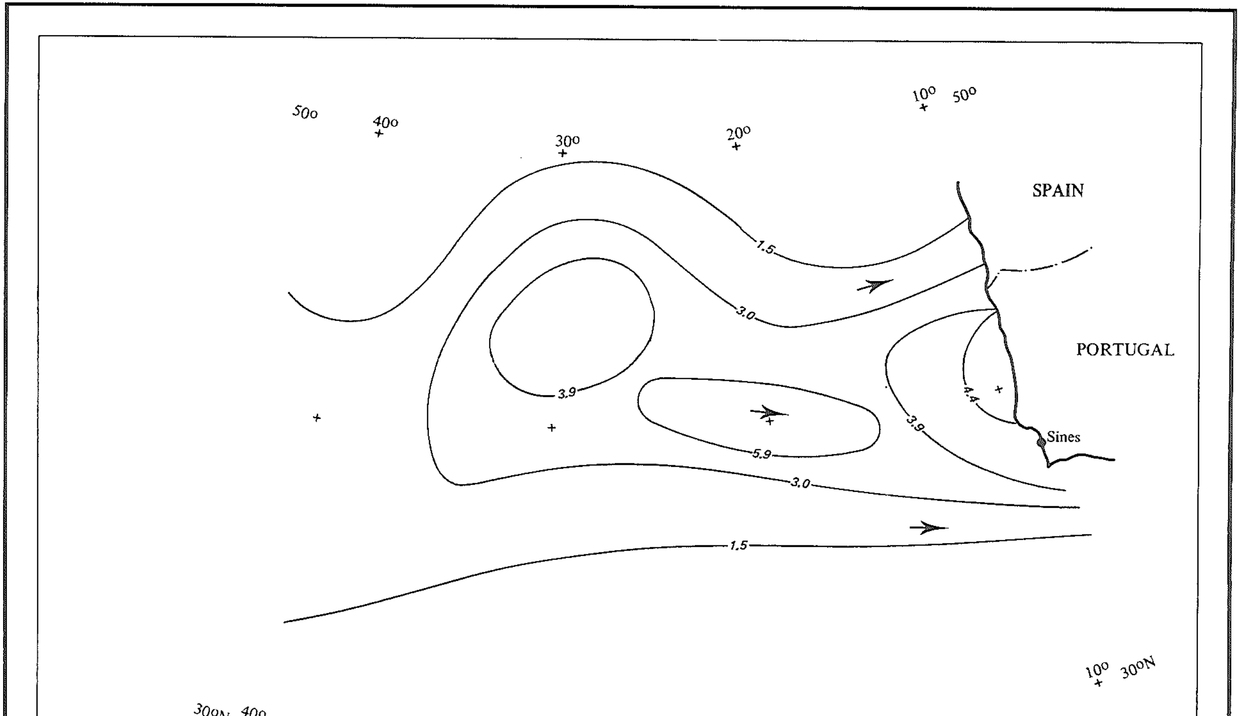
SPAIN
PORTUGAL
Sines
1.4
3.0
5.0
7.3
8.0
10.0
11.2
11.8
10° 50°
40°
50°
30°
20°
10° 30°N
30°N 40°
30°
20°
SIGNIFICANT WAVE HEIGHT FIELD
H_s
DATE: 26/6/78
TIME: 0000Z
NOTE: ALL CONTOURS ARE IN METERS


SPAIN
PORTUGAL
Sines
9.0
8.0
6.0
3.2
1.4
5.9
10° 50°
20°
30°
40°
50°
10° 30°N
20°
30°
30°N 40°
SIGNIFICANT WAVE HEIGHT FIELD
H_s
DATE: 26/6/78
TIME: 1200Z
NOTE: ALL CONTOURS ARE IN METERS


SPAIN
PORTUGAL
Sines
3.2
2.0
3.5
4.0
4.4
4.6
3.3
1.5
50o
40o
30o
20o
10o
30oN
SIGNIFICANT WAVE HEIGHT FIELD
Hs
DATE: 27/2/78
TIME: 0000Z
NOTE: ALL CONTOURS ARE IN METERS

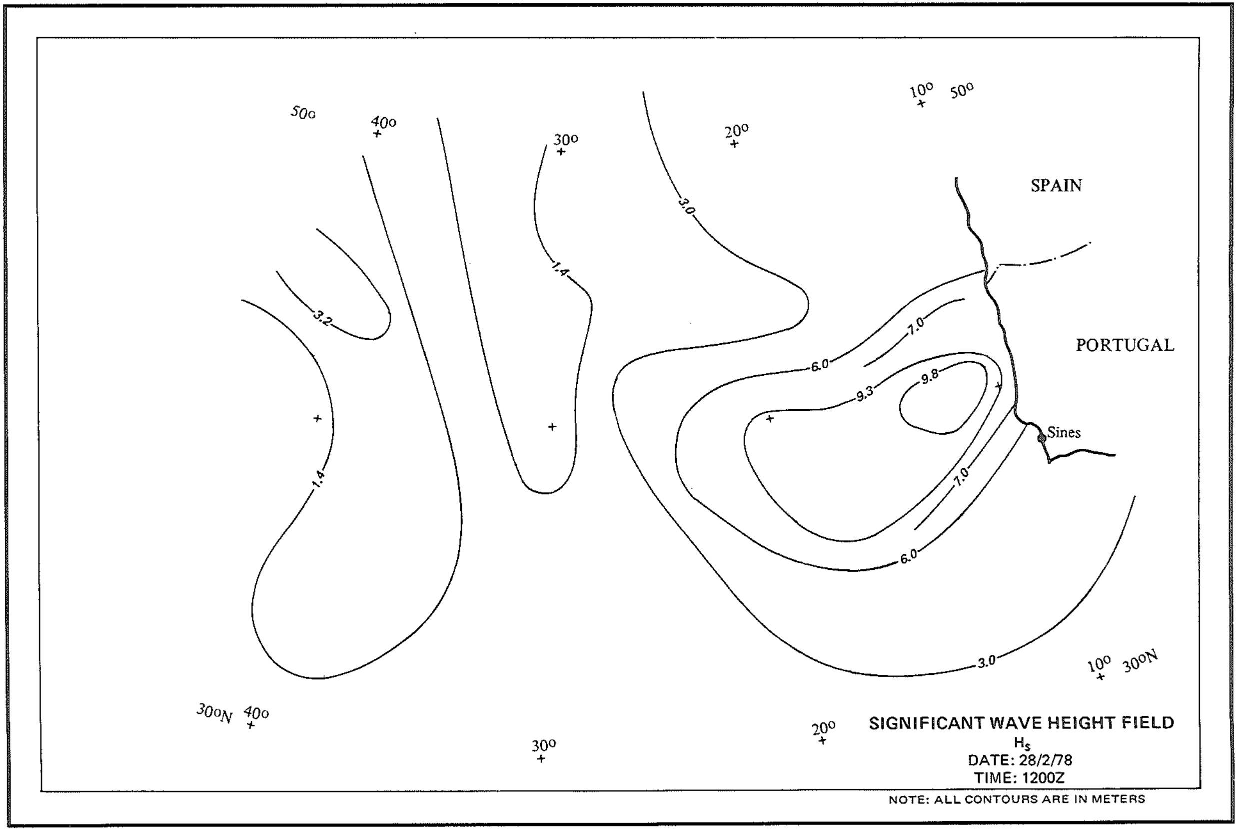


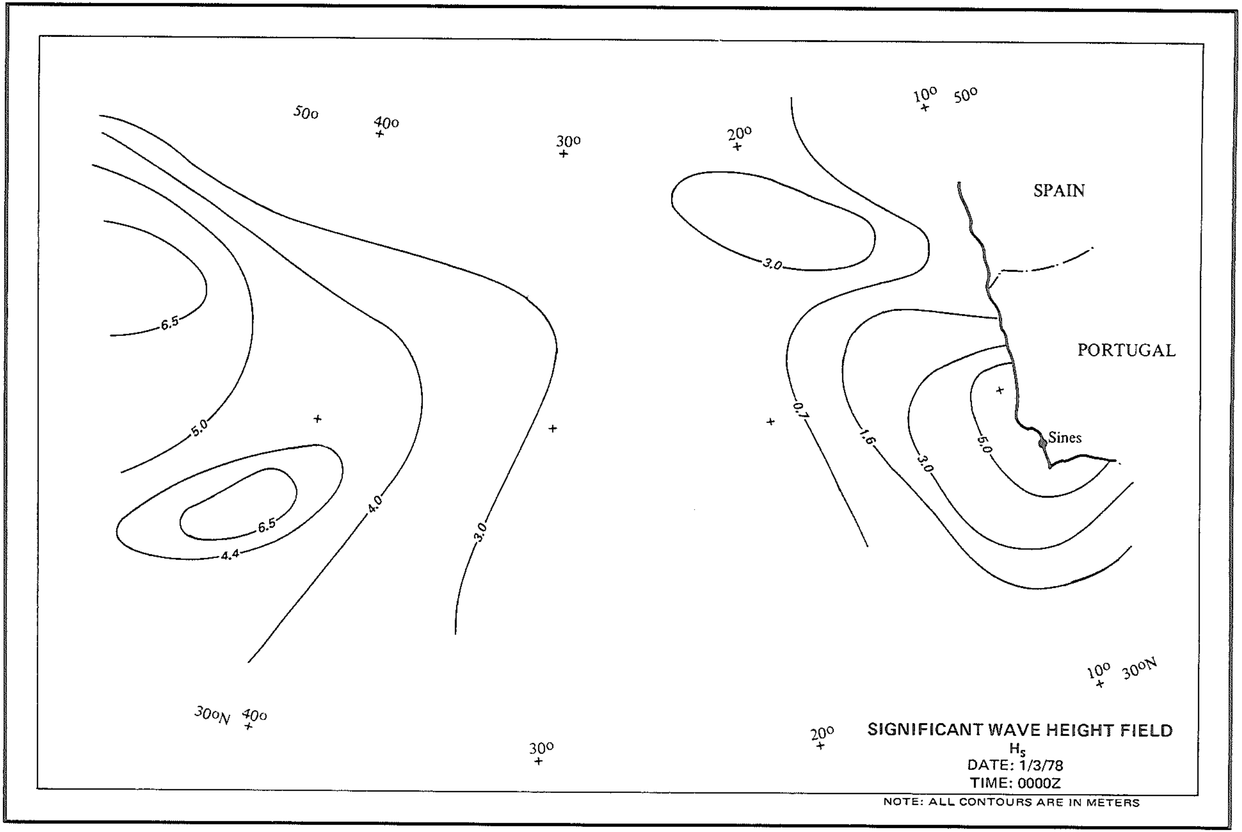
SIGNIFICANT WAVE HEIGHT FIELD
H_s
DATE: 27/2/78
TIME: 1200Z
NOTE: ALL CONTOURS ARE IN METERS



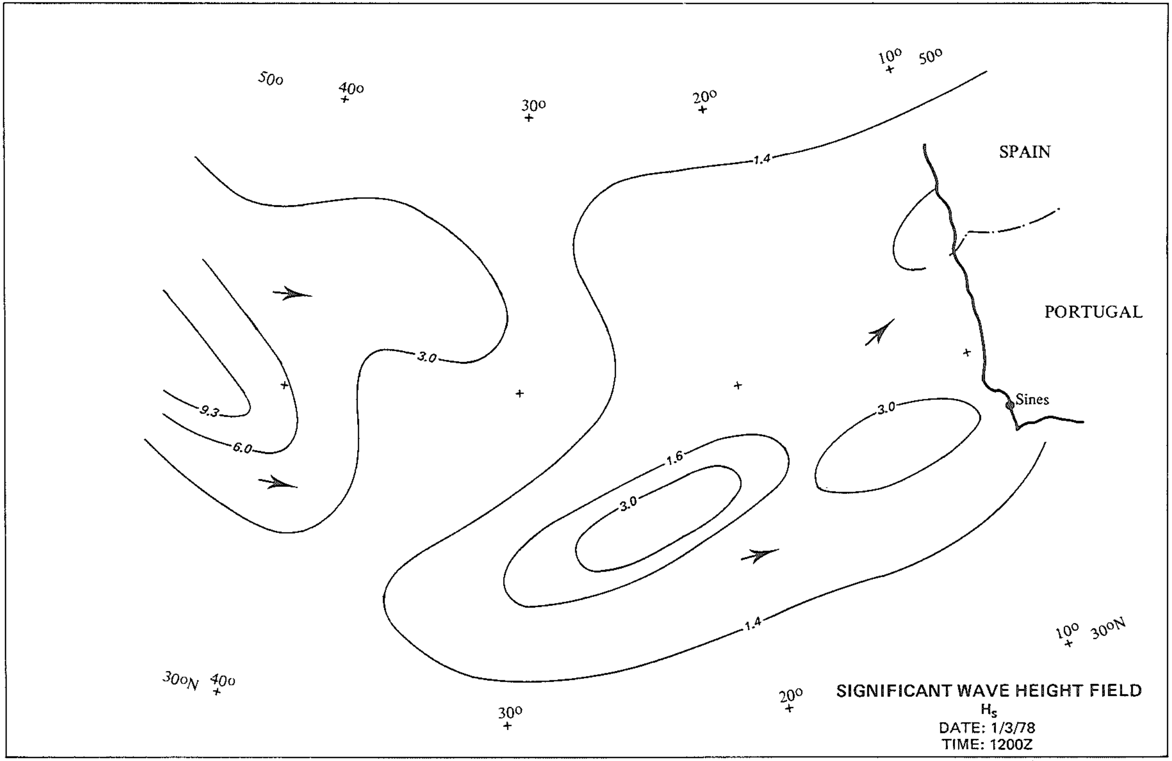
SIGNIFICANT WAVE HEIGHT FIELD
H_s
DATE: 28/2/78
TIME: 0000Z

NOTE: ALL CONTOURS ARE IN METERS


SPAIN
PORTUGAL
Sines
50°
40°
30°
20°
10° 50°
10° 30°N
30°N 40°
30°
20°
3.0
1.4
3.2
6.0
7.0
9.3
9.8
SIGNIFICANT WAVE HEIGHT FIELD
H_s
DATE: 28/2/78
TIME: 1200Z
NOTE: ALL CONTOURS ARE IN METERS

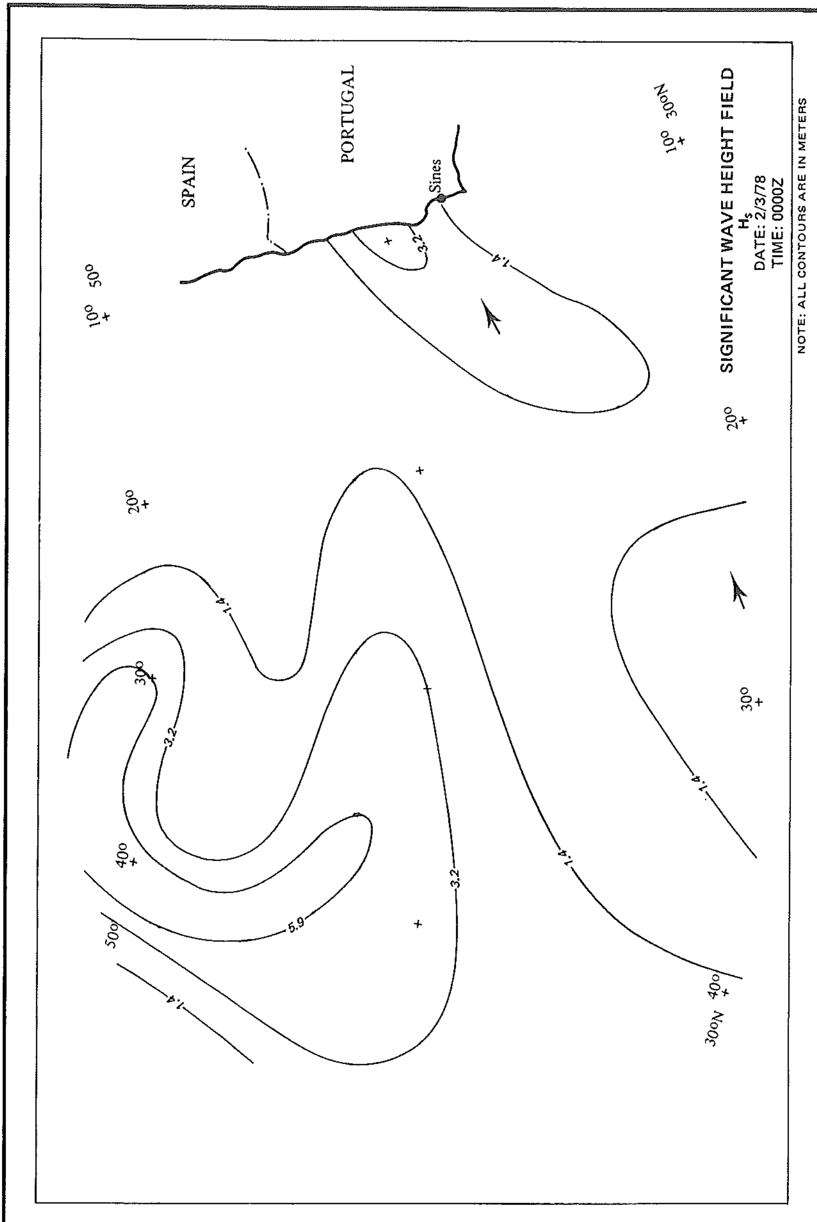


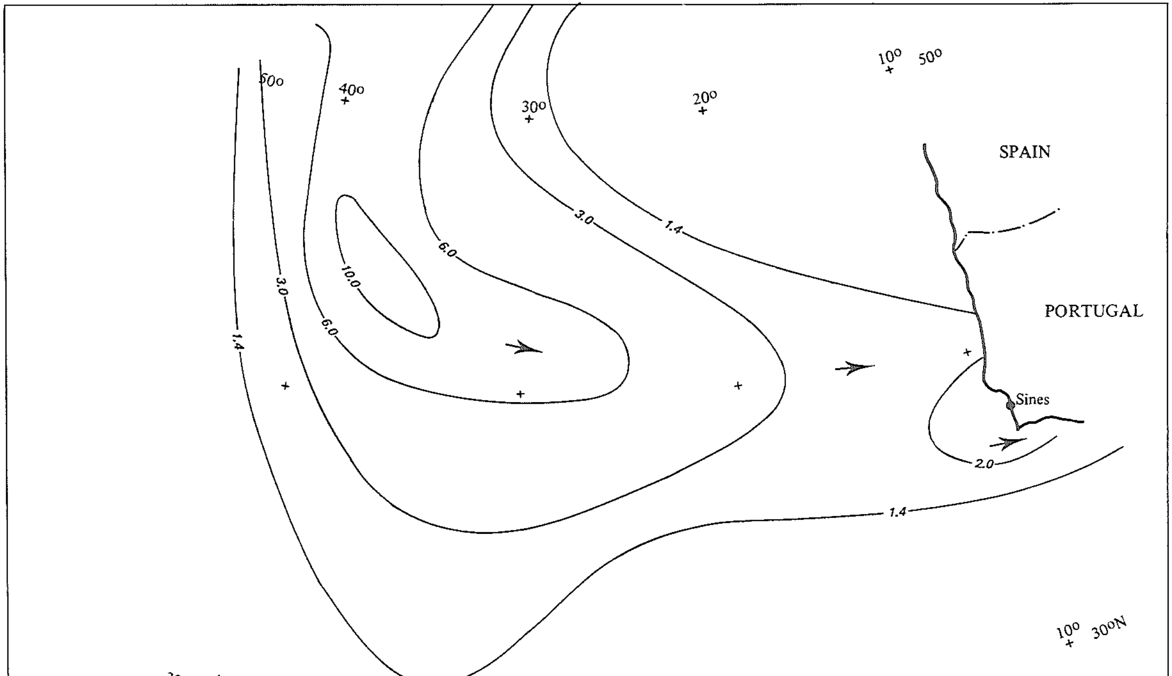
SIGNIFICANT WAVE HEIGHT FIELD
H_s
DATE: 1/3/78
TIME: 0000Z
NOTE: ALL CONTOURS ARE IN METERS



SIGNIFICANT WAVE HEIGHT FIELD
H_s
DATE: 1/3/78
TIME: 1200Z

NOTE: ALL CONTOURS ARE IN METERS


SPAIN
PORTUGAL
Sines
10° 50°
10° 30°N
20°
30°
40°
50°
30°N
1.4
3.2
5.9
SIGNIFICANT WAVE HEIGHT FIELD
H_s
DATE: 2/3/78
TIME: 0000Z
NOTE: ALL CONTOURS ARE IN METERS



SIGNIFICANT WAVE HEIGHT FIELD
H_s
DATE: 2/3/78
TIME: 1200Z

NOTE: ALL CONTOURS ARE IN METERS

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LABORATOIRE CENTRAL D'HYDRAULIQUE DE FRANCE

PORT DE SINES

CARACTERISTIQUES AU LARGE

DE LA TEMPETE

DU 26 FEVRIER 1978

RAPPORT N°1

1 - TEXTE

1 - INTRODUCTION

Le 26 Fevrier 1978, une tempete sevissant au large des cotes du PORTUGAL, a cause des dommages importants a la digue de protection du port en construction a SINES.

La bouee de houle mouillee a l'exterieur du port ayant cesse d'emettre et de renseigner sur les houles a la cote, la "SOCIETA ITALIANA PER CONDOTTE D'ACQUA" a demande au Laboratoire Central d'Hydraulique de France (L.C.H.F.) de determiner les caracteristiques de cette tempete au large du port a l'aide des donnees disponibles pour d'autres sites ou des etudes specifiques susceptibles d'etre realisees.

Le present rapport presente les conclusions deduites de l'examen, de l'interpretation et de la synthese des donnees ainsi reunies. Afin que sa lecture en soit aisee, il est constitue d'un texte limite a l'essentiel et complete par des annexes relatives aux principales sources d'informations utilisees.

2 - GENESE DE LA TEMPETE

Au cours du mois de Fevrier 1978, une situation depressionnaire a persiste sur l'Atlantique Nord.

- du 15 au 20 Fevrier, une premiere depression est restee quasi stationnaire avec des minimums barometriques, compris entre 944 et 984 mb,
- du 21 au 23 Fevrier, une seconde depression a maintenu des zones de basse pression au niveau du Centre Atlantique Nord,
- du 24 au 28, une nouvelle perturbation, avec des depressions allant graduellement de 948 a 980 mb, d'abord peu mobile, s'est deplacee vers le NE.

Cette circonstance, d'un type classique en hiver, a engendre des vents violents, parfois superieurs a 60 noeuds, qui ont a leur tour leve au large des vagues ayant atteint les cotes du PORTUGAL le 26 Fevrier 78 ou elles se sont combinees avec celles d'une forte mer de vent causees par la position alors atteinte par la depression.

On peut des maintenant remarquer, pour fixer un premier ordre de grandeur que l'application des abaques de BREITSCHNEIDER a une situation de ce type (vent de 60 noeuds pendant 12 h sur un fetch de 800 milles) conduit a prevoir des amplitudes et periodes significatives de 10 m et 12,5s respectivement, valeur qui, de par la structure de la houle, seront necessairement depassees par les plus fortes vagues.

3 - MESURES DIRECTES (SUR BOUEES DE HOULE)

3.1 - INTRODUCTION

Le long des cotes du PORTUGAL, des mesures sur bouee de houle sont effectuees en principe toutes les 6 heures, a CABO DA ROCA a 100 km au nord de SINES, ainsi qu'a l'exterieur et a l'interieur du port de SINES. Cependant, la bouee de SINES exterieur a cesse d'emettre entre le 25 a 12 h et le 27 a 12 h, et la bouee SINES interieur n'a ete mise en service que le 25 a 18 h. Seule la bouee de CABO DA ROCA a emis de facon normale au cours de cette periode.

3.2 - RESULTATS

Le tableau ci-dessous indique, pour les 25 et 26 Fevrier, les amplitudes dites "H max"* pour SINES et dites "H significatif" pour CABO DA ROCA et la periode mesuree, (periode moyenne "zero crossing").

Date	Heure	Bouee SINES ext. (H max en m)	Bouee SINES int. (H max en m)	Bouee CABO DA ROCA (Hs en m)	(To en s)
25	00	6,8	-	4,7	8,3
	06	10,6	-	4,0	8,6
	12	7,8	3,1	4,0	8,4
	18	-	2,8	4,3	8,4
26	00	-	3,2	5,0	8,6
	06	-	3,0	7,0	13,0
	12	-	3,0	8,5	13,8
	18	-	3,4	5,9	12,9

* - En fait deduites d'enregistrements de 10 minutes, correspondant a une population de l'ordre de 50 vagues seulement.

Il resulte des valeurs ci-dessus, plusieurs resultats importants qui permettent d'etablir les caracteristiques de la mer, au large de SINES, a cette epoque:

a) - Les enregistrements ayant eu lieu toutes les 6 heures, il parait evident compte-tenu de l'heure du debut du paroxysme a SINES* - que l'amplitude significative au paroxysme a CABO DA ROCA a atteint 9 m. (Voir fig. 3 de l'annexe 2).

b) - La correlation que l'on peut faire (voir fig. 2 annexe 2) entre les valeurs dites "H max" pour SINES et "H sig" pour CABO DA ROCA a montre que cette meme valeur de 9 m caracterise aussi le "H sig" au large de SINES.

(Rappelons que, en theorie, la "hauteur significative" d'une houle est la moyenne des amplitudes du tiers des plus hautes vagues, cependant que l'amplitude maximale, par definition est la plus haute denivelee creux-crete observable dans un enregistrement donne. Lorsque cet enregistrement est suffisamment long "H max" vaut 1,8 fois H s environ).

c) - La periode moyenne au paroxysme, enregistree, soit 13,8 s implique des valeurs nettement plus longues pour les vagues bien formees les plus hautes. Les relations usuelles (Cf. par exemple SIBUL, cite par WIEGEL) impliquent alors, pour les plus hautes vagues, des periodes de 17 a 20 s.

En conclusion, les mesures disponibles, pour cette tempete et dans cette partie du littoral indiquent:

- une amplitude significative de 9 m,
- une periode moyenne de pres de 14 s et par consequent des vagues les plus hautes, de 12 a 16 m avec des periodes associees de 17 a 20 s au large du port.

* - Franchissement de la digue a basse-mer, par des vagues ayant emporte deux ouvriers.

4 - INFORMATIONS COMPLEMENTAIRES

4.1 - INTRODUCTION

Parmi les autres informations disponibles precisant les ordres de grandeur des caracteristiques de la tempete, nous avons retenu les suivantes:

- les observations de navigateurs et les observations visuelles de houle depuis la cote,
- les previsions de houle,
- la reconstitution par une methode de calcul dite "DSA" (densite spectrale angulaire) mise au point par la meteorologie francaise,
- les enregistrements realises en d'autres sites,
- les temoignages, les films, les photos.

Compte tenu de l'imprecision ou du caractere subjectif de certaines d'entre elles, nous les considerons plutot comme des indices venant renforcer les faits deja presentes ci-dessus. L'annexe 3 precise les informations reusmees ci-apres.

4.2 - OBSERVATIONS DE NAVIGATEURS

4.2.1 - Observations au large

Plusieurs navigateurs ont ressenti la presence du paroxysme de l'agitation entre le 25 et le 26 avec des houles et/ou des mers de vent atteignant jusqu'a 18 m de hauteur significative, les periodes, egalement significatives, atteignant 15 s.

4.2.2. - Observations face aux cotes du PORTUGAL

Le paroxysme mesure dans un cercle de 200 km de rayon autour de SINES avait, d'apres les navigateurs les valeurs suivantes:

le 26 a 00 h	Hs = 12,3 m	Ts = 9s	D = 300^{o}
le 26 a 6 h	Hs = 14,7 m	Ts = 11s	D = 310^{o}
le 26 a 12 h	Hs = 9,6 m	Ts = 10,4s	D = 288^{o}
le 26 a 18 h	Hs = 10,3 m	Ts = 12,5s	D = 290^{o}

4.2.3 - Observations visuelles depuis la cote

Les observations visuelles effectuees a CABO DA ROCA font ressortir un premier paroxysme le 25 a 6 h ayant une amplitude significative de 5-7 m, et un second le 26 avec 4-6 m.

4.3 - PREVISION DE HOULE

4.3.1 - Prevision au large

Le maximum d'agitation prevu entre le 20 et le 28 Fevrier s'est deplace, comme cela est indique dans l'annexe 4, atteignant son paroxysme le 25 a 12 h avec une amplitude H $_{1/10}$ de 10,4 m.

4.3.2 - Prevision pour les cotes du PORTUGAL

De nombreuses previsions de l'agitation, d'ailleurs contradictoires, on ete faites. Il en ressort que pour le 25, les amplitudes sont de l'ordre de 5 m alors que pour le 26 elles atteignent ou depassent 10 m (H $_{1/10}$).

4.4 - RECONSTITUTION PAR LA METHODE DSA

4.4.1 - Introduction

A notre demande, la section maritime du service meteorologie metropolitain de Paris a procede au calcul des caracteristiques de la tempete par la methode dite DSA (densite spectrale angulaire).

Ce calcul permet de connaitre a des intervalles de temps rapproches -3 heures dans ce cas - les caracteristiques de l'agitation suivant trois spectres: direction, amplitude et periode - Ces "sorties" etant obtenues pour des points predetermines, nous avons choisi 6 points proches de SINES et 6 autres points, 3 vers les cotes marocaines et 3 vers le cap Finistere afin de permettre des recoupements eventuels avec d'autres observations.

4.4.2 - Principe de la methode

La methode mise au point par le service maritime de la meteorologie metropolitain de Paris, appelee Densite Spectrale Angulaire (DSA) consiste en un modele mathematique permettant le calcul en un point donne des caracteristiques spectrales de l'agitation. Ce calcul est effectue en utilisant les formules classiques de transferts d'energie entre le vent et la mer, un sous programme etablissant les modifications des caracteristiques ainsi calculees pour tenir compte de la propagation. Precisons que ce modele a ete ajuste puis controle par calcul a posteriori de situation effectivement mesurees par les donnees fournies par plusieurs fregates meteorologiques.

4.4.3 - Resultats

Le tableau ci-dessous resume les caracteristiques de l'agitation reconstituee pour le point 1331, que l'on peut qualifier de "au large de SINES" (voir schema 1 de l'annexe 2).

CARACTERISTIQUES DE L'AGITATION - POINT 1331

Date	Heure	Amplitude ($H_{1/10}$) (m)	Periode correspondant au max. d'energie (s)	Direction correspondant au max. d'energie
25	12	6,2	10	WNW
	15	6,7	10	"
	18	7,0	10	"
	21	7,6	10	"
26	00	7,7	10	WNW
	03	8,5	19	"
	06	8,5	19	"
	09	8,4	16	"
	12	8,5	16	"
	15	8,4	16	"
	18	8,0	16	"
	21	7,6	16	"
27	00	6,6	16	MNW

Le maximum d'energie totale, ou moment d'ordre zero, a ete calcule egal a 4,52 m^2 pour ce point le 26 a 12 h, dont plus de 60% etait concentre pour des periodes superieures a 14,5s et des directions comprises entre W et NW.

L'annexe 2 qui presente les resultats complets de cette reconstitution montre que le paroxysme de la tempete est caracterise par les valeurs suivantes:

- amplitude ($H_{1/10}$) de l'ordre de 9 m,
- direction de provenance: entre W et NW,
- periodes du ($H_{1/10}$): entre 16 et 19s.

4.5 - LES ENREGISTREMENTS EN D'AUTRES SITES

Nous avons pu etudier les enregistrements de houle effectues devant les cotes du Maroc et dans l'estuaire de la Gironde (France). Pour ce meme 26 Fevrier, il est nettement etabli en ces points que les vagues les mieux formees et les plus fortes (14,6 m au Maroc, 9,2 m dans la Gironde) avaient des periodes de 20-22 secondes. Les rapports entre T max et To d'une part, H max et H s d'autre part etaient respectivement egaux a 2 et 1,6.

4.6 - FILM, PHOTOS

Un film a ete realise pendant la tempete et de nombreuses photos ont ete prises. Il en ressort que:

- la periode des vagues, deduite de celle des jaillissements, filmee, etait de 18 s en moyenne,
- ces jaillissements, atteignant une centaine de metres, ont ete photographies, ce qui laisse supposer des vagues incidents de 16-17 m (en admettant que la hauteur d'un jaillissement sur un parement vertical est egale a 6 fois l'amplitude incidente).

4.7 - CONCLUSION

Les informations complementaires recueillies - qui sont detaillees dans l'annexe 4 - permettent donc de confirmer les valeurs deduites du calcul pour les caracteristiques de la tempete. Si la precision de ces informations est parfois discutable leur convergence d'ensemble et leur bon accord avec les donnees indiscutables conduit finalement a une certitude dans l'appreciation des phenomenes.

5 - CONCLUSIONS

Compte tenu des elements ci-dessus, les principaux elements caracterisant aux larges de SINES la tempete qui y a culmine le 26.2.78 sont les suivants:

5.1 - GENESE DE LA TEMPETE

La tempete du 26 Fevrier 1978 a ete engendree par une situation depressionnaire dont les caracteristiques statiques (depression et gradient barometrique, situation a un moment donne) et dynamique (trajectoire suivie et vitesse de deplacement) ont permis de lever des vagues, de les amplifier et de les deplacer jusqu'au paroxysme mesure et subi sur de nombreuses cotes.

5.2 - CARACTERISTIQUES DE LA TEMPETE AUX APPROCHES DE SINES

En utilisant et en exploitant les nombreux documents, d'ordre general ou plus specifiques relatifs a cette tempete, qui ont pu etre reunis nous avons finalement abouti aux conclusions suivantes:

- le paroxysme a eu lieu le 26 Fevrier 1978, entre 9 et 12 h,
- la direction d'origine des vagues etait entre W et WNW,
- l'amplitude significative etait alors comprise entre 8,5 et 9,5 m
- l'amplitude H1/10 des vagues etait de 12 a 16 m (avec quelques vagues exceptionnelles encore plus hautes),
- la periode moyenne etait de 12-14s
- la periode liee au $H_{1/10}$ etait de 18-19s,
- certaines vagues avaient une periode de 20-22s

Outre le fait que l'amplitude de cette tempete etait elevee, qu'elle survenait 3 jours apres la precedente, nettement moins forte cependant, il semble que ses effets destructeurs ressentis depuis le MAROC jusqu'en FRANCE, soient principalement lies a la periode des vagues les mieux formees.

L'Ingenieur charge de l'Etude — Le Directeur des Etudes

C. LANGLAIS — C. ORGERON

APPENDIX E

Rationale for Unreinforced Dolos at Sines

Prepared by Bertlin and Partners

April 1978

SINES ACCIDENT

Reasons for not providing steel reinforcement in dolos units

1. Introduction

1.1 The dolos armor unit was developed in South Africa by Mr. Merrifield and first used at East London in 1964. Hydraulic model tests on the unit were carried out initially by Zwamborn at the CSIR laboratory at Stellenbosch, in 1965.

1.2 The information on the units was published at the coastal engineering conference in September 1966. Since that date they have been used on an increasing number of breakwater contracts throughout the world. Hydraulic model testing has been used in developing most of the designs. For this purpose and for basic research, hydraulic model testing of dolos armor units has been carried out in most of the major hydraulic laboratories.

1.3 The proportions of the dolos units have generally followed the proportions initially adopted by Merrifield. Changes which have been introduced related generally to the adoption of thicker units, particularly as the units become heavier, and the introduction of fillets or chamfers at the junction between the trunk and the legs in order to improve the strength of the units at this point.

1.4 Dolos units in South Africa were developed using mass concrete; and as far as we are aware reinforcement has only been introduced into dolos on one project in America (Humboldt Bay).

1.5 A useful summary of the number of dolos used at 15 of the main sites throughout the world has been included in a paper by Lillevang presented to the coastal engineering conference in 1976. This summarizes information on breakage of dolos on projects where a total of 150,000 dolos were used ranging from 3 to 40 t in weight.

2. Breakage of dolos

2.1 The breakage records reported by Lillevang for in-service conditions range from 0 to 2.8 percent for unreinforced dolos. The one example of reinforced dolos had an in-service breakage of 0.75 percent.

2.2 Breakage in the casting yard or during construction of the breakwater generally exceeds the breakage in-service.

2.3 The two worst sites for breakage during casting and construction are at Crescent City and Gaansbaai, with breakages of about 9 percent and about 4 percent, respectively.

2.4 Full details of the experience in breaking is not included in the paper, but Lillevang mentions some unusual factors causing breakages during construction at Crescent City, where cracking developed due to high temperature during casting, which contributes to the development of shrinkage stresses and formation of minute cracks. During construction numerous dolos which had not yet been interlocked in the armor face were moved during a storm and suffered impact breakages.

2.5 At Gaansbaai we understand that a sharp corner was provided at the junction between the legs and trunk and that shrinkage cracks were noted on many of the units in the casting yard. It is likely that these were the cause of breakage during placing.

2.6 One factor which Gaansbaai Bay and Crescent City dolos have in common is that the units were in both cases more slender than is recommended by Merrifield. The dolos were of 20 t and 40 t, with waist ratios of less than 0.30 and 0.32, respectively. These dimensions would only be adopted by Merrifield for smaller units. We understand that Merrifield would regard ratios of 0.32 and 0.35 to be more suitable in these cases.

3. Reinforced units at Humboldt Bay

3.1 It should be noted that the dolos at Humboldt Bay, the only one where reinforcement has been adopted, were identical in size to those used at Crescent City. The moulds used at Humboldt Bay were subsequently transferred to Crescent City for re-use on that project.

3.2 The adoption of reinforcement at Humboldt Bay was an arbitrary decision about which there has been some considerable controversy. Proposals were made during construction to omit the reinforcement, but this was only done experimentally for a limited number of dolos towards the end of the contract. Breakage of these un-reinforced units was greater than breakage of the reinforced units, but it is reported that this may be partly due to the fact that the unreinforced units were placed loosely on top of the completed armor face so that they were not interlocked and would have been very vunerable to movement under wave attack.

3.3 Static tests to destruction of both the reinforced and unreinforced dolos at Humboldt Bay showed no very significant increase in strength of the reinforced units compared with those without reinforcement. It should be noted that the amount of reinforcement used was relatively small, being of the order of minimum temperature steel. Having even this small amount of steel involved a substantial amount of money.

3.4 The Crescent City dolos constructed after completion of the Humboldt Bay jetty were all built without reinforcement.

4. Stresses in dolos units

4.1 These units cannot be fully analyzed by normal theoretical methods because of their complex shape.

4.2 The time allowed for the design of the Sines breakwater did not allow time for photoelastic stress analysis. A design with a waist ratio of 0.35 (compared with 0.32 at Humboldt Bay) was adopted in combination with large chamfers at the junction of leg and trunk. In the absence of photoelastic analysis some simplified stress calculations were carried out to compare the stresses likely to be developed in the Sines dolos with those for other dolos. This showed the advantages of the thicker waist and of the chamfer.

4.3 During preparation of the designs for the Sines dolos we received some advance information from Lillevang on the research work using photoelastic stress analysis which he was carrying out at that time. These results showed that a large chamfer of the type adopted in our ante projecto design was highly beneficial but that some further improvement could be obtained by replacing the chamfer by a radiused fillet. This information reached us after the dolos moulds were fabricated and it was not considered necessary to change the moulds at that stage. This view was confirmed when concreting started on site, and it was seen that the quality of concrete near the chamfer was good and that shrinkage cracks were not being formed at this point.

4.4 This paper by Lillevang has since been published and this confirms that the additional benefit of providing a rounded fillet compared with the type of large chamfer adopted at Sines is not great.

4.5 The paper also shows that the high local stresses at the junction between the leg and the trunk of the dolos occur so close to the surface that reinforcement would not be effective unless it were placed so close to the surface that corrosion of the steel would be unavoidable. He concluded that use of steel reinforcement is unnecessary provided that, for units over 30 t, chamfers or fillets are used and concrete of high tensile strength is specified.

5. Specification

5.1 In our specification for the Sines dolos we required a high strength concrete of 400 kg per sq cm at 28 days. We also specified the use of a low heat pozzolanic cement in order to minimize temperature effects during setting of the cement. We also specified careful curing. It should be noted that the contractor's design of molds included a compressible joint in the mold for the trunk of the dolos. The effect of this would be to reduce the tendency for tensile stresses to develop during shrinking.

5.2 As an additional precaution provision was made in the contract documents for a number of dolos to be tested to destruction, statically and dynamically, to check their strength. BCL letter 000557 dated 14th August 1974 refers.

6. Conclusion

6.1 With all these precautions and in light of the knowledge available to us at the time of design and subsequently, we are satisfied that the decision not to provide reinforcement was a correct one.

6.2 In this connection it should be noted that breakages during placing have been reported in the range of 2 - 4 percent. In view of the fact that most of the dolos have been placed using a floating crane in an area where the sea is rarely completely calm the dolos would appear to be well designed for casting and for the stresses imposed during placing.

6.3 We have seen no reports on the behavior of the dolos after placing. During site visits [before the storm of 26 February] we have seen very few broken dolos above water level. We have also noted a number of dolos which have moved a considered distance during the storm from their initial position without breaking. We therefore consider that there is evidence that the dolos had sufficient strength to resist normal movements in service.

We have no information regarding the number of breakages revealed by underwater inspection of the dolos which have been moved considerable distances during the collapse which occurred on 26th February. We would, however, expect such breakages as a consequence of the major displacements which have occurred.

APPENDIX F

Record of Dolos Placement by Day from 1974 to 1978

TABLE F-1 NUMBER OF DOLOS PLACED BY DAY IN 1974

Day	January		February		March		April		May		June		July		August		September		October		Nobember		December	
	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land
1																				9				
2																				32		17		30
3																				54				33
4																				9		33		10
5																						32		46
6																						35		6
7																						34		33
8																						35		
9																				16		18		28
10																				20				15
11																				22				32
12																						10		20
13																								27
14																				30		17		27
15																				73				
16																								29
17																								
18																				29		12		10
19																				15		20		31
20																						25		18
21																						50		
22																						16		
23																				15				
24																				22				
25																				24		16		
26																				17		45		
27																						32		
28																				30		24		
29																				35		39		
30																						32		
31																				26				
TOTAL																		75		478		542		394

TOTAL By Sea 0

TOTAL By Land 1489

TABLE F-2 NUMBER OF DOLOS PLACED BY DAY IN 1975

Day	January		February		March		April		May		June		July		August		September		October		November		December	
	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land
1													20		18		32							
2		52								30			20		20		18						36	8
3				69				7	20				20				34				36			
4		12		33			25						40			77	36				18		54	
5				24			16		55				34			52	18						78	
6		15		30					38							42					18		36	
7		15				46	20	20	15				20						18					
8		30			19	12	20	19					40			53			36					
9		40					27			26	18		41						36				16	
10		18						37		16			40				36				12		52	
11				20		29		55					36				36							
12					32								19								14		31	
13					27	29					36				32				18		30	63	17	
14						25				16	28		20	79							18			
15																							26	
16											18		20					20						21
17						55					18		19				36	14						105
18						81		33					20		16		35		17				53	
19											16		20		48		18	18			28		26	
20			22								37				48				36	73			36	
21			14	9				36			20		37								27			
22								53					20				54	18			54		18	
23											20		20		14		52	19		27			21	39
24				15		26					41		39		32		36					50		24
25											38		40	42	48			74	36					
26											14		20	57	32	55		86						
27										29	73				16	40		17						
28													20						14					
29													40						18			13		
30								54			16		39							45				
31													20											
TOTAL		182	36	200	78	303	108	314	128	117	393		724	178	324	319	441	266	229	145	255	126	500	197

TOTAL by Sea 3216

TOTAL by Land 2347

TABLE F-3 NUMBER OF DOLOS PLACED BY DAY IN 1976

Day	January		February		March		April		May		June		July		August		September		October		November		December	
	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land
1								27					54						18					
2													36		18				36					
3	34								16				12		54									
4	16								48	25	48	39			52	21			46					
5	68			19					32	15	48					31					36			
6													36								15			
7									48		25		54	42	4			16	54					
8									48	32	32	68	30				126		54			33		
9		40				19				26				52	68				14			34		
10										36			32		50									20
11									32		30	68				18							72	27
12									48			20	56			54							36	16
13									16						18									25
14									16			59	36	57	54		126	22						
15									14	81				47							36			10
16			12	25							32				48				36		36			
17		60	12			26							54								36		16	
18			12			13												24	18		11			
19		70	11			5					16	37			18	21			18					
20			11						48	21	32	16	54		54		36			32	51			39
21		112	11						48		31		12	41			88		18	42	36			
22		19	11	58		35		43	32	14	16		36		36	24		27		13	54			
23		8	11	69		32							54		12	35	10	23			36			
24		16	11	22		35					48		30	15				71			36			
25						3					54	30			54	61		31						
26							32				12	5		45	54	85					72			
27				13			17		48						14									24
28				15					70				36							47				
29						32	29					43	36				18							
30							16				18		35	18		22		17			36			
31													17			85								
TOTAL	118	325	102	221		199	94	70	564	250	442	385	710	317	608	457	404	231	312	134	491	67	124	161

TOTAL by Sea 3969

TOTAL by Land 2817

TABLE F-4 NUMBER OF DOLOS PLACED BY DAY IN 1977

Day	January		February		March		April		May		June		July		August		September		October		November		December	
	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land
1				32									21	2		18		56		9				
2									54		54					54		48						
3				24		24			17		71				18	36		27						
4				11		43			79				35		18	17				10				
5				19					71				36	15	16	40		27						
6									71		18		18	4	33	18		30		20				
7				10		35					18		53					34						
8											36		36		26	18		20						
9									36				36		18	36		33						
10									17						36	18		3		16				
11									36				54		52					20				
12									54				36			13								
13											54		36		34	10								
14									72		36		54									4		
15											54		90					27				22		
16							30	13			72	9	36	17	36	18		33				12		
17											36	20			54	31						14		
18											18	36	72	66	54	34						20		
19											18	34	59	54	18	10		8				25		
20							72	22	36	21			36	46	36	5								
21							18	5	36	54	18	31	94									25		
22							103	17		48	34	19	30		18	56						12		
23				20			18		53		18	45				18		22				33		
24				23			54	100	72	25	18	64				7						24		
25				17					49	86				19								21		
26				41				19	103	45						25		29				19		
27								28	34	46						38		12						
28								45		22												4		
29							16	26				33	36			15								
30							18							25		32								
31																34								
TOTAL				197		102	329	275	890	347	573	291	869	248	467	601		409		75		235		

TOTAL by Sea 3128

TOTAL by Land 2780

TABLE F-5 NUMBER OF DOLOS PLACED BY DAY IN 1978

Day	January		February		March		April		May		June		July		August		September		October		November		December	
	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land
1																								
2																								
3																								
4			14																					
5																								
6																								
7																								
8																								
9																								
10																								
11																								
12																								
13																								
14																								
15																								
16																								
17																								
18																								
19																								
20																								
21																								
22																								
23																								
24																								
25																								
26	14																							
27	28																							
28																								
29																								
30																								
31																								
TOTAL	42		14																					

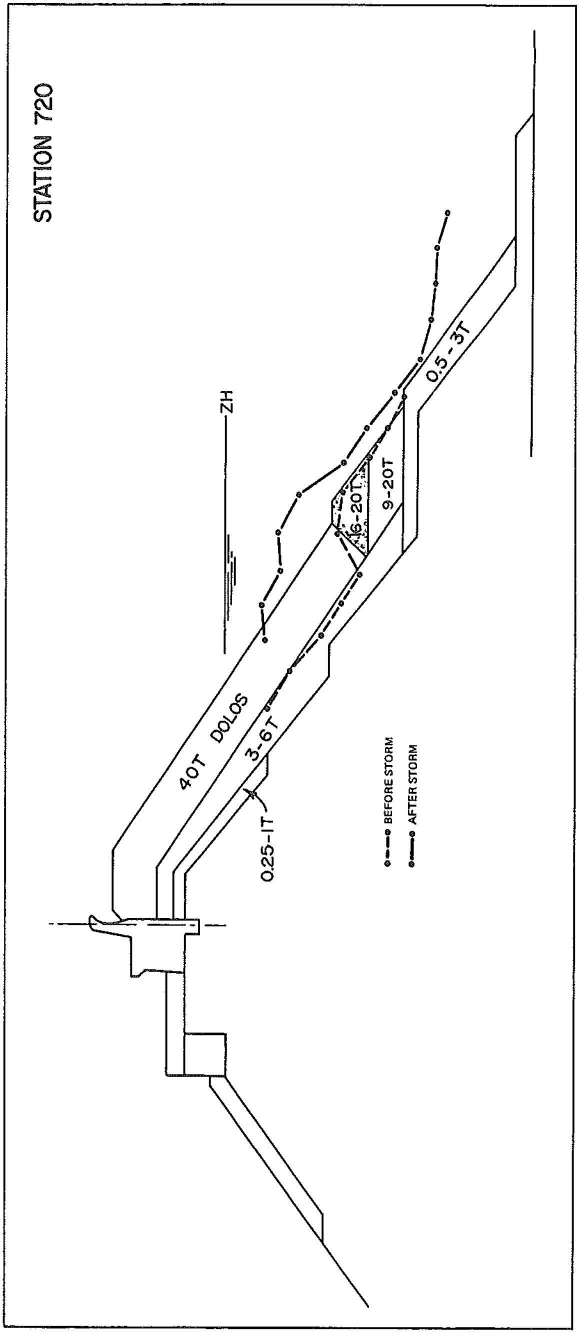
TOTAL by Sea 56

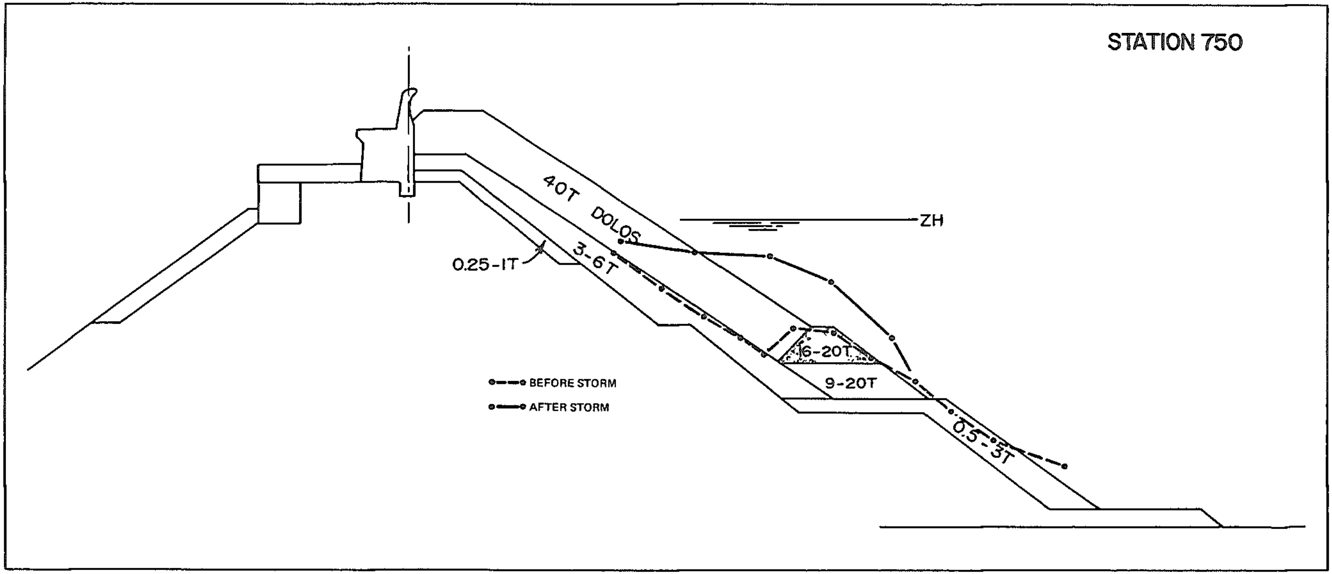
TOTAL by Land

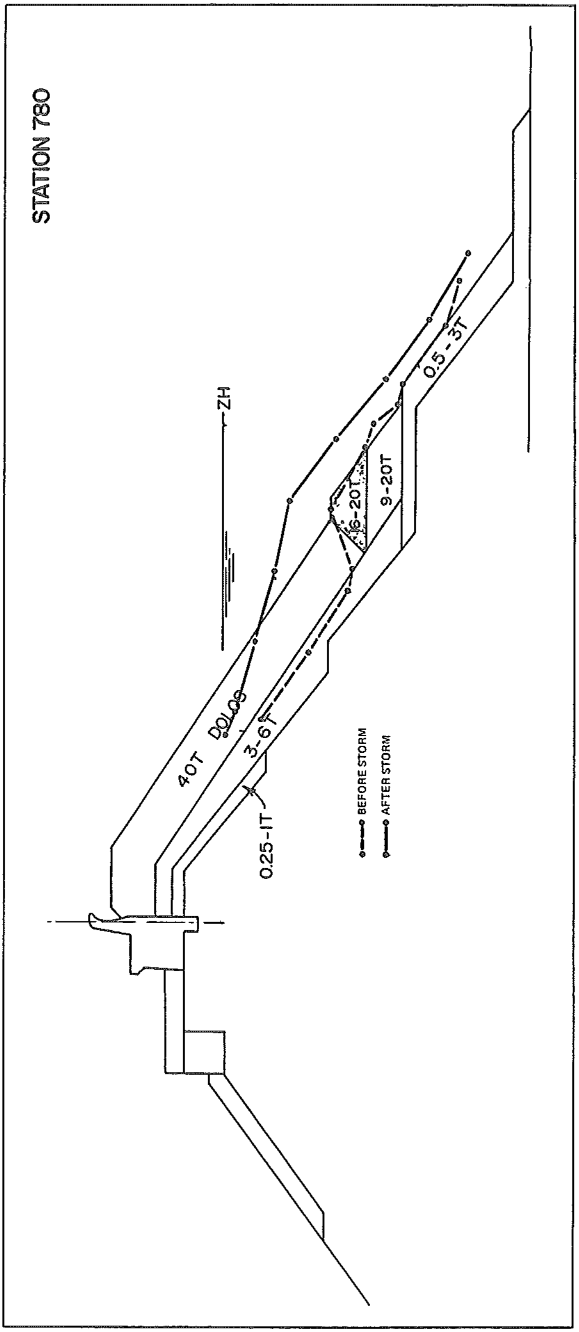
APPENDIX G

Cross Sections of Breakwater Before and After February Storm

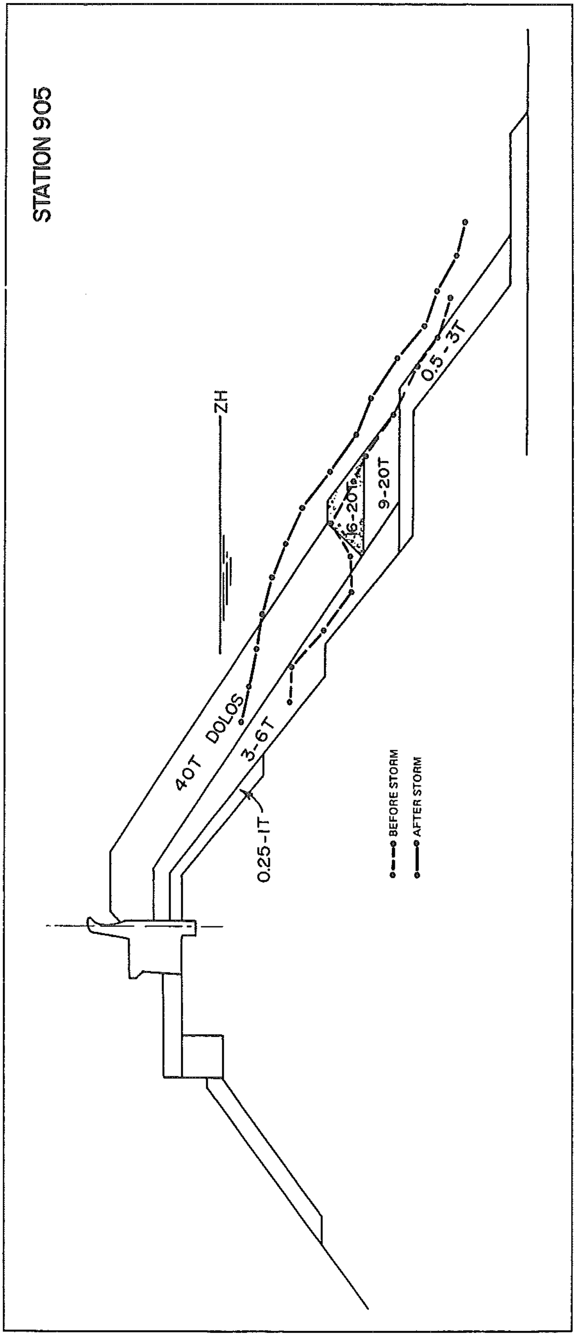
Note that only the first underlayer was profiled before the storm; nearly all dolos were in place prior to the storm, but profiles were not available.

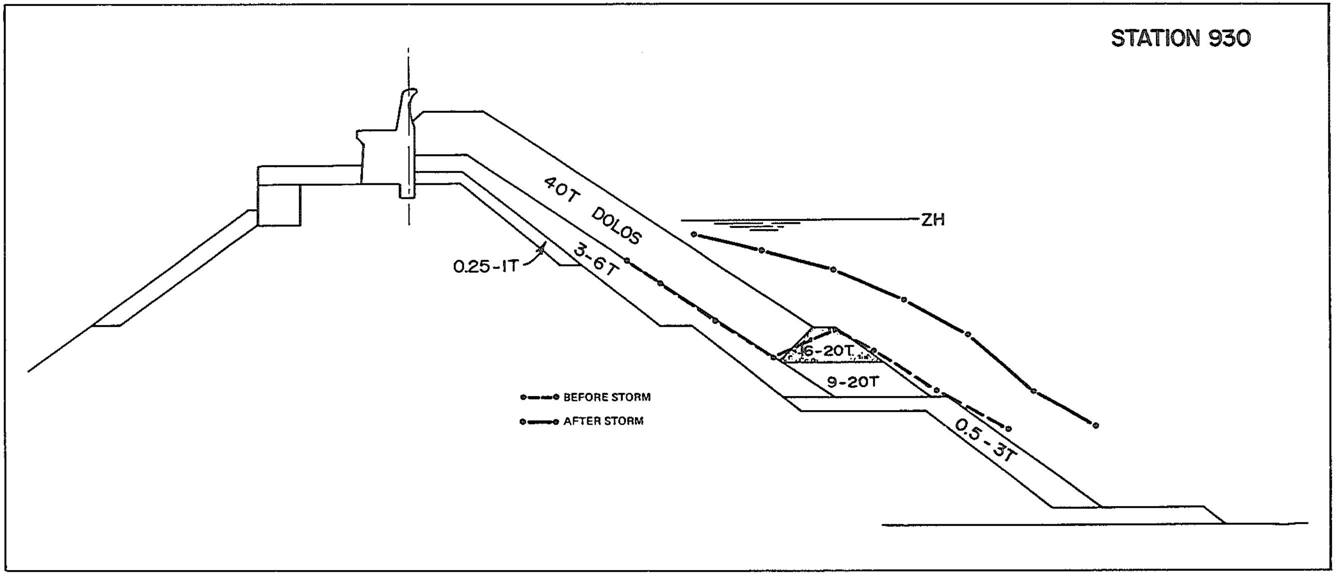

STATION 720
ZH
40T DOLOS
3-6T
0.25-1T
16-20T
9-20T
0.5-3T
BEFORE STORM
AFTER STORM


STATION 750
40T DOLOS
ZH
3-6T
0.25-IT
6-20T
9-20T
0.5-3T
BEFORE STORM
AFTER STORM



STATION 780
40T DOLOS
3-6T
0.25-1T
16-20T
9-20T
0.5 - 3T
BEFORE STORM
AFTER STORM


STATION 905
ZH
40T DOLOS
3-6T
16-20T
9-20T
0.5 - 3T
0.25-1T
BEFORE STORM
AFTER STORM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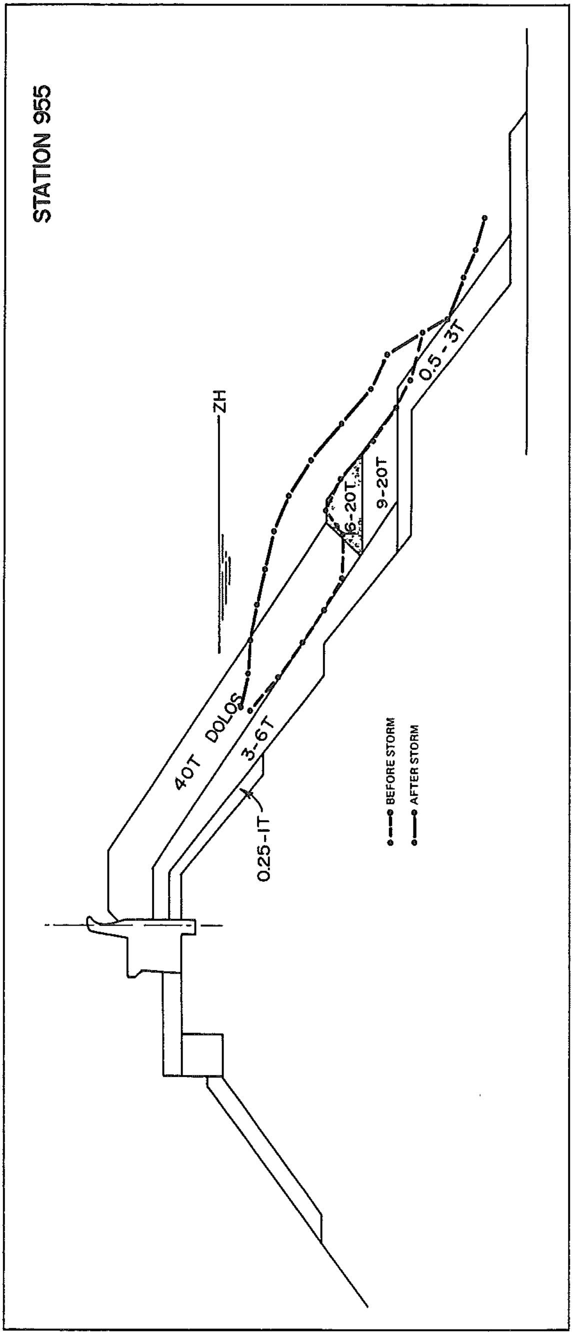
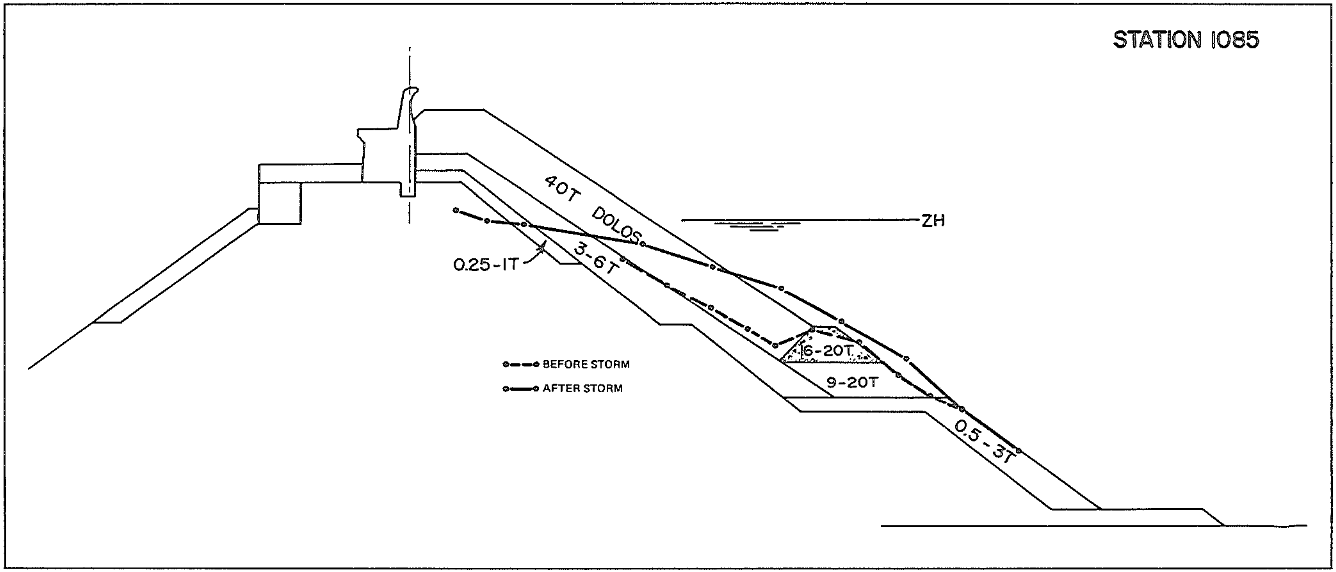
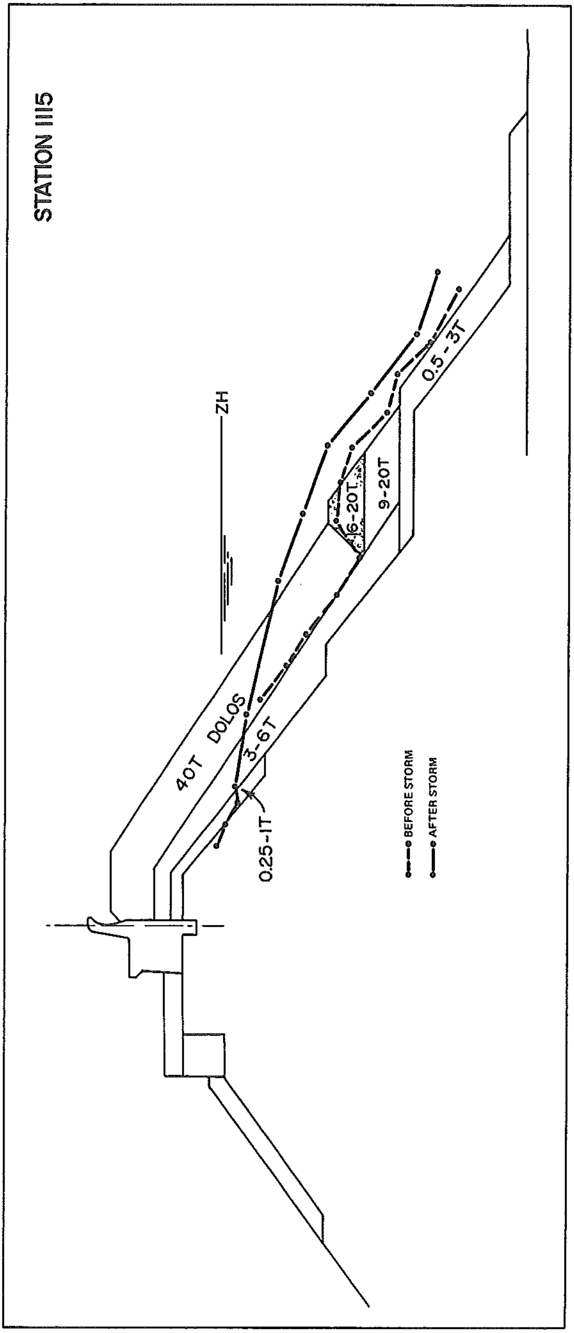
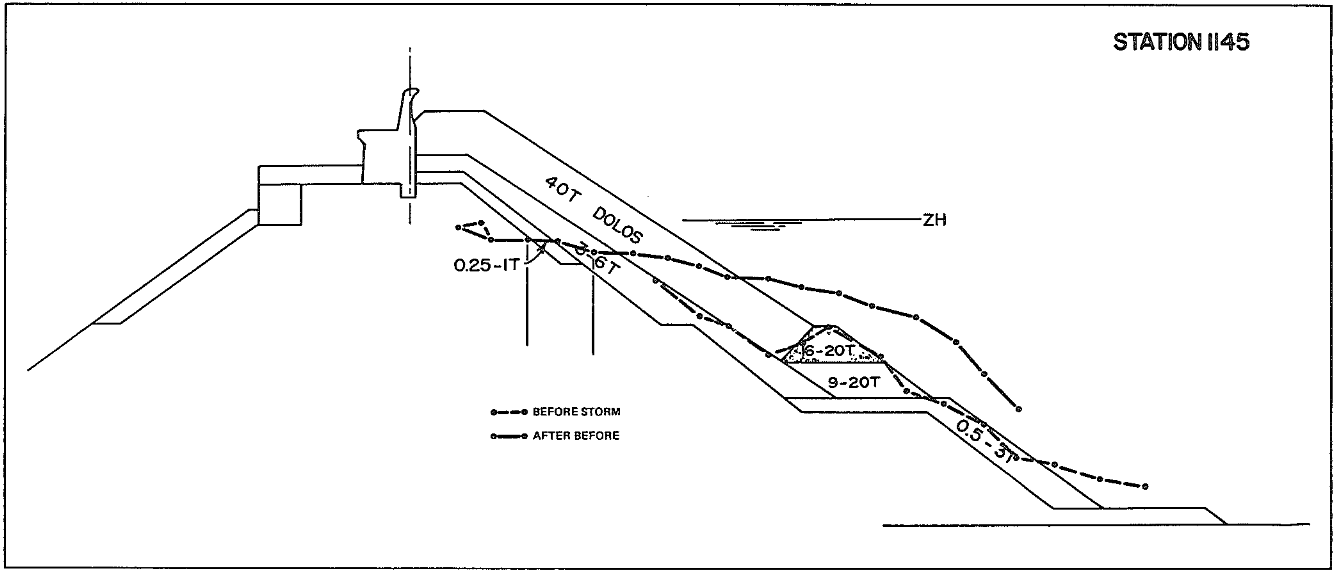
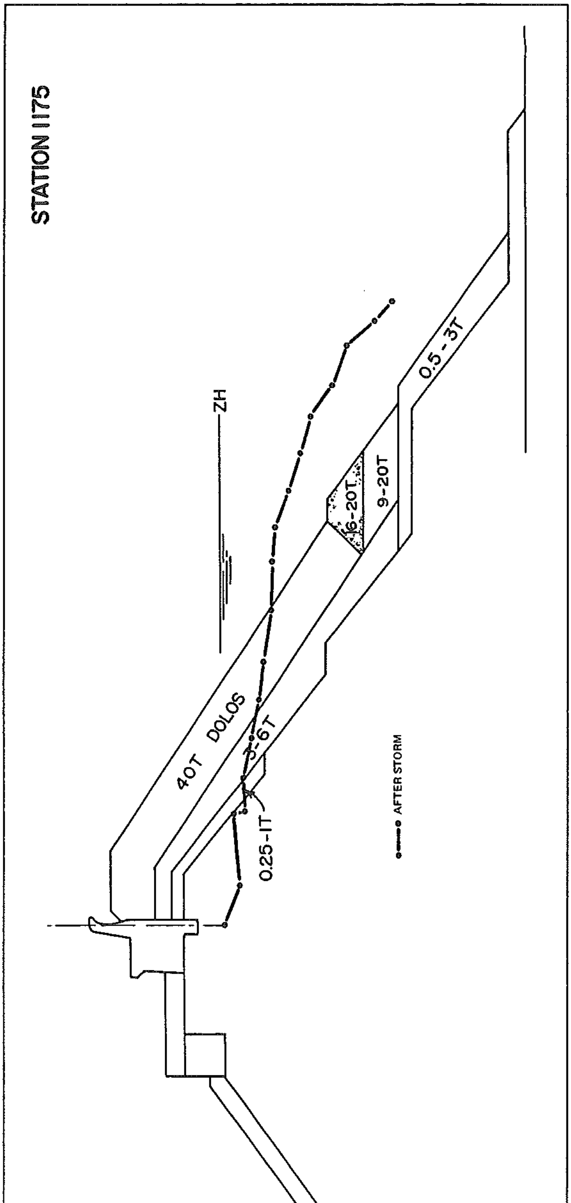
ZH
40T DOLOS
3-6T
0.25-1T
6-20T
9-20T
0.5 - 3T
BEFORE STORM
AFTER STORM

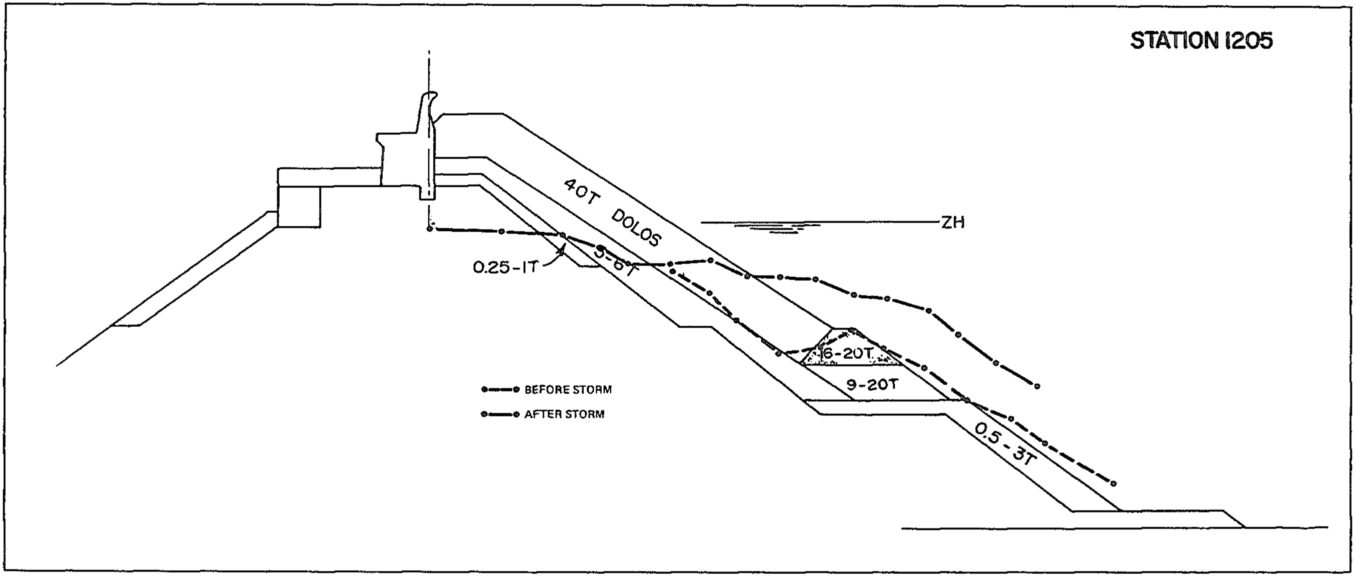

STATION 955
40T DOLOS
3-6T
0.25-1T
6-20T
9-20T
0.5-3T
BEFORE STORM
AFTER STORM

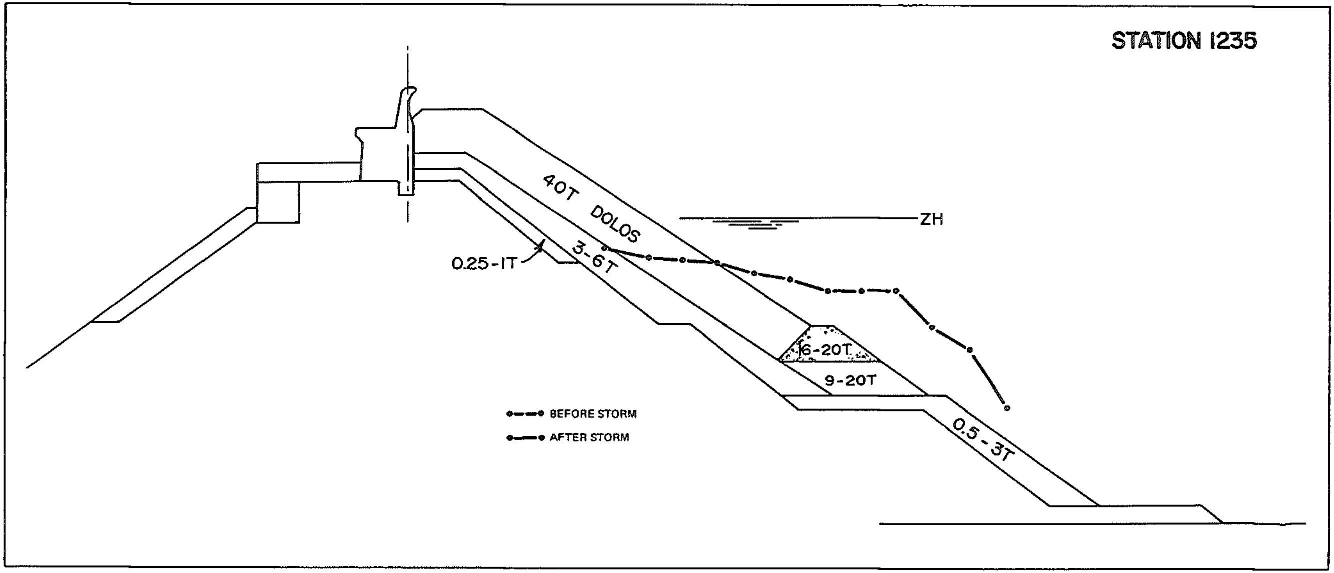

STATION 1085
40T DOLOS
ZH
3-6T
0.25-1T
6-20T
9-20T
0.5-3T
BEFORE STORM
AFTER STORM

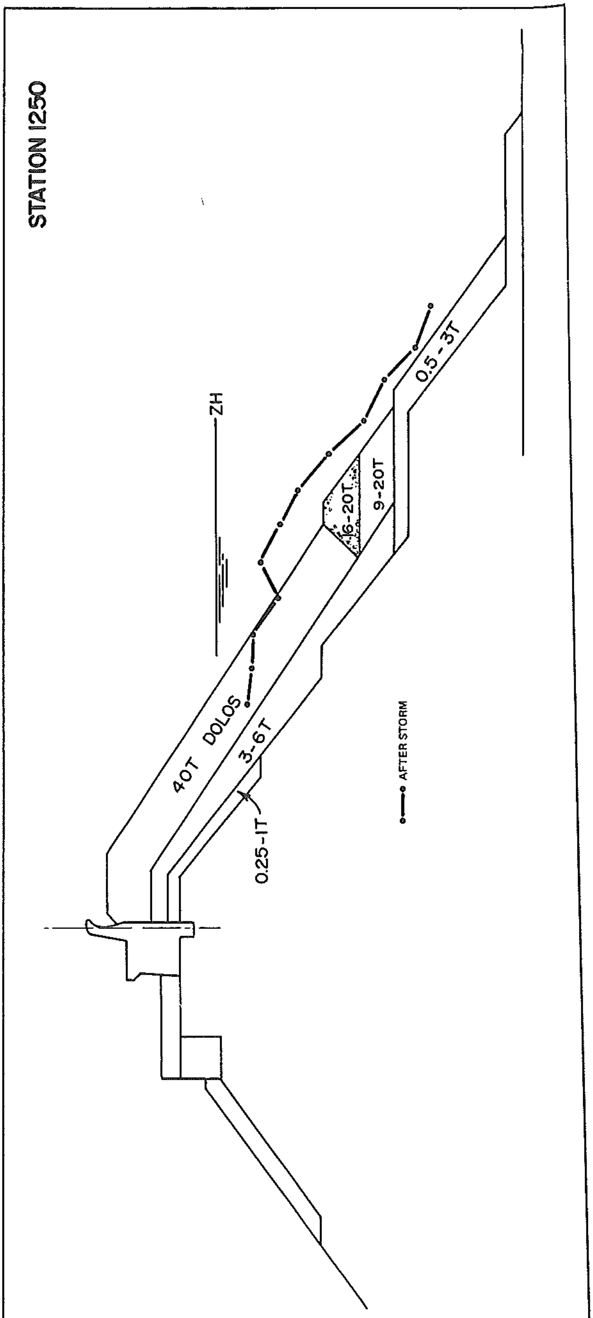

STATION 1115
ZH
40T DOLOS
3-6T
0.25-1T
16-20T
9-20T
0.5 - 3T
BEFORE STORM
AFTER STORM

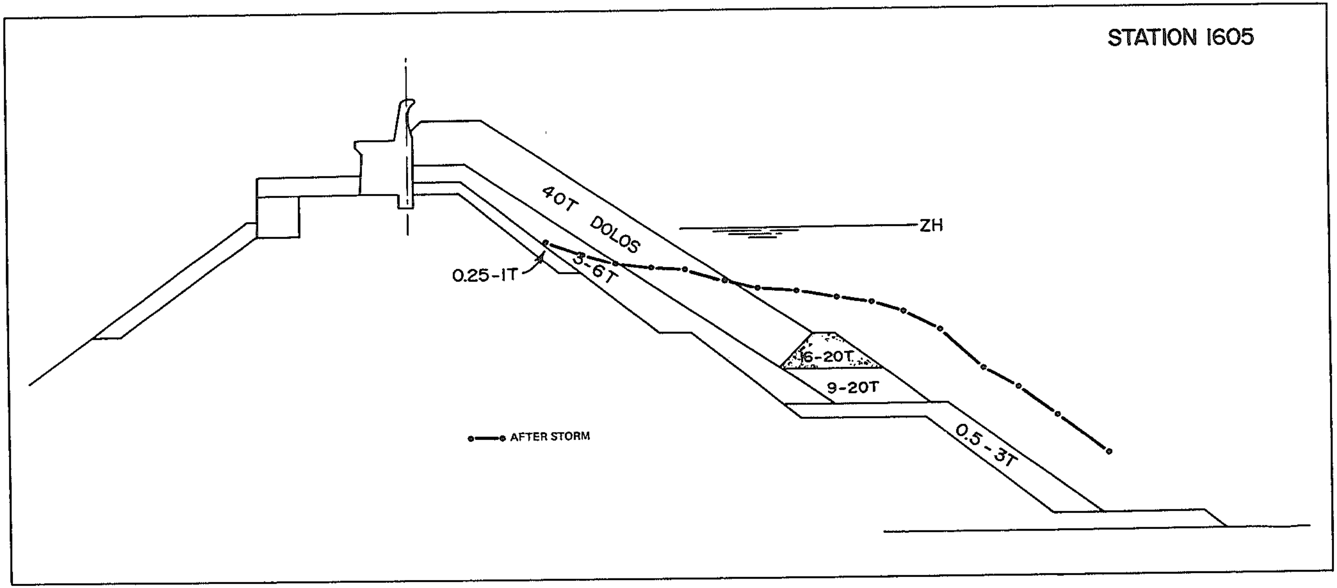

STATION 1145
40T DOLOS
ZH
0.25-1T
3-6T
6-20T
9-20T
0.5-3T
BEFORE STORM
AFTER BEFORE

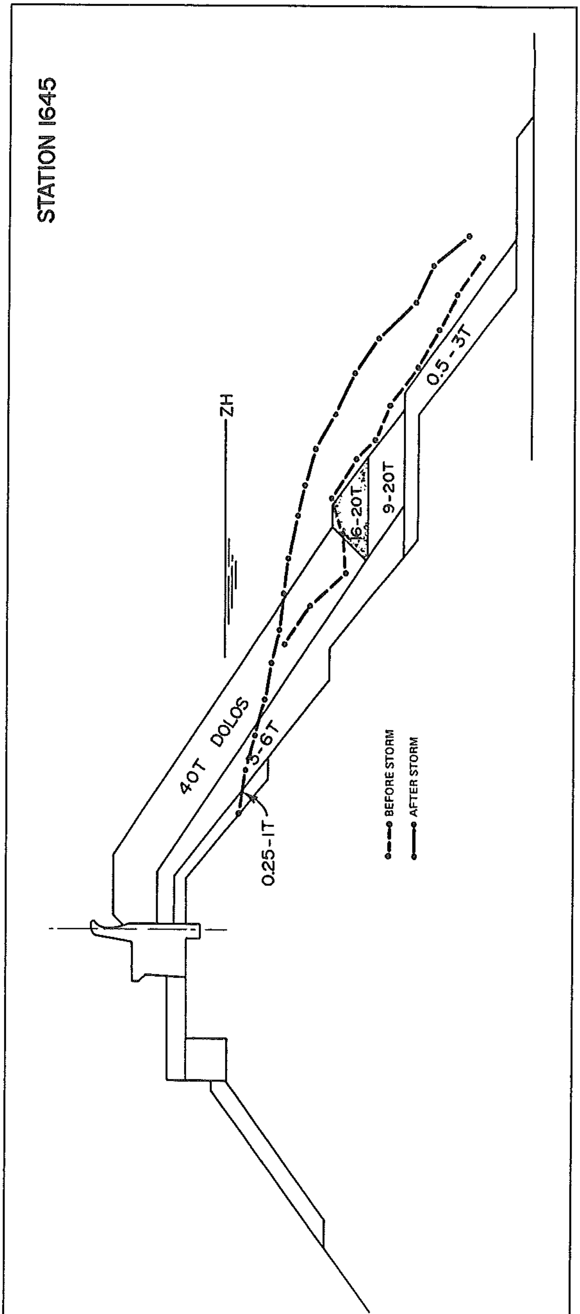

STATION 1175
ZH
40T DOLOS
3-6T
0.25-1T
16-20T
9-20T
0.5 - 3T
AFTER STORM

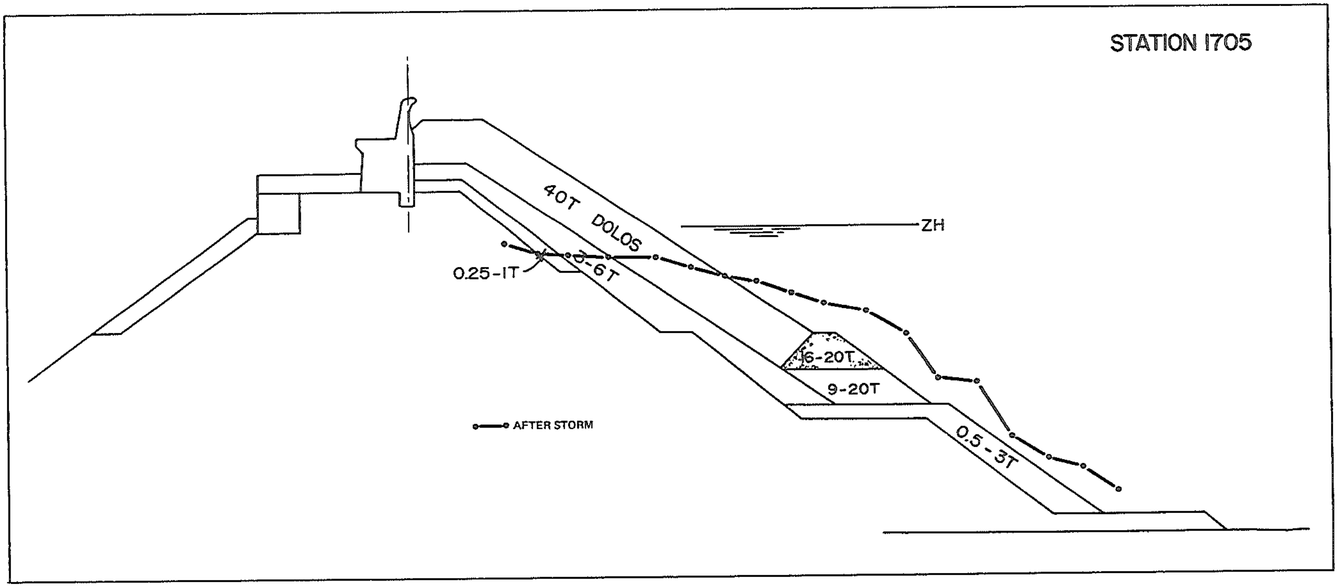

STATION 1205
ZH
40T DOLOS
5-6T
0.25-1T
6-20T
9-20T
0.5-3T
BEFORE STORM
AFTER STORM


STATION 1235
ZH
40T DOLOS
3-6T
0.25-1T
6-20T
9-20T
0.5 - 3T
BEFORE STORM
AFTER STORM


STATION 1250
40T DOLOS
3-6T
0.25-1T
16-20T
9-20T
0.5 - 3T
AFTER STORM


STATION 1605
40T DOLOS
ZH
3-6T
0.25-1T
16-20T
9-20T
0.5 - 3T
AFTER STORM


STATION 1645
ZH
40T DOLOS
3-6T
0.25-1T
16-20T
9-20T
0.5 - 3T
BEFORE STORM
AFTER STORM


STATION 1705
40T DOLOS
ZH
3-6T
0.25-1T
16-20T
9-20T
0.5 - 3T
AFTER STORM

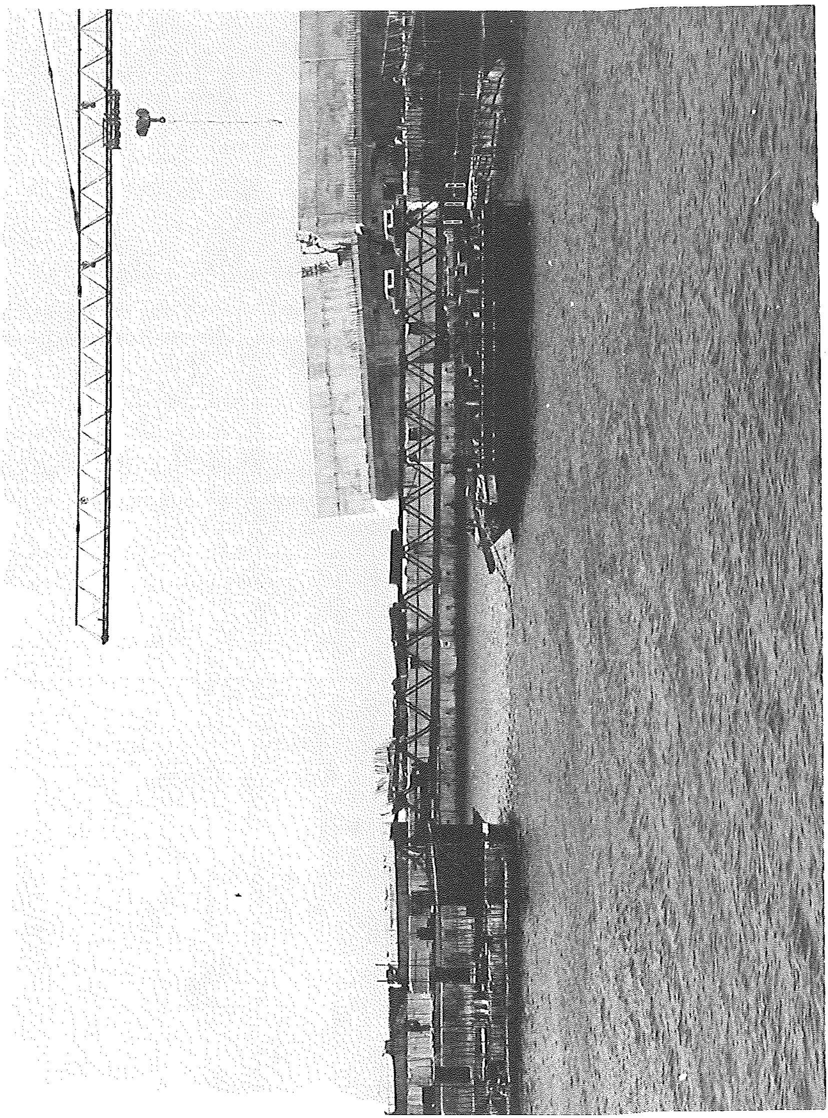
APPENDIX H

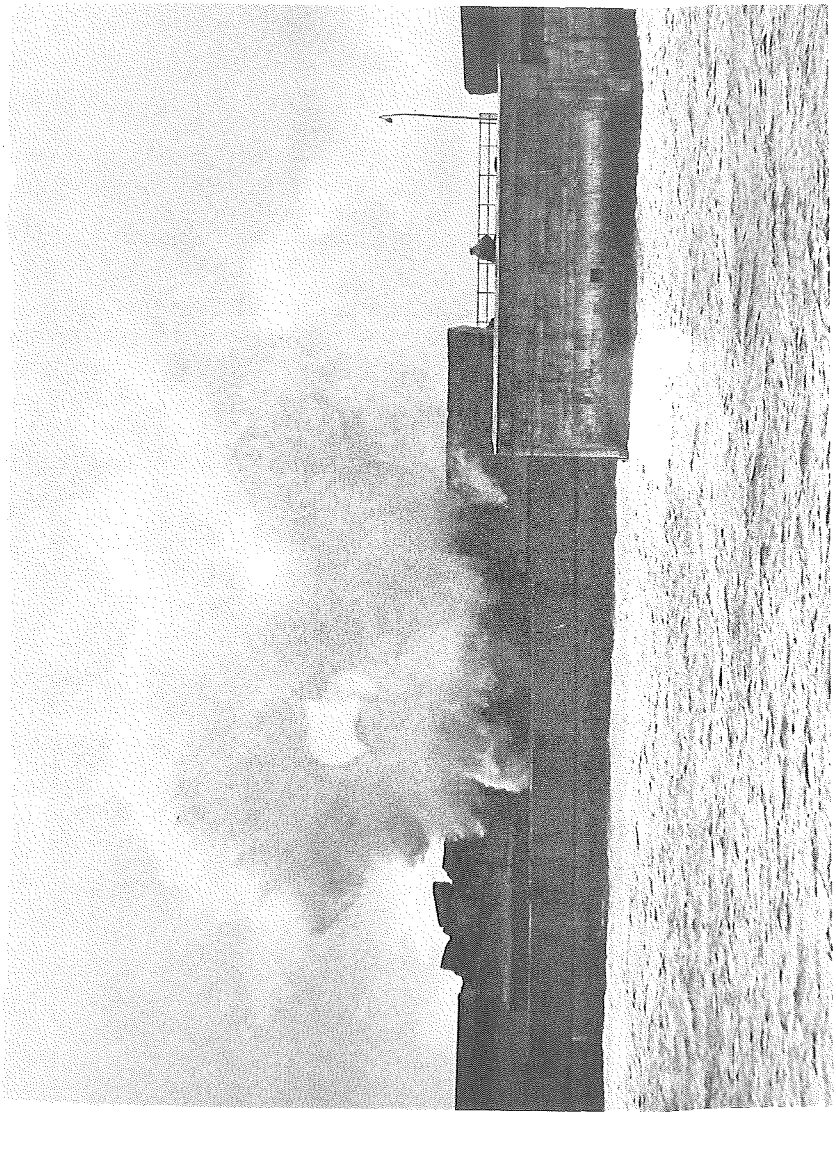
Photographs Showing Damage of Breakwater After February Storm

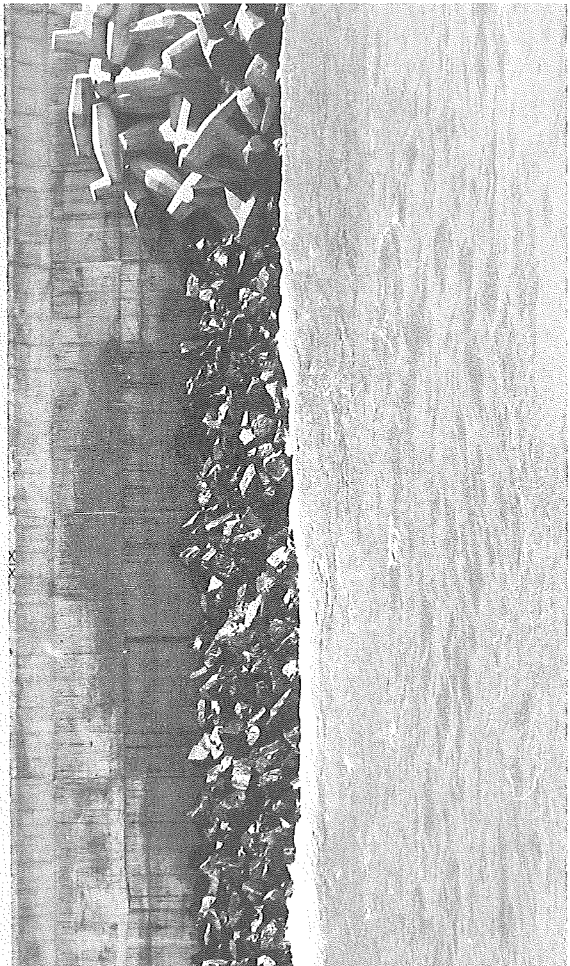
Photographs were provided by:

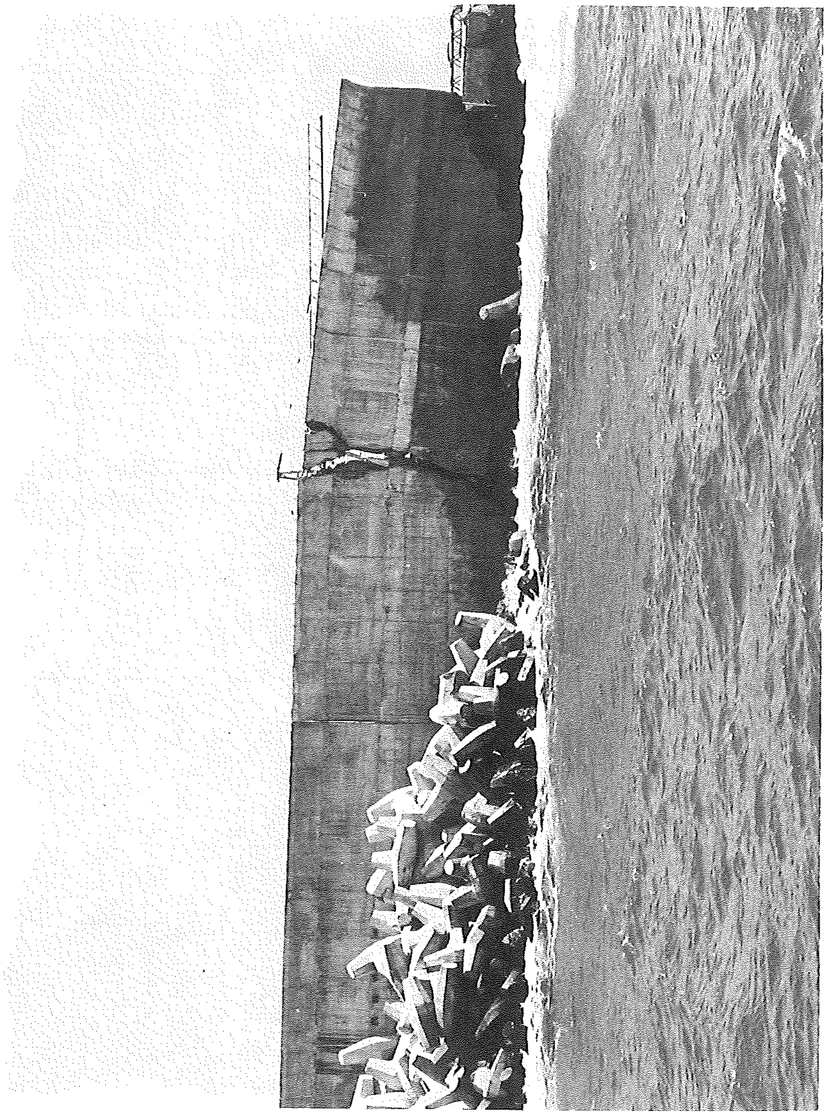
Gabinete da Area de Sines
Bertlin & Partners, U.K.
Virginia Fairweather
William F. Baird

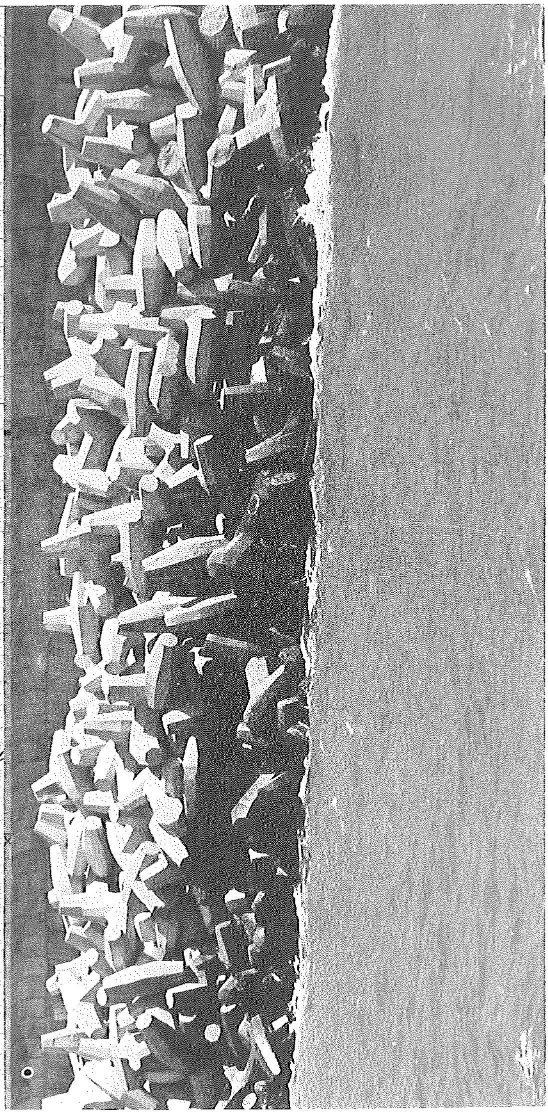
The photographs on pages 226-234 were taken on or before March 10, 1978. Those on pages 235-237 were taken subsequently.

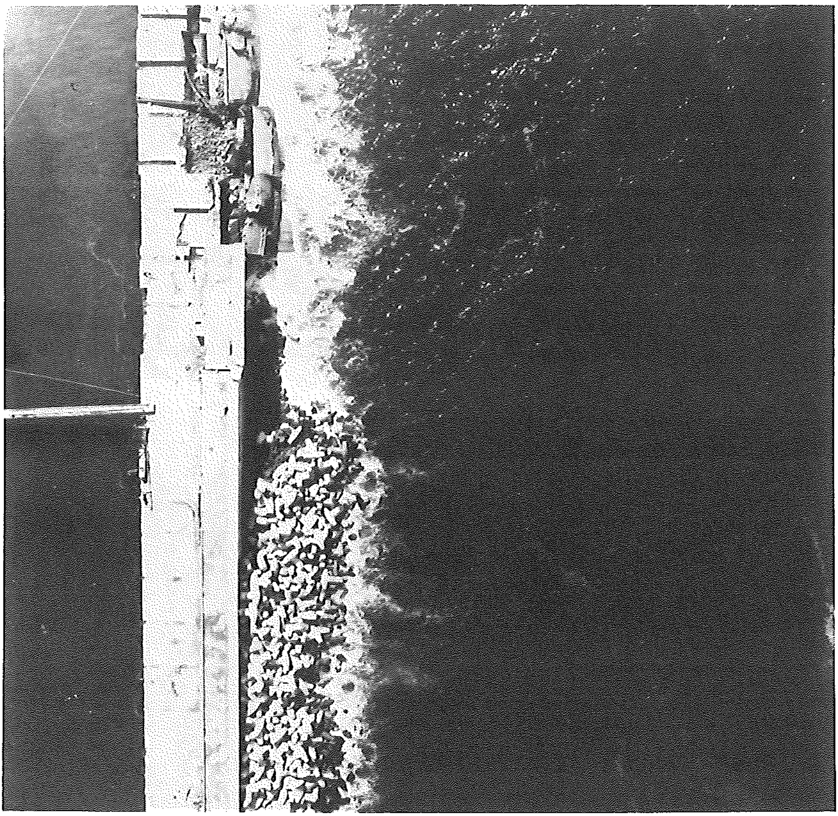


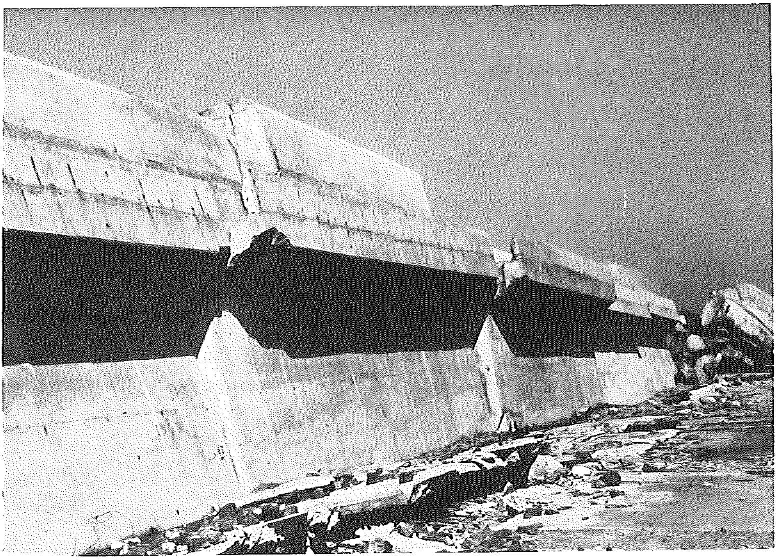












APPENDIX I

Listing of Documents Available to the Panel

Listing of Documents Available to the Panel

Bertlin and Partners, "Summary of Calculations from Geophysical Investigations," internal report, undated.

Bertlin, Consulmar, and Lusotecna, "Reposta A Comissao de Avaliacao do Acidente do Molhe Oeste de Sines," "Reply to the Investigation Committee of the Sines West Breakwater Accident," reports (in Portuguese and English) submitted in reply to the Commission of Inquiry on the Accident to the West Mole at Sines, April, 1978.

Bertlin, Consulmar, and Lusotecna, "Concurso de Construcao Civil -- 1.ª fase, Volume 4, Anexo 1 -- Condicoes Naturais," "Course of Civil Construction, Natural Condition and Interior Waves," report prepared for GAS, 1972

Bertlin, Consulmar, and Lusotecna, "English Translation of the Reply to the Investigation Committee of the Sines West Breakwater Accident," translated by R.P. Yates, May 1978.

Bertlin, Consulmar, and Lusotecna, "Clausulas Tecnicas Gerais: 2.4 Enrocamentos," also English translation "General Technical Clauses: 2.4 Armoring," by R.P. Yates, from construction bid document prepared for GAS, undated.

Bertlin, Consulmar, and Lusotecna, "Clausulas Tecnias Especiais: 2.4 Encrocamentos," also English translation "Special Technical Clauses: 2.4 Armoring," by R.P. Yates, from construction bid document prepared for GAS, undated.

Gabinet da Area de Sines, "Sines Harbour: Study of Alternative Solutions, Volume III, Basic Factors," report prepared for Gabinet da Area de Sines, undated.

Gabinet da Area de Sines, "Sines Development Area," brochure prepared for Gabinet da Area de Sines, undated.

Gabinet da Area de Sines, "Producao da Pedreira--1976 - 1978," "Excavation of Quarry Stone by Size," internal report, undated.

Gabinet da Area de Sines, "Record of Damage: Superstructure on Breakwater, Affected Zones from the Storm, Zones of Affected Superstructure," internal report, 3 Charts, undated.

Gabinet da Area de Sines, "Post Storm Profiles of the Breakwater with Diver Annotations," 16 Charts, internal report, undated.

Gabinet da Area de Sines, "Porto de Sines, Molhe Oeste, Vista Frontal da Superstrutura," one drawing showing elevations along the superstructure after the storm, Drawing No. DPA 189A/78, April, 1978.

Laboratoire Central d'Hydraulique de France, "L'estimation des caracteristiques de la houle au large du port," "Estimation of the Characteristics of the large storm at the Port," report No. 1 for Societa Italiana per Condotte d'Acqua, August, 1978.

Laboratoire Central d'Hydraulique de France, "Conditions de propogation de la houle entre le large et la dique: Etude sur modele mathematique," "Study of the refraction of long period waves: mathematical model," report No. 2 for Societa Italiana per Condotte d' Acqua, August 1978.

Laboratoire Central d'Hydraulique de France, "Analyse du comportenent des dolosses dans un champs hydrodynamique: Etudes in situ et sur modele reduit," "Analysis of the Hydrodynamic effects on a Layer of dolos," report #4 for Societa Italiana per Condotte d'Acqua, November 1978.

Laboratoire Central d'Hydraulique de France, "Conditions de propogation de la houle entre le large et la dique: Etude sur modele physique," "Study of the refraction of long period waves: physical model," report No. 5 for Societa Italiano per Condotte d' Acqua, November, 1978.

Laboratorio Nacional de Eugenharia Civil, "Comportamento do Mohle Oeste do Porto de Sines Durante a Tempestade de 20/2/78 a 4/3/78: Ensaios de Estabilidade em Canal de Ondas Irregulares," "Behavior of the Breakwater of Port of Sines During the Storm from 2/20/78 to 3/4/78: Study on Establishing Profile with Irregular Waves," technical notes, June, 1978.

Laboratorio Nacional de Engenharia Civil, "Comportanento do Mohle Oeste do Porto de Sines Durante o Temporal de 20/2/78 a 4/3/78 Ensaios em modelo reduzido do perfil de reconstrucao de emergencia," "Behavior of the Breakwater of Port of Sines during the Storm from 20/2/78 to 4/3/78," Emergency reconstruction of profile, Technical notes, July, 1978.

Laboratorio Nacional de Engenharia Civil, "Performance of West Breakwater of Port Sines During Storms of 20/2 to 4/3/78 - Processing of Wave Data," Translated by R.P. Yates, A report prepared by C.A.R. Pita of LNEC for GAS, March 1978.

Larras J., Dubois J., Couprie, P., "Dommages Subis Par La Dique Principale Au Cours De La Tempete Des 23 - 28 Fevrier 1978," "Principal causes of failure of the breakwater from the storm of February 23 - 28, 1978," expert report, undated.

Mansard, E.P.D. and Ploeg, J., "Model Tests of Sines Breakwater," National Research Council of Canada Report No. LTR-HY-67, August 1978.

Moore, William W., "Marine Site Investigations, Breakwater and Berthing Facilities for Proposed Port of Sines, Portugal," report prepared by Dames & Moore for the Gabinete da Area de Sines, 1974.

Paolella, Giuseppe and Agostini, Romano, "Impiego dei dolos per il porto oceanico di Sines in Portogalla," "Construction with dolos for the ocean port of Sines, Portugal," journal published by Estratto da "L'Industria Italiana del Cemento," February, 1978.

Rodeia, J., "Novo Porto de Sines, Empreitada 7/76, Configuracao dos Fundos Nas Aproximacoes da Area Alrigada (176)," "Bathymetry of Port de Sines - Measured July, 1976," prepared for GAS, one sheet, scale 1:10,000, May, 1978.

Societa Italiana per Condotte d'Acqua, "Construcao Civil - Primeira Fase: Mohle Oeste Processo Constructivo da Superestrutura," "Civil Construction -First Phase: Construction Procedures for the Superstructure," bid document for GAS, undated.

Societa Italiana per Condotte d'Acqua, "Construcao Civil - Primeira Fase: Mohle Oeste Processo Constructivo da superestructura," "Civil Construction -First Phase: Construction Procedures for the Superstructure," bid document for GAS, one drawing, undated.

Societa Italiana per Condotte d'Acqua, "Construcao Civil - Primeira Fase: Mohle Oeste Cabaca Perfis Anexo ao Processo Constructivo," "Civil Construction -First Phase: Description of Construction by Profile," bid document for GAS, one drawing, May, 1977.

Societa Italiana per Condotte d'Acqua, "Construcao Civil - Primeira Fase: Mohle Oeste Processo Constructivo," "Civil Construction - First Phase: Proposed Construction Procedures," bid document consisting of one drawing to GAS, August, 1976.

Societa Italiana per Condotte d'Acqua "Construcao Civil - Primeira Fase: Mohle Oeste Processo Constructivo," "Civil Construction - First Phase: Proposed Construction Procedures," bid document for GAS, undated.

APPENDIX J

State-of-the-Art of Rubble Mound Breakwater Design

by

William F. Baird

July 1979

(Prepared at the request of the

Port Sines Investigating Panel)

State-of-the-Art of Rubble Mound Breakwater Design

Rubble-mound breakwaters have been a proven method of providing protection in coastal areas subjected to wind-generated waves. The engineering literature on the design of these structures is voluminous and readily available. American standard rubble-mound breakwater design is predicated on the "Hudson Formula," which was derived from observations of hydraulic models using small pieces of stone and laboratory facilities at the US Army Waterways Experiment Station in Vicksburg, Mississippi.

As breakwater designs were required for deeper waters and more severe wave conditions, the use of quarry stone became impractical because of the large stone sizes and flat slopes that would be required to meet design conditions. Numerous concrete shapes from simple tetrahedrons to complex and intricate shapes have been promoted over the years as solutions to deepwater conditions with high wave action. Unfortunately, the initial constraints which led Hudson to his design equation were ignored in the new generation of testing which followed the introduction of the new concrete armor shapes. Hudson (1958) noted in the report on his research that

> "For breakwaters constructed by dumping or by placing armor units essentially pellmell, the forces resisting displacement are the bouyant weight of the individual units and the friction between units. Except for isolated instances where wedging action is involved, friction between armor units can be neglected."

Hudson's model work was conducted with concrete armor units which would not break in the model and also with essentially monochromatic waves of relatively uniform height. Inasmuch as most rubble-mound structures being designed and constructed about the time Hudson presented his material and immediately thereafter were limited by depth conditions offshore, Hudson's constraint of essentially monochromatic waves of constant wave height did not appear important. Subsequently, as these structures have been designed for deeper and deeper water, greater recognition has been given to the wave spectrum and most recently to wave groups.

Nevertheless, the basic cross section of the breakwater has not changed significantly. As breakwaters are built in deeper water and more exposed locations, the size of stone required becomes considerable; attention therefore has been necessarily given to artificial armor units. As noted by Magoon and Shimuzu (1971), the requirements of handling, placing and using large concrete armor units has required the consideration of stresses created in those units. Further consideration of those stresses can and on occasion has led to reinforcement of the concrete armor units. The arguments for and against reinforcement of large concrete armor units are many and generally site-specific. The question of reinforcement has become somewhat polarized with the majority of the engineering practice indicating that concrete armor units should be unreinforced. (The reasons why the dolos at Sines are unreinforced, as stated by John Mettam, are given in Appendix E.)

Design Approach

During the design process for rubble mound structures, decisions are made concerning the following details:

(1) The size, weight, and density of units in the armor layer protecting the exposed face of the breakwater.

(2) The slope of the armor layer and its extent above and below the water level.

(3) The stone sizes making up the filter layers and the extent of these layers.

(4) The requirement for a mass concrete superstructure along the top of the breakwater.

(5) The armor protection required on the harborside of the breakwater.

(6) The size of armor units, slope, etc., required at the head or seaward end of the breakwater.

(7) The elevation of the top of the structure.

For a major breakwater, these decisions will be made by engineers using equations relating the design wave parameters to armor unit sizes, run-up, etc., and scale models of breakwaters exposed to wave action. The selection of the design wave parameters is, of course, a very critical step.

Adequate design of the armor layer is of paramount importance to the integrity of the breakwater. If the armor layer is removed by wave action, the breakwater will be destroyed. The following discussion is concerned with the design of the armor layer.

Theoretical Approach

A number of equations have been proposed that equate the size of an armor unit required to protect a rubble mound breakwater to the incident wave conditions. Such equations have been summarized, for example, by PIANC (1977). These equations generally relate wave parameters to the size of an armor unit that will not be displaced from the armor layer. The variables of these equations include the following:

- wave height parameter
- wave period parameter
- wave length parameter
- weight of armor unit
- slope of breakwater
- unit weight of armor unit
- unit weight of water
- stability coefficient of the armor unit
- coefficient of friction

The equation most frequently used in North America for the preliminary design of rubble mound breakwaters is that proposed by Hudson. Hudson's equation was developed for use with quarried stones, and results from equating the drag force of the uprushing wave on a stone with the weight of the stone. The equation is as follows:

$$W = \frac{W_r \; H^3}{K_D \; (S_r - 1)^3 \cot \Theta}$$

Where W = weight of unit

H = wave height

K_D = damage or stability coefficient

W_r = unit weight of unit

S_r = specific gravity of armor unit

cot Θ = slope of breakwater

The stability coefficient, K_D, is determined from data derived from previous hydraulic model studies and from site-specific studies.

Use of Hudson's Equation

The use of Hudson's equations in breakwater design involves decisions regarding the following factors:

(A) The wave height parameter:

- o the return period of the design storm,
- o the effects of bathemetry in front of the breakwater in increasing or decreasing the deep water waves,
- o the wave height parameter to be used in the equation (H_{max}, $H_{1/3}$, etc.) This will depend on the wave height parameter used in the model studies from which the stability coefficient was obtained.

(B) Damage:

The damage that will occur during the design storm(s) considered in (A). This will also depend on the definition of damage used in the model studies from which the stability coefficient was obtained.

Limitations of Hudson's Equation

A limitation of Hudson's equation is that the drag force is assumed to be independent of wave period. It is likely that the stability of most armor units is dependent on wave period, wave steepness, the offshore parameter $\tan\alpha/(H/L_o)^{\frac{1}{2}}$ and the way in which the wave breaks on the breakwater. The equation was first proposed for the design of breakwaters using quarried stones and where interlocking between adjacent units was not a major consideration. The equation has, however, been extensively used to describe the effectiveness of dolos units, which are characterized by their interlocking characteristics.

The stability coefficient and the relationship between the damage to the armor layer and the wave height is entirely dependent on model studies. The limitations of these model studies therefore impose limitations on the use of the equation. Limitations of model studies are discussed in the following section.

Model Studies

Model studies are an essential component of the design procedure for large breakwater structures. A correctly designed model study will provide the most complete information to the designer.

Many laboratories throughout the world have undertaken and published results of model tests on rubble mound breakwaters. The results of these different studies cannot be directly compared because of the difference in the procedures followed in the tests. Some of the differences are as follows:

(1) Individual armor units (concrete).

Differences in:

- surface friction of units
- dimensions of units
- unit weights of units
- strength of material of units

(2) Construction of the Model Breakwater.

In some studies construction of the model has followed prototype construction practices in detail. In other studies, however, individual units have been placed with high interlocking obtained by carefully locating the unit and pressing the unit into the armor layer. Model construction should follow the placement action of a crane. In some studies the breakwater is built in the dry and in others with water in the testing flume.

(3) Wave parameters.

The first model studies were made with sinusoidal waves of constant height and period. In recent years laboratories have used irregular waves produced either by the superposition of discrete sinusoidal components or by the filtering of a white noise signal provided to the wave generator. Today some laboratories are generating water surface profiles identical to surface profiles recorded in the sea. Such profiles may contain a series of large waves directly following each other.

Finally, laboratories have also produced waves by direct wind generation in the flume. The wind may produce a realistic wave steepness as well as a realistic distribution of heights and periods. The creation of waves via wind may require different similitude relationships. Studies show that the different wave profiles do produce different results on a rubble mound breakwater.

(4) Test procedures.

The scales at which model studies have been carried out have varied, as well as the duration of tests and the sequence of testing at different wave heights and periods. Further, the depths of water, the tidal ranges, the angles of wave attack, and the details of the structure such as the front slope and the structure height have varied. All affect the published results.

(5) Reporting of results.

A review of reports of model studies shows great variation in the reporting of the results of wave interaction with the armor units. Damage has been variously defined as the oscillation of individual units in the armor layer, the movement of units from one location to another in the armor layer, or the displacement of units from the armor layer. Damage is also described in terms of changes in profile or cross-section of the armor layer. Oscillation or rocking of individual units can be difficult to observe because it occurs immediately below the level of the breaking wave.

The amount of damage occurring has been expressed as percentages of all units or of units within defined areas about the mean water levels.

There has been little conformity in the reporting of model studies and as a result there are considerable variations in the reported values of stability coefficients.

The limitations of stability coefficients or model studies in general depend on how representative the construction procedures and wave conditions were of prototype conditions. Motions within the armor layer must be observed and accounted for when designing the individual units of the armor layer.

Design of Individual Armor Units

Individual units within the armor layer should be designed to withstand wave forces and any movements that may occur within the armor layer.

In general, the design of individual units has been based on experience rather than complete design analysis. Units have been designed to minimize stress simply by increasing the section dimension and eliminating stress concentrations rather than by determining values of stress and design the unit accordingly. Some concrete armor units have been designed with reinforcing steel. It has been generally assumed that the wave- or movement-induced loadings will not produce tensile stresses producing failure.

For large armor units the stresses induced by the weight of the unit itself will be significant, and tolerable additional loadings will be less than for lighter units.

Concrete armor units have not been designed following accepted concrete design practice results because the movements of units, the loadings imposed on units, and the resulting stress distributions are not known. For structures where large units are used and where the integral strength of individual units is critical to the survival of the armor layer, the units should be designed such that little or no movement occurs within the armor layer. Alternatively, consideration can be given to small in-place movements if reinforcement can be designed to accommodate the dynamic stresses involved.

A more detailed treatment of the analytical formulae for design of various armor types is given in the report of the International Commission for the Study of Waves (PIANC, 1976). The report is the standard reference for design of rubble mound structures in Europe. The discussion in this section is intended to supplement that given in the PIANC report.

APPENDIX K

REPORT ON DIMENSIONS OF DOLOS
TO BE USED ON MAIN BREAKWATER

A letter report from
Bertlin-Consulmar-Lusotechna
to Gabinete da Area de Sines,
dated 30 August 1973.

PORT OF SINES

REPORT ON DIMENSIONS

OF DOLOS TO BE USED ON MAIN BREAKWATER

Revised Version of 30th August 1973

1 - INTRODUCTION

1.1 - Dolos are artificial blocks of concrete for use in sloped breakwaters, and of very recent development, which offer the advantage over older forms of greater stability. The application of formulas for its calculation/design, of the type of the Hudson formula for other types of blocks, is still in the research and initial study stages for which recourse to model tests is the only available means for dimensioning breakwaters with such blocks.

1.2 - The summary tests completed in August 1973 by LNEC indicated that the 30-ton dolos specified in the ante-project could resist, with very little damage, waves of the significant height of 12 m corresponding to a probability of occurrence of once in 100 years.

1.3 - The very careful and extensive tests completed also by LNEC, and described in the Report of January 1973, led to a more pessimistic conclusion. However, the dolos used in these tests were of a lesser density than the phototype dolos, for which reason it is difficult to interpret these results.

1.4 - The (wave) series initially adopted for project design waves (H_s = 12 m, H_s = 10 m, H_s = 9 m, H_s = 6.5 m, for the maximum significant wave which could occur respectively: once in 100 yrs, once in 30 yrs, once in 10 yrs, and once in 1 year) which was deduced from the visual test made at Figueira da Foz, must be presumed, in view of the records of 2 years made by the Datawell buoy located at Sines, to correspond to conditions worse than the true conditions.

1.5 - Reference must be made to the tests executed in Paris by Condotte d'Acqua which indicated that the 30 t dolos were stable. Their interpretation is, however, dubious.

1.6 - Taking into account information available at the time, the present report is destined to ascertain whether the 30-t dolos retained in the ante-project phase can be used, or in case of doubt, what are the dimensions to be retained to avoid trouble in the execution of the works and a deficient performance of the structure.

2 - SELECTION OF PROJECT/DESIGN WAVE

2.1 - Criteria

As stated in Volume II of the ante-project "Basic Data," the selection of blocks for the protection cover was based on the following criteria:

(1) Oscillation of the blocks in storms of a violence with a frequency of 1 in 10 years;

(2) Initial displacement of some blocks in storms with a frequency of 1 in 30 years;

(3) Displacement from original position of up to 1% of the blocks in storms with a frequency of 1 in 100 years.

It is believed that there is no reason <u>not</u> to select the block weight by these criteria.

2.2 - Observations at Figueira da Foz

The determination of the wave height by visual observations made by buoy at Figueira da Foz is based on the report by Castanho and Reis de Carvalho (May 1972). These data were the only ones available at the time of preparing the ante-projects. The basic data were derived from 6 years of visual observations at Figueira da Foz.

Unfortunately, these observations were not executed continuously, presumably because of the damage suffered by the buoy during the more severe storms. In fact, nearly a half of the observations that should have been made are not included in the records. In view of the position of Sines at 240 kms to the south of Figueira da Foz, it is not likely that storms at Sines are as severe as those recorded at Figueira da Foz. An adjustment of the heights of waves was made by Engineers Castanho, Reis de Carvalho, and Vera Cruz. It is possible that this adjustment resulted in a distortion of the probability factor and an excessive reduction of the greatest wave heights.

2.3 - Records at Sines

As stated in the ante-project, a few more data on turbulence taken by the Datawell buoy at Sines were taken into account. The analyses of statistical probability were made on the basis of regular waves during a period of 1 year (October 1971 to October 1972). The period of observation is too short for the reliability of the selection, all the more so because the turbulence around our coasts is known to vary considerably from year to year. The records at Sines have the merit of having been taken in the location of the proposed breakwater, by recording equipment of high reliability. Though the period of observation is very short, the records at Sines deserve, after all, at least the same confidence as the results of the visual observations made at Figueira da Foz.

2.4 - Second Report by Engineers Castanho, Reis de Carvalho, and Vera Cruz

In August 1973, there was sent to the Consortium BCL the report covering the 2nd phase of the Marine Turbulence Study of Sines drawn up by Engineers Castanho, Reis de Carvalho, and Vera Cruz. This report contains a review of all the information available up to July 1973, indicating the most probable significant wave heights as follows:

in 5 years	:	7.40 m
in 10 years	:	8.10 m
in 30 years	:	8.90 m

Given the short period of observations made at Sines it is considered imprudent to base on these values the dimensioning of a work as large as the west breakwater.

2.5 - Design wave to adopt for dimensioning the dolos

Considering that the records of Figueira da Foz are incomplete, there cannot be much confidence placed in the transposition of the Figueira da Foz wave-height for that of Sines. Therefore, we advocate the adoption of the mean of the values established on the basis of the observations of Figueira da Foz during 6 years, and those established on the basis of the observations at Sines during 1 year, in order to dimension the dolos. The two sets of wave-heights and their mean are summarised in Table 1 as also the recommended values of the Second Phase Report of Engineers Castanho, Reis de Carvalho, and Vera Cruz.

TABLE 1 - Maximum Wave-heights Established for Sines

Maximum Significant Wave H_s (m)	Evaluated for Figueira	Evaluated for Sines by adjustment of Figueira Values	Evaluated from Datawell Observations at Sines	Recommended Values (Mean of 2 preceding values)	Second phase Castanho Report
Once in 10 yrs	12.8 m	9 m	7.7 m	8.4 m	8.1 m
Once in 30 yrs	--	10 m	8.8 m	9.4 m	8.9 m
Once in 100 yrs	16.4 m	12 m	9.9 m	11.0 m	--

3 - SELECTION OF PROTECTION BLOCKS

3.1 - Criteria

The unit weight of the blocks should be selected with the magnitude of the works in mind, as also the repercussions which an accident to the breakwater could have on the whole complex. That is why the execution of model tests, for the study of the proposed breakwater cross-section, is considered to be very important, using, of course, dolos of the correct down-scale.

Four series of model tests have already been made, as summarized below.

3.2 - LNEC tests of August 1972

The report on the tests made on a cross-section similar to that adopted for the ante-project concluded that the 30-t dolos would resist wave heights of approximately 14 m. Only one dolo was displaced by a wave-height of 12 m.

However, these tests were made hurriedly with a view of obtaining a preliminary indication and were not as complete as those mentioned in the later report.

Report on LNEC tests of January 1973

3.3 - The cross-section tested by LNEC in January 1973 was the one presented in the ante-project excepting some details indicated below.

3.4 - The results of the tests were certainly influenced by the fact of the porosity--particularly in the upper part of the TOT core--having been greater than that of the prototype. The effect of the greater porosity is, however, difficult to evaluate.

3.5 - To accelerate the execution of the tests, it was agreed to dispense with part of the systematization initially recommended. This circumstance also contributed to making the interpretation of the results difficult, on which only provisional conclusions can be drawn, to be confirmed in later tests.

3.6 - Table 2 presents a summary of the conclusions made on the basis of the various groups of test results.

TABLE 2 - Results of LNEC Tests of January 1973

Performance of dolos	Wave-height H_s
Beginning of oscillation	8 m
Beginning of displacement	10 m
1% of dolos displaced	10.5 m
5% of dolos displaced	12.5 m

3.7 - These results are much worse than those obtained in the initial tests. However, it was found that the models used were too light. As a matter of fact, the specific weight of the dolos models was 2.24 instead of 2.47, which would represent in fresh water the performance in sea water of dolos with a specific weight of 2.53, a value which is the minimum specific weight permissible for concrete used for dolos to be made in Sines.

3.8 - Furthermore, the dimensions of the dolos models were slightly greater than those corresponding to the reduction scale of 1/62, and therefore more subject to stress by the action of the waves.

For that reason, it is believed that the wave heights indicated in the report should be reduced by a factor in the order of 59/62 by reason of the linear scale of the dolos models being 1/59 and not 1/62 as stated in the report.

3.9 - The repercussions of these discrepancies between model and prototype may be evaluated by means of Hudson's formula created for stone rubble covers, or by using the results of studies now being conducted by Zwamborn and Beute.

3.10 - Correction by Hudson's Formula

At present Hudson's formula is the one most frequently used for determining the blocks of protection cover on the basis of wave height:

In this equation H is proportional to $(\gamma_s/\gamma_m - 1)$.

$$W = \frac{H^3}{k,\ (\gamma_s/\gamma_m - 1)^3 \cot\alpha}$$

The wave heights obtained should therefore be multiplied by the factor

$$\left(\frac{2.53}{1.025} - 1\right) : \left(\frac{2.24}{1.00} - 1\right) = 1.185$$

3.11 - Correction by Zwamborn and Beute formula

According to these authors the interlocking capacity of the dolos is one of the main factors contributing to their stability. Hudson's formula was created for blocks in which the main stabilising factor is weight. Zwamborn and Beute propose, therefore, that Hudson's formula be corrected and admit that for dolos

H be proportional to $(\gamma_s/\gamma_m - 1)^{1/3}$.

Thus, the wave-heights corresponding to the results of the LNEC tests should be multiplied by the factor

$$\sqrt[3]{1.185} = 1.06$$

3.12 - Comparison of Both Methods of Correction

The results of the corrections of wave heights for dolos of a nominal weight of 30 t (the present minimum weight of these blocks is 31.4 t) are as presented in Table 3.

TABLE 3 - Corrected Wave-heights

Performance of 30-t dolos	Results interpolated in model (Jan. 1973)	Corrected Heights Hudson	Zwamborn
Beginning of oscillation	8.0	9.0	8.1
Beginning of displacement	10.0	11.3	10.1
1% of dolos displaced	10.5	11.85	10.6
5% of dolos displaced	12.5	14.1	12.6

3.13 - On comparing the values of Tables 3 and 1 the conclusion is that if the correction was made by Hudson's formula, the 30-t dolos accord with the project criteria and in practical terms with the criteria of the ante-project. On the other hand, the Zwamborn and Beute formula indicate that the 30 t dolos would be under-dimensioned for the extrapolated waves of Figueira da Foz, but could be adequate for the waves at Sines determined on the basis of the Datawell records. For wave heights of 12 m, damage could amount to 3 to 4% of dolos displaced, much greater than the 1% which could be considered permissible.

3.14 - In view of the doubts existing at the time of the tender completion in regard to the performance of 30 t dolos, the competitors were requested to quote prices for the use of 40 t dolos instead of 30 t dolos.

3.15 - The results of the LNEC tests were used to evaluate the stability of 40 t dolos on the basis of criteria mentioned in 3.10 and 3.11.

Table 4 gives the wave heights for different degrees of displacement of these blocks.

TABLE 4 - Wave-heights for 40 t dolos

Performance of 40-t dolos	Wave Heights Hudson	Wave Heights Zwamborn
Beginning of oscillation	9.75 m	8.75 m
Beginning of displacement	12.2 m	10.9 m
1% of dolos displaced	12.8 m	11.45 m
5% of dolos displaced	15.25 m	13.6 m

3.16 - With the reservations inherent in the form of evaluation, it is concluded that the 40 t dolos accord with the criteria proposed for the project, except in the case of application of the Zwamborn and Beute method to the waves determined on the basis of the Figueira da Foz observations.

3.17 - Given that no adequate method is known for correcting the results of tests made in the condition cited above, it is imperative to execute new tests, using dolos models in full scale of both weight and dimensions to obtain confirmation of the adequacy of 30 t dolos. The new tests should be made with the breakwater cross-section proposed by the adjudicator and which will be adopted for the project.

Tests by Condotte d'Acqua

3.18 - Condotte d'Acqua had tests made on models at LCHF, Paris, for checking the performance of their alternative solution for the west breakwater.

3.19 - In principle, the results indicated that the 30 t dolos would certainly be adequate for 15 m high waves. The tables of the report indicate little damage. However, this is deemed to be a defect in the report, as the text states that a total of 6% of the dolos were displaced during the tests.

3.20 - On the other hand, the cross-section proposed by the competitor was tested for only 1 to 3.5 hours (time in prototype), instead of the 12 hours specified in the basic data.

The dolos used in these tests were also too light. These deficiencies were considered acceptable, given the short time available for the tests, and the fact of claiming to appreciate in principle the magnitude of the wave-topping in the alternative solution. However, the circumstances cited are prejudicial to the value of the tests as a means of evaluating the stability of 30 t dolos.

3.21 - LNEC Tests with Irregular Waves

The ante-project cross-section was tested at LNEC with irregular waves. Unfortunately, the test technique is very recent and does not yet permit drawing favorable conclusions on the stability of the dolos. The report concludes that their stability is not inferior to that indicated with regular wave.

4 - COST OF INCREASING DIMENSIONS OF DOLOS

4.1 - The adjudicator has established an increase in cost of $200 per cubic meter of concrete in the use of 40 t instead of 30 t dolos.

4.2 - Given that the increase in volume of concrete would be 8% (were the 40 t dolos to be substituted for the 30 t and the prices now proposed accepted), the increase in the total cost of the breakwater would be about 70,000 contos.

4.3 - If the unit prices proposed by the competitor for 30 t and 15 t dolos had not been increased, as other competitors allowed, the use of 40 t dolos would entail an increase in cost of the order of 20,000 contos.

5 - REPERCUSSIONS OF EVENTUAL DAMAGE TO DOLOS COVER

5.1 - Available information leads one to fear that the 30 t dolos would suffer displacements totalling over 1%, a percentage said to have a frequency of 1 in 100 years. Should the displacements be of the order of 3% and broken/severed dolos of a similar order, the repair of damage could reach about 6% of the total number of blocks, or about 1,100 dolos (in accordance with ante-project calculations).

Given that the cost of repair work would be three times the Condotte proposal cost, the cost of repairs would total about 52,000 contos.

5.2 - It should be noted that the tests made in various laboratories indicate that the stability of dolos covers is attributable to the interlocking system used, and also that the dolos covers begin to disintegrate as soon as some dolos are displaced from their original positions.

5.3 - These circumstances cause doubts about the dimensioning of the dolos because of the performance of the dolos covers, and make advisable a selection of weight of dolos which would limit the percentage of displacement to a very low level, at most 1 to 2% even for storms of very low frequency.

5.4 - The repercussions on the exploitation of the port which extensive repair works on the breakwater cover would cause are difficult to assess. Even if the breakwater cross-section were designed in such a way as to permit the operation of the dolos-placing hoist at high levels with a minimum disturbance of the loading and unload ing operations, experience shows that those services would have to be interrupted and could be resumed only after the lapse of some time following the completion of repair works. The costs resulting from the interruption of port services would depend on the density of traffic, but would in any case be very high. Therefore, there could be no justification for economizing on the cost of cover and raising the cost of port operations by increased maintenance, through having adopted criteria inferior to that adopted initially as mentioned in 2.1 of this report.

6 - CONCLUSIONS

6.1 - There are grounds for believing that the extreme values established for wave heights at Sines on the basis of analysis of the Figueira da Foz observations are excessive. However, the fact of only 1 year of records at Sines does not permit complete rejection of the forecasts made. Therefore, it is recommended that until more reliable information is available, a mean of the Figueira da Foz-based value and the Sines-based value should be adopted.

6.2 - The most reliable laboratory tests for evaluating the stability of the dolos are those mentioned in the LNEC Report of January 1973, with regular waves. However, the dolos used were too light, and the interpretation of the results does not permit reliable conclusions on whether 30-t dolos are adequate for the Sines breakwater.

6.3 - It was urgently recommended to proceed with tests on models with the cross-section of the breakwater to be constructed, using 30-t dolos with the correct weight and dimensions in scale. These tests would confirm whether 30-t dolos would be adequate for Sines.

6.4 - A summary interpretation of the tests already executed indicates that 40-t dolos would be sufficiently stable in the conditions of turbulence which are probable for Sines.

6.5 - A cover of 40-t dolos would have a greater thickness, which would contribute to the reduction of wave-topping, a great advantage for the maintenance of the pipes.

6.6 - Given that the laboratory tests could not be completed before the deadline for the contractor to place orders for necessary equipment, it is advisable to make provision for the use of 40-t dolos, in case the 30-t dolos are revealed by the tests to be insufficiently stable, yet trying nevertheless to reduce to a minimum the costs resulting from present doubts.

6.7 - Despite the fact that the use of 40 t dolos imposes an increase of 8% in the quantity of concrete required for the blocks of the protection cover, and that the increase in costs of $200/m^3 of concrete ordered by the adjudicator, we are of the opinion that it is economically preferable to increase the weight of the dolos of the cover than to run the risk of later changes of equipment causing trouble for the progress of the works, or the risk of excessive costs of repairs to the cover or the pipes.

Lisbon, 30th August 1973.

APPENDIX L

Summary of Wave Heights During Construction

and

Placement Schedule of Quarry Material

TABLE L-1 WAVE HEIGHTS (m) DURING CONSTRUCTION 1974

		Week 1	2	3	4
January	Max.	9.30	9.00	6.50	4.60
	Min.	0.60	1.90	0.90	1.50
February	Max.	10.00	4.70	9.50	6.50
	Min.	1.60	1.30	1.90	1.50
March	Max.	3.60	4.40	6.50	4.70
	Min.	0.30	1.40	1.80	0.80
April	Max.	6.60	4.90	3.20	6.00
	Min.	0.90	1.00	1.20	1.30
May	Max.	7.00	4.40	4.20	7.30
	Min.	1.70	1.10	1.10	0.40
June	Max.	2.50	2.70	4.30	4.80
	Min.	1.20	0.80	0.80	1.70
July	Max.	3.40	4.40	3.00	2.80
	Min.	1.10	1.00	0.70	0.80
August	Max.	3.20	3.60	2.80	3.40
	Min.	0.80	1.10	1.00	1.20
September	Max.	4.00	a1	2.10	3.20
	Min.	1.40	a1	1.20	1.20
October	Max.	2.50	2.60	3.40	4.40
	Min.	0.60	0.80	1.10	1.60
November	Max.	3.10	8.60	5.70	5.40
	Min.	0.90	1.40	1.60	1.00
December	Max.	3.50	3.80	5.50	7.90
	Min.	1.00	1.10	1.10	1.60

TABLE L-2 WAVE HEIGHTS (m) DURING CONSTRUCTION 1975

		Week			
		1	2	3	4
January	Max.	2.70	4.40	9.40	6.50
	Min.	0.90	0.70	2.90	2.60
February	Max.	5.30	7.30	9.00	4.10
	Min.	2.00	1.90	2.70	1.70
March	Max.	7.10	10.10	7.30	5.40
	Min.	1.90	1.70	1.30	1.40
April	Max.	2.80	3.50	5.50	3.40
	Min.	0.80	1.00	1.20	0.70
May	Max.	2.20	3.20	3.70	3.40
	Min.	0.60	0.90	1.40	0.80
June	Max.	3.10	3.80	2.60	1.30
	Min.	0.70	1.00	0.60	0.50
July	Max.	2.10	2.80	2.50	1.40
	Min.	0.60	0.50	0.30	0.20
August	Max.	2.80	3.30	3.30	2.00
	Min.	0.30	1.00	1.00	0.90
September	Max.	2.20	3.20	4.50	3.30
	Min.	0.90	0.90	0.80	1.10
October	Max.	5.20	3.70	3.10	4.80
	Min.	2.20	1.50	1.20	1.30
November	Max.	4.00	3.40	4.60	3.90
	Min.	1.30	0.80	1.30	0.70
December	Max.	8.80	5.30	3.20	4.40
	Min.	3.00	1.30	0.80	0.70

TABLE L-3 WAVE HEIGHTS (m) DURING CONSTRUCTION 1976

		Week 1	2	3	4
January	Max.	3.70	3.40	2.90	4.00
	Min.	0.80	1.20	1.00	1.10
February	Max.	9.10	9.10	6.10	5.70
	Min.	2.10	2.60	2.10	1.70
March	Max.	5.30	6.90	8.80	6.60
	Min.	1.40	1.60	2.60	1.60
April	Max.	5.60	3.70	6.00	4.50
	Min.	1.10	0.60	1.70	0.90
May	Max.	4.00	3.90	4.30	4.40
	Min.	1.10	1.20	1.30	0.90
June	Max.	3.80	3.10	5.50	2.50
	Min.	0.90	1.00	1.20	0.80
July	Max.	2.00	3.50	3.50	2.40
	Min.	0.50	0.50	0.90	0.20
August	Max.	1.90	1.90	1.70	3.20
	Min.	0.30	0.50	0.40	0.80
September	Max.	3.80	4.20	3.30	5.30
	Min.	0.90	1.50	0.90	1.00
October	Max.	4.10	4.20	4.00	5.70
	Min.	0.50	1.40	1.70	0.60
November	Max.	4.00	4.00	2.00	1.80
	Min.	0.80	1.10	0.40	1.00
December	Max.	7.20	9.80	5.40	7.80
	Min.	1.30	0.80	0.80	1.30

TABLE L-4 WAVE HEIGHTS (m) DURING CONSTRUCTION 1977

		Week 1	2	3	4
January	Max.	3.80	4.70	4.70	6.40
	Min.	0.80	0.80	1.30	1.30
February	Max.	6.70	7.40	4.90	6.80
	Min.	1.90	2.50	2.00	2.90
March	Max.	5.40	7.60	6.80	4.50
	Min.	1.10	1.10	1.00	0.70
April	Max.	3.20	5.60	3.00	3.50
	Min.	1.20	1.30	0.70	1.00
May	Max.	3.10	2.80	5.40	2.80
	Min.	1.20	1.20	1.80	0.80
June	Max.	2.30	3.40	2.50	2.00
	Min.	0.80	0.80	0.60	0.70
July	Max.	2.80	2.40	2.60	3.00
	Min.	0.90	1.00	1.10	1.40
August	Max.	3.80	2.30	2.90	4.00
	Min.	1.30	1.20	0.80	1.00
September	Max.	2.80	2.60	2.90	4.00
	Min.	0.70	0.60	1.10	0.50
October	Max.	NA	NA	NA	NA
	Min.	NA	NA	NA	NA
November	Max.	NA	4.50	3.90	3.80
	Min.	NA	1.60	1.30	0.60
December	Max.	6.60	9.60	5.70	5.90
	Min.	1.00	3.40	1.90	1.20

TABLE L-5 WAVE HEIGHTS (m) DURING CONSTRUCTION 1978

		Week 1	Week 2	Week 3	Week 4
January	Max.	3.60	4.60	6.00	5.60
	Min.	0.50	1.10	1.60	2.10
February	Max.	3.10	4.50	7.00	10.00
	Min.	1.60	1.00	1.50	2.00
March	Max.	9.80	5.50		
	Min.	0.00	0.30		
April	Max.				
	Min.				
May	Max.				
	Min.				
June	Max.				
	Min.				
July	Max.				
	Min.				
August	Max.				
	Min.				
September	Max.				
	Min.				
October	Max.				
	Min.				
November	Max.				
	Min.				
December	Max.				
	Min.				

TABLE L-6 PLACEMENT SCHEDULE OF QUARRY MATERIAL* – 1976 (GAS)

	January		February		March		April		May		June		July		August		September		October		November		December	
Type	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea
TOT Select	1		2		2		46		61		74		114		116		105		70		93		15	
TOT	3	148	< 1	160		243	28	214	37	209	23	208		275	2	290		327		388	1	205	< 1	203
3/6 t	11	1	5		5		12		24		22		22		25		21		19		24		10	
50/100 kg	1		< 1		2		2		2		2		2		2		1		1		1		1	
0.5/3 t	< 1		1		< 1		4		7		6		10		10		9		5	1	1	1	3	
0.25/1 t	< 1		1		< 1		7		6		7		9		8		11		13		15		5	
9/20 t	< 1	2	< 1				< 1	5		17	1	13		18		23		14		12		7		2
9/12 t		< 1	1		1		< 1	< 1	< 1		1		1		1		1		1		1		2	
12/16 t			8		2										< 1		< 1							
0/100 kg							1	2	< 1	5		10		5		< 1								

* In hundred thousand tons.

TABLE L-7 PLACEMENT SCHEDULE OF QUARRY MATERIAL* - 1977 (GAS)

Type	January Land	January Sea	February Land	February Sea	March Land	March Sea	April Land	April Sea	May Land	May Sea	June Land	June Sea	July Land	July Sea	August Land	August Sea	September Land	September Sea	October Land	October Sea	November Land	November Sea	December Land	December Sea
TOT Select							52				53		1		1									
TOT		230		209		358	26	162	143	254	8	78	3	85		4								
3/6 t	4				3		19		28	3	25	33	16	19	8	3	2	2	3		2	1	3	
50/100 kg	2		1		1		1		1		3		4		2		2		5					
0.5/3 t	1		2	2	< 1	15		6	1	23	< 1	44	< 1	23	< 1	1		1	< 1		1	$\frac{1}{2}$	< 1	
0.25/1 t	< 1				1		13		14		12		2		< 1		1							
9/20 t							6			16	21			17	< 1		1							
9/12 t	1		< 1		2		< 1				1		3		4		2		1					
12/16 t																< 1								
0/100 kg						1									1		< 1							

* In hundred thousand tons.

TABLE L-8 PLACEMENT SCHEDULE OF QUARRY MATERIAL* - 1978 (GAS)

	January		February		March		April		May		June		July		August		September		October		November		December	
Type	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea	Land	Sea
TOT Select																								
TOT																								
3/6 t					4																			
50/100 kg																								
0.5/3 t			1		< 1																			
0.25/1 t	< 1																							
9/20 t																								
9/12 t																								
12/16 t																								
0/100 kg																								

* In hundred thousand tons.

APPENDIX M

Evaluation of Causes of Damage to Main Breakwater

Reproduced with permission from: "Sines Harbour: Evaluation of Causes of Damage to Main Breakwater," CSIR Report C/SEA 8005, Council for Scientific and Industrial Research, Stellenbosch, South Africa, March 1980.

Prepared by CSIR for: Societa Italiana per Condotte d'Acqua

SINES BREAKWATER DAMAGE

(26th February, 1978)

	POSSIBLE CAUSES	DISCUSSION	CONCLUSION
DESIGN ASPECTS	Inadequate Dolos mass for the design wave spectrum with H_s=11m (actual Dolos mass 41,8t, volume, V =16,6m³, s.g. 2,5).	For H_s=11m, Hudson's K_D = 17, which is conservatively low. However, Morais[22] found damage for irregular waves to be significantly more than for equivalent regular waves, even for -35m depth (actual depth -35 to -50m). NRC[19] showed, in *one* test, complete failure for H_s=10 to 13m; this is the only complete test with irregular waves on the as-built structure. This test suggests that the design criteria, i.e. no Dolos movement up to H_s=6,5m and only 1% damage for H_s=11m, were not nearly met.	Based on conventional design techniques the Dolos mass should be adequate but the available test data is insufficient to confirm that this is so, in fact, there are indications that, due to the great depth in front of the structure combined with irregular waves, the 42t Dolosse *do not meet the design criteria*. This can only be confirmed by additional tests at an adequate scale (e.g. ~1:50).
	Too steep a slope of the Dolos armour (actual slope 1:1½)	WES[7] indicates greater stability when the same size Dolosse are used on a flatter slope but others have found Dolos stability to be independent of slope.[2,19,25] Virtually all Dolos structures in South Africa[27] are built to a 1:1½ slope without any indication of failure. Due to the Dolos' interlocking character, its angle of repose is at least 80 degrees and they are therefore particularly effective on steeper slopes[1]. However, one test at NRC[19] with breaking model Dolosse shows an improvement for a 1:2 slope and some recent LNEC test results[25] indicate that a 1:2 slope *under water* may be slightly better.	The 1:1½ slope of the Dolos armour *cannot be the cause* of the major failure which occurred at Sines. A flatter slope may or may not have reduced the extent of the damage, available data is insufficient on this point to reach a definite conclusion.
	Incorrect Dolos packing density, e.g. insufficient number of Dolosse per unit area of breakwater slope.	A packing density of 0,155 units/m² was prescribed by the designers.[20] *Recent* research [8,30] shows that the 'optimum' packing density (greatest K_D value) is 0,9 to 1,0 $V^{-2/3}$= 0,138 to 0,154 units/m². The prescribed packing density would result in a layer thickness of 2 x 1,03 x $V^{1/3}$= 5,3m whereas a layer thickness of 6,5m was prescribed (Fig. 1). This is over 20% more than the correct value of 5,3m and it is thus obvious that considerably more Dolosse would have to be placed to reach the prescribed profile (the packing density corresponding with 6,5m follows from 1,2$V^{-2/3}$= 0,184 units/m², i.e. 19% extra Dolosse).	The correct packing density was prescribed but it was inconsistent with the specified layer thickness. Although, as a result, more than the 'optimum' number of Dolosse were placed, this *cannot be the cause* of the failure.
	Inadequate size of underlayer stone and too thin a layer.	3 to 6t stone was specified and the layer thickness varied from 2,3 to 5,4m (Fig. 1). For Dolosse, an underlayer stone mass of 1/5 x Dolos mass is recommended[29], based on normal densities of 2,4 and 2,65 for concrete and stone resp. Thus, the required stone *volume* becomes V_u=2,4V/2,65x5=16,6/5,5=3m³. The stone at Sines has an s.g. of 3,0 and, for proper interlock, the average underlayer stone mass should thus be 9t and the layer thickness[29] 2 x 1,15 x 3 $^{1/3}$=3,3m. Some tests carried out by LCHF[15] indicate, however, that an increase in stone mass from 3 to 6 to 9 to 20t (4m layer) does not significantly effect the Dolos stability.	Although the specified stone mass or size is smaller than that normally used under Dolosse, this *cannot be the cause* of the failure of the armouring.
	Inadequate berm width and stone mass in the berm resulting in toe failure.	Considering the stone size (16 to 20t) and the depth of water (-15m) at which the berm had to be constructed, the specified width of 3,2m (Fig.1) is very small; a minimum width of twice the equivalent stone size, i.e. 2 x 1,15 x $(18/3)^{1/3}$=4,2m being considered more appropriate. The originally intended 6 to 9t stone in the berm proved to be unstable in the LNEC model tests and were therefore replaced by the present 16 to 20t stone, which proved to be stable in the model[22]. Profiles and underwater surveys after the February storm confirm that the toe of the structure remained stable.	There is no doubt that the specified narrow berm was *difficult to construct* at a waterdepth of -15m but there is considerable evidence to conclude that toe failure has *not played a significant role* at Sines.
	Inadequate breakwater crest design.	The Dolos armour was taken to + 16,0m and the vertical wave wall to + 19,0m CD(Fig. 1). For H_s=11m, the Dolos armour was thus 1,11 H_s above high water (+ 3,8m CD) which is above the damage zone for regular waves[30] but not for the larger waves of a spectrum (for H_{max}=1,6 x 11=17,6m, crest level is 0,7 H_{max} above high water). Thus, for the higher waves which could cause runup far exceeding the Dolos crest level, considerable reflection of water will take place which could adversely effect the Dolos armour. Some tests by LCHF[15] show a considerable reduction in damage, for H_s=11 to 12m, when the vertical wall is taken away (up to a factor 4) with a corresponding increase in overtopping. A further test indicates that the wall considerably increases the rate of damage, particularly for the longer wave periods (> 18s).	Particularly the unprotected section of the vertical wave wall (from + 16 to + 19m CD) causes considerable reflection of water for the long and high waves in the spectrum. This reflected water has an adverse effect on the stability of the Dolos armour and has no doubt contributed to the damage. However, this *cannot be the cause* of the failure.

	POSSIBLE CAUSES	DISCUSSION	CONCLUSION
CONSTRUCTION ASPECTS	Bad quality of concrete and Dolos breakage during placing.	Observed Dolos quality was excellent. Most test cubes gave compressive strengths from 40 to 60 MPa whereas 40 MPa was specified. Laboratory rupture strength tests also show very high values, i.e. > 6 MPa, indicating a high quality concrete. Placing of Dolosse by floating crane was limited to 'quiet' sea conditions[24]. Some model tests were done in Stellenbosch to determine *underwater* oscillation periods and amplitudes for 42t model Dolosse suspended from a fixed point above water. From the results, maximum velocities of movement were calculated which gave a maximum value of about 0,45 m/s for waves up to 1,5m height and 15s period with a 20m cable length. The *maximum value,* corresponding to resonance conditions, i.e. critical cable lengths, was found to be about 0,7 m/s for the same wave conditions. Two tests carried out at Sines with 42t Dolosse showed impact velocities of ≥ 0,5 m/s for leg breakage and ≥ 1,5 m/s for shank breakage. Assuming that these tests are representative, and, because the maximum possible lowering speed of the cranes is 0,2 m/s, there seems to be no reason to believe that the 42t Dolosse should break during placing with waves up to 1,0 to 1,5 m. The unlikelihood that a significant number of 42t Dolosse would have broken during placing is *confirmed* by the fact that 1988 unreinforced Dolosse have been placed, as part of the repair work started in August 1978, of which only 21 or 1,0% have broken(confirmed by diver inspection). No detailed 'as built' inspection report is available, estimated breakages varying from 1,3 to 3%.[10,20,24] An underwater inspection 'report' by GAS[11] of a very limited survey, carried out in August 1977, made available recently, shows breakages between 3,5 and 16,8% of the *top layer* (or 1,7 and 8,4% of the total armour, i.e. double layer) in a 134m section between ch. 1086 and 1220, just seaward of Berth no. 2. In the same section, 3 clusters of 4 broken Dolosse were indicated. This data, however, does not agree with the information provided previously[10]. In any case, tests carried out by WES[3] showed that stability is only affected when random breakage exceeds 15% and cluster breakage exceeds 3 units. LNEC test results, reported verbally in June 1978, confirm that random breakage of 10 to 20% does not appear to affect stability (50 to 70% breakage between + 2 and -6m CD was required to cause the slope to fail). A recently completed underwater survey of the breakwaters of the port of Ashdod shows breakages of 40t tetrapods up to 65% of the top layer while the armour was still judged to be safe.[5] A complete series of photographs taken in November 1977 shows hardly any broken Dolosse above water except at ch. 706 where there was storm damage in March 1977.	It is unfortunate that no proper 'as built' underwater surveys are available and any conclusion may therefore be subject to certain doubt. Above water, the quality of the work can only be called good while available evidence strongly indicates that Dolos breakages underwater were such that they could harly have affected the armour stability. Although a cluster of broken Dolosse constitutes a potential weak point where damage may have initiated, Dolos breakage before the February storm *is not considered to be an important contributary cause* of the breakwater failure.
	Incorrect placing and uneven distribution of Dolosse on the slope.	It has been suggested that Dolos placing was not sufficiently controlled which could have effected their interlocking. Placing according to a fixed grid is essential, particularly underwater, to ensure an even packing density. Where practical, the use of divers to control the placing is also advisable. However, experience in South Africa, both with models and in the field, shows that random placing is the only method which results in a well-interlocking armour; pattern packing is neither practical nor desirable. Condotte's records show an actual packing density of 0,1816 units/m^2. This is 17% more than the prescribed 0,155 units/m^2 and these extra Dolosse must have considerably reduced the chance of the formation of local thin spots in the underwater armour due to possible inaccurate placing techniques. However no 'as built' profiles of the Dolos slope are available and there is, therefore, no certainty regarding even placing.	The visible part of the Dolos, armour shows good interlocking and, also because of the high packing density, there is no reason to believe that the underwater part has had serious deficiencies in this regard. *Assuming* the units were placed at random and reasonably evenly distributed this factor *cannot have played a significant role* in the failure.
	Inaccuracies in the underlayer thickness.	The measured profiles of the underlayer stone generally agree within 1 to 1,5m with the theoretical profile but there are some sections where the underlayer appears to be only 1m thick, e.g. sections ch. 945, 1045 and 1545. According to the contractor, these sections have been rectified before placing Dolosse, which is confirmed by a subsequent profile at ch. 945 dated 22-9-75. Even if the Dolosse were placed directly on the 'thin' underlayer the chance of a wash-out of core material is not considered very great because the thin sections were always close to the toe of the armour (near -15m) where water	The inaccuracies in underlayer thickness, shown by the available profiles, are *not considered to be a significant cause* for the failure of the breakwater slope.

	POSSIBLE CAUSES	DISCUSSION	CONCLUSION
CONSTRUCTION ASPECTS (Continued)	(continued)	movements are much less severe than near the still waterline. This conclusion was confirmed by model tests carried out at LCHF on the section at ch. 945, assuming an underlayer of only 1m thick. These tests showed no difference in the damage compared with previous tests using the full underlayer thickness.[15] The possibility that the underlayer could have been much thicker in places, resulting in a too thin Dolos layer, has also been mentioned, particularly with regard to the possibly not complete removal of temporary winter protection stone near ch. 700, 1100 and 1800. Available profiles, however, do not support this suggestion, in fact, the only section which shows too thick (2 m) an underlayer is ch. 1645 which does not agree with the above winter protection areas. Moreover, because of the 17% extra Dolosse actually placed, a possible 'bump' in the underlayer would, more than likely, still get the required minimum cover of Dolosse, i.e. 0,138 to 0,154 units/m^2.	(continued)
	Unsatisfactory toe construction (berm).	Considering the rather small berm width (3,2m), the measured profiles conform quite well with the theoretical section. LNEC test results, reported verbally in June 1978, show no effect on stability, even when the berm width is reduced to zero (H_s=8,5m). Moreover, when studying the measured profiles, after the February storm, the conclusion is reached that toe failure cannot have played a major role because of the absence of deposits at the foot of the breakwater in sections 2,4,11 and 30 (Fig.3). Also, during a recent site visit (25.8.78) it was observed that unbroken Dolosse were lifted from the toe section of the armour near Berth no. 2 (major damage area) which provides proof that the toe did not yield.	Inaccuracies in the construction of the required berm profile *cannot be held responsible* for the breakwater failure and the effects on armour stability are considered *negligible*.
	Too much fines in the filter layer or in the core material.	A possible cause of failure could be the wash-out of fines from the 0,25 to 1t filter stone (Fig. 1) and further wash-out of core material causing settlement of the Dolosse and the crest structure. Tests were carried out at LNEC (reported verbally, June 1978) which showed indeed some settlement of the crest but the main armour slope was not affected significantly. Some tests done at NRC[19] using core material of 0 to 100 kg and the prescribed 0 to 3000kg also showed no noticeable effect on the damage behaviour of the Dolos armour.	Possible failure of the filter layer or too fine core material has *not played a significant role* in the failure of the breakwater.
	Incomplete wave wall and Dolos armour.	At the time of the storm, the wave wall seaward of ch. 1645 had not been completed and final Dolos placing (Fig. 1) had not yet taken place in this area. The consultants (July 1978) also show an area near ch. 1100 where the final Dolosse had not yet been placed. Because of the greater overtopping, the incomplete section of the breakwater should have been more stable on the seaward side. This was confirmed by model tests at LCHF which showed a reduction in damage without the wave wall.[15] The section at ch. 1100 could have caused some problems because of Dolos movements towards the wall to fill the open space. However, test results from LNEC (reported verbally, June 1978) indeed showed that the model Dolosse filled the gap, but the main armour did not fail. Moreover, serious damage was also caused in areas where the cross-section was complete so that the 'construction stage' cannot have been responsible for the failure.	The incomplete wave wall and Dolos armour is *not considered to be the cause* of the breakwater failure although shortage of Dolosse against the complete or incomplete wall could have caused some increased wave reflection.
WAVE CONDITIONS	Incident significant wave height exceeded the design wave height of H_s=11m.	Because no storm records are available from the waverider at Sines, the maximum wave heights, which occurred on the 26th February, can only be estimated. A maximum significant wave height of 8,5m was recorded at Cabo da Roca, about 120km north-west of Sines in '40m' water depth (the hydrographic chart shows only 20m at the waverider position!) A detailed analysis of all storm data was made by LCHF[13], later supplemented by a LNEC Technical Note on the storm wave data.[26] The results show H_{s-max} = 8,5 to 10m with a mean period of 12 to 14s and a peak period T_p = 18 to 20s. A further analysis, based on a correlation of the maximum recorded storm wave heights at Sines and Cabo da Roca, gives as the most probable value H_{s-max} = 10m (incident).	Although there are no wave records at Sines for the 26th February it appears that the incident significant wave height did not exceed about 10m and the *failure can, therefore, not be due* to the incident wave height exceeding the design wave height.

WAVE CONDITIONS (Continued)

POSSIBLE CAUSES	DISCUSSION	CONCLUSION
Local significant wave heights exceeded the design wave height of $H_s = 11m$.	Although damage occurred along the entire breakwater seaward of Berth no. 3, failure of the Dolos armour was concentrated in the four local areas shown in Fig. 3. It was, therefore, thought that wave refraction/concentration could have played an important role. Detailed refraction studies carried out by LCHF[14,17] show considerable wave concentration for wave periods in excess of 16s. The positions of the concentration areas *shift along the breakwater* depending on wave direction and wave period. However, when taking the mean refraction factors, based on the refraction diagrammes, for the storm directions (W to WNW) and wave periods between 16 and 22s, wave heights were, on average, found to be up to 20% higher than the incident wave height in some of the failure areas, while in the areas opposite Berths no. 1 and 2 which did not completely fail, the waves were on average about 15% lower than the incident height. The refraction digrammes also showed a maximum increase in wave height of up to 100% in the failure areas while the physical model showed maximum values of up to about 50%.	Accepting a maximum incident wave height of 9 to 10m, there is no doubt that local significant wave heights have exceeded the design wave height of 11m by at least 0,5 to 1m. There is a reasonably good correlation between these high wave areas and the failure areas and *local high waves are considered to be a major contributary factor* to the armour failure.
High waves in the spectrum, i.e. waves in excess of 11m.	The particular destructive force of the higher waves in the spectrum was demonstrated by LNEC during the stability tests on the Sines breakwater. Morais[22], as a result, concluded that the significant wave height cannot be used to describe a wave spectrum in the case of armour stability. Test results showing the opposite are also available[6,23]. However, these test results refer to different waterdepths and from this apparent contradiction it is concluded that, in the case of relatively shallow conditions, the significant wave is determinative, whereas in *deep water* (water depth exceeds $1{,}3\ H_{max} \simeq 2H_s = 22m$), the condition existing at Sines from Berth no. 3 seaward, the high waves in the spectrum are not reduced in height by the bottom and although their number is limited, they are responsible for most of the damage. This conclusion is supported by various stability tests carried out on the Sines breakwater[15,19] as well as more basic tests carried out by LCHF[16] which showed that for wave periods between 16 to 18s and wave heights in excess of 11m, the vertical upward water velocity in the area near the still water line is of the same order of magnitude as the terminal settling velocity of 42t Dolosse in still water, the latter being confirmed by prototype tests at Sines. This explains why for certain wave height and period combinations a group of Dolosse can be lifted from the slope by one large wave, a phenomenon frequently observed in model tests.	Considering that armour stability is inversely proportional with the wave height cubed and taking into account the possibility of individual waves further increasing in height due to refraction, it is concluded that the high waves in the spectrum have played the *dominant role* in the damage processes at Sines.
Long period waves and wave grouping.	Bruun[3] suggests that stability for Dolosse is least when $\xi = tg\ \alpha/\sqrt{H/L_o} \simeq 3$ which occurs for $H = 11m$ and $L_o = 223m$ (or a wave period of 12s) but the change in K_D for different ξ values is very small (for $T = 16s$, $\xi = 4$ and $T = 20s$, $\xi = 5$). Tests carried out at Wallingford[4] indicate that K_D reduces for larger ξ (i.e. T) values, which is in disagreement with Bruun's results. The reduction in stability for $\xi = 3$ to 5 (applicable to the February storm) is, however, quite small although these test results do stress the importance, with regard to damage, of the long waves which occurred during the storm at Sines. Wave grouping, i.e. several large waves grouped together, has recently received considerable attention. Johnson and Ploeg[12] showed that grouped waves can cause more damage than random waves although the difference appears to be only significant for wave steepnesses of 0,04 to 0,05 while the peak storm waves at Sines ($H = 10m$, $T \simeq 14s$) had a wave steepness $H/L = 0{,}03$. Burcharth[4], on the other hand, got very little difference between the damage caused on a Dolos slope for regular waves and grouped irregular waves. His tests, however, were done at quite a small scale and in a shallow water depth which may have affected the results.	The direct effects of long period waves and wave grouping are *not considered to have made an important contribution* to the breakwater failure although there is little doubt that, particularly after the wave wall became partly exposed, the long waves (and possibly wave grouping) increased the rate of failure.

POSSIBLE CAUSES		DISCUSSION	CONCLUSION
DOLOS BEHAVIOUR	Dolos movements (rocking).	As far back as 1966, rocking of Dolosse, in addition to Dolos displacement, have been recorded in model tests[21] but because the number of units which were seen to move was small and observations had to be done visually, little attention was given to these rocking movements until recently. Magoon and Baird[18] report on observations on a glass-walled flume and conclude that movement starts for a wave height between 0,5 and 0,7 H_d and severe movement and significant displacement takes place for 0,9 to 1,0 H_d, where H_d is the design wave height (not defined). Zwamborn[30] describes a time-lapse cine technique, developed recently, which makes it possible to accurately record unit movements. The results of available tests show the following : no movement $0 < H \leq 0{,}5\ H_d$; occasional rocking $0{,}5\ H_d < H \leq 0{,}7\ H_d$; intermittent plus occasional rocking $0{,}7\ H_d < H \leq 0{,}9\ H_d$ and continuous rocking and displacement $0{,}9\ H_d < H \leq 1{,}0\ H_d$, where H_d is based on $K_D = 25$ (about 2% displacement). These movements, which take place mainly near the water line, are much more severe than can be observed visually but, provided the units do not break, as is the case in the model tests described above, stability of the armour is not affected. A question which remains to be answered is whether the model results are representative for full-scale Dolos behaviour. The lower Reynolds number in the model increases viscous forces while too smooth model units reduce frictional resistance, both possibly resulting in too great rocking movements. Some test data reported by Brorsen et al[2] indeed showed significantly more damage for smooth plastic Dolosse than for concrete (cement) units.	In the model, rocking movements of Dolosse start near the still water line with wave heights between 0,5 and 0,7 times the design wave height and severe rocking occurs, over a larger area, when the wave height approaches the design wave height. Accepting that similar movements occur in the prototype but assuming, for a moment, that the units maintain their basic shape, these *movements themselves cannot have caused* the failure of the armour.
	Dolos breakage.	Knowing now that armour units can rock considerably for wave heights below the design waveheight, the question arises whether the units can withstand the resulting stresses and, if not, what is the effect on the armour stability when the units start to break. Unfortunately, no comprehensive quantitative data on Dolos breakage at Sines is available. Photographic evidence and observations during site inspections show relatively few broken units above water, even where they have obviously fallen down at the sides of the failure areas. Underwater, in the failure areas, virtually all visible Dolosse were broken while in the partly damaged areas, most Dolosse on the water line and just below, appear to have been broken. In answer to the second question, tests carried out at WES,[9] LCHF[15] and LNEC (reported verbally, June 1978) show that a relatively large percentage (10 to 20%) of the Dolosse can break before stability is affected. In the LNEC tests, some 60% of the Dolosse between + 2m and -6m CD had to be broken to reproduce failure with H_s = 8,5m (maximum). NRC[19] used a new test technique whereby a breaking plane was introduced in the shank of the Dolosse with a linearly scaled tensile strength. Reproducing Cabo da Roca wave data, the results of one test show 2% breakage before wave action (due to the weight of other Dolosse), 9% breakage for H_s = 4 to 6m, about 25% breakage for H_s = 6,2m and complete failure for H_s = 8,6m. Although this test technique has considerable merit, it is obvious that the structural behaviour of the units was not fully reproduced in these tests, limitations being, inter alia, the fixed breaking plane (prototype units break at random through the flukes and shanks) and certain doubts regarding the correct scaling of tensile, compressive and impact strengths. Wave heights in excess of H_s = 6m have occurred at Sines in the past without any apparent damage to the armour[24]. Morover, complete failure of the armour occurred along only half the deep water length of the breakwater while the NRC test result indicates that the entire deep water length should have failed.	Because it has been shown, by model tests, that a relatively large percentage breakage is required to significantly effect the Dolos armour stability and since some available prototype data shows that unreinforced Dolosse of various sizes can withstand waves up to the design wave, it is concluded that the failure at Sines *cannot be due solely* to the structural failure of the Dolosse.

	POSSIBLE CAUSES	DISCUSSION	CONCLUSION
DOLOS BEHAVIOUR (Continued)	(continued)	To find an answer to the first question the approach appears to be to study prototype behaviour of existing breakwaters. Over 120 000 unreinforced Dolosse have been placed successfully in coastal works in South Africa alone, varying in mass from 3 to 30t. Undersized Dolosse (unreinforced) were used for a Dolos island at Oranjemund[28], i.e. 10t in stead of 18,5t. As a result, these units moved around excessively and some 10% loss/breakage occurred during one storm with 6m breaking waves. Thus, when the wave height exceeds H_d, there is no doubt that Dolosse can break, *regardless of their size*. On the other hand, 12,4 and 17,1t Dolosse used at Gansbaai[27] were found to be virtually undamaged (also underwater) eight years after construction in 1970, although wave heights were regularly recorded to be equal and even exceed H_d. The main breakwater head at Richards Bay[6], which consists of 30t Dolosse, also shows no significant visual damage, 2½ years after its completion, although wave heights up to 0,6 H_d have already occurred. Dolos breakage tests carried out at Sines (pendulum tests) gave impact velocities of $\geq$ 0,5 m/s for leg breakage and $\geq$ 1,5 m/s (about) for shank failure. Preliminary model test results[16] on Dolos movements caused by wave action gave maximum velocities of the Dolos' centre of gravity of between 0,5 and 1,5 m/s for corresponding wave heights between 7 and 15m, indicating that shank breakage should be limited to the higher wave heights.	(continued)

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