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BREAKWATER DAMAGE BY SEVERE  
STORM WAVES AND TSUNAMI WAVES

by

Robert L. Wiegel

5 March 1982

Berkeley, California

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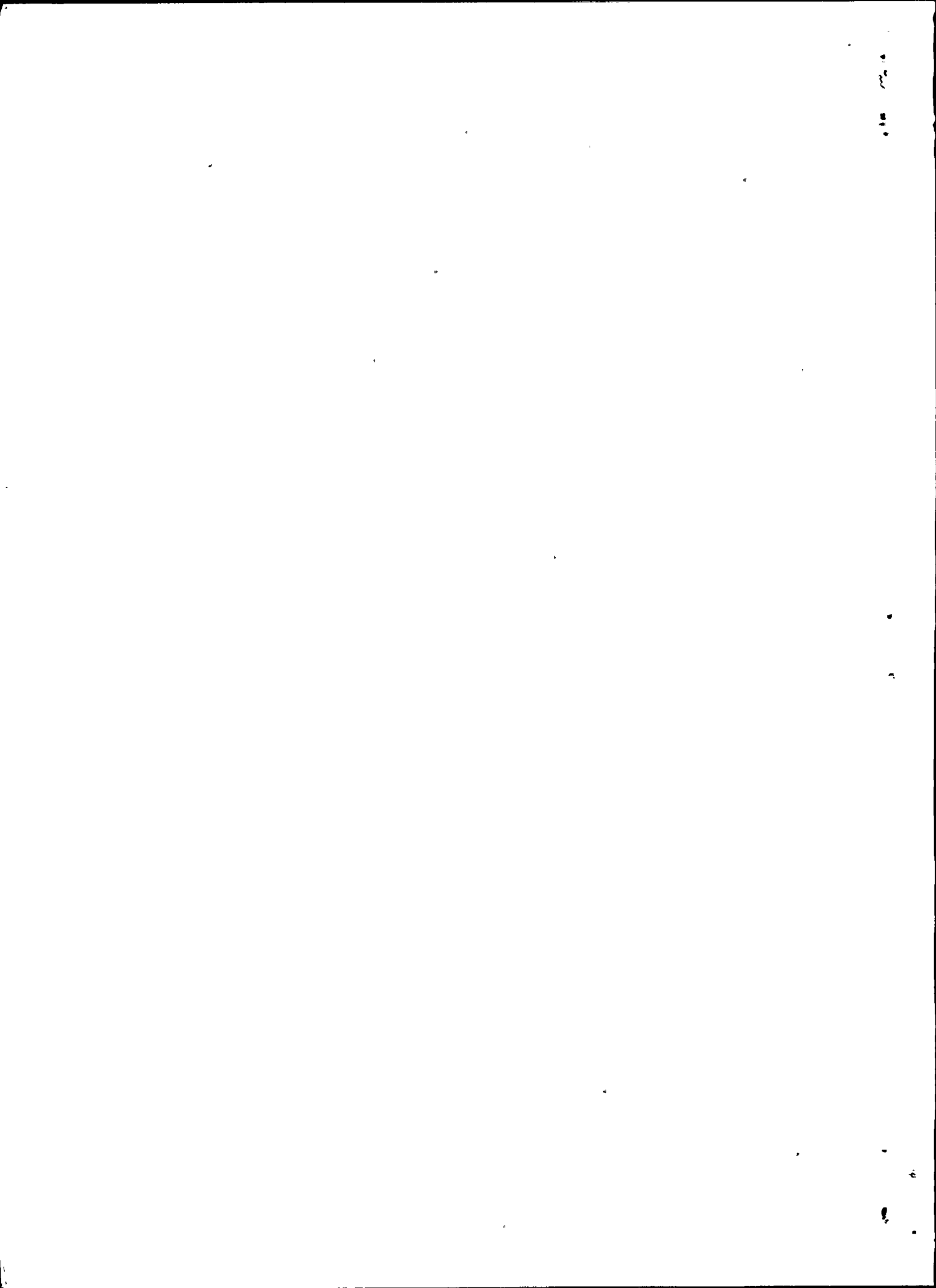
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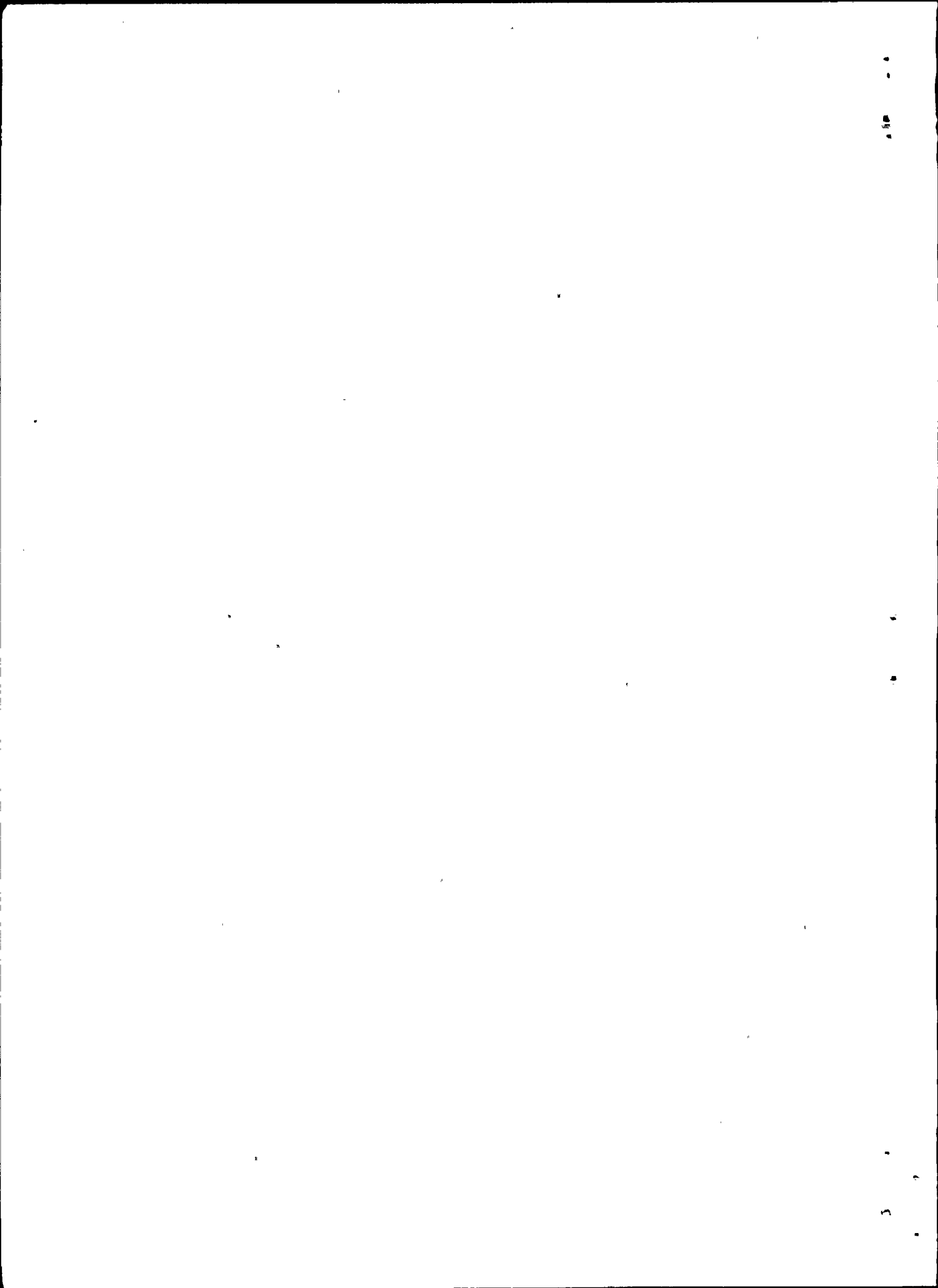
## TABLE OF CONTENTS

	Page
Introduction . . . . .	1
Conclusions . . . . .	3
Tsunami Damage to Breakwaters . . . . .	5
Severe Storm Wave Damage to Breakwaters . . . . .	19
Rosslyn Bay Breakwater, Queensland, Australia . . . . .	19
Praia da Vitoria Harbour, Azores . . . . .	24
Leixoes, Portugal . . . . .	30
Bilboa, Spain . . . . .	33
San Ciprian Harbor, Spain . . . . .	35
Crescent City Harbor, California . . . . .	35
Humboldt, California, Jetties . . . . .	38
Kahului Harbor, Maui, Hawaii . . . . .	41
Nawiliwili Harbor, Kauai, Hawaii . . . . .	47
Gansbaai, South Africa . . . . .	47
San Pedro Bay, California . . . . .	50
Silver Bay, Minnesota . . . . .	52
Sandy Bay Cape Ann, Massachusetts . . . . .	52
Wick Bay, Scotland . . . . .	52
Port Sines, Portugal . . . . .	54
Columbia River South Jetty, Oregon . . . . .	61
Japan . . . . .	67
References . . . . .	70

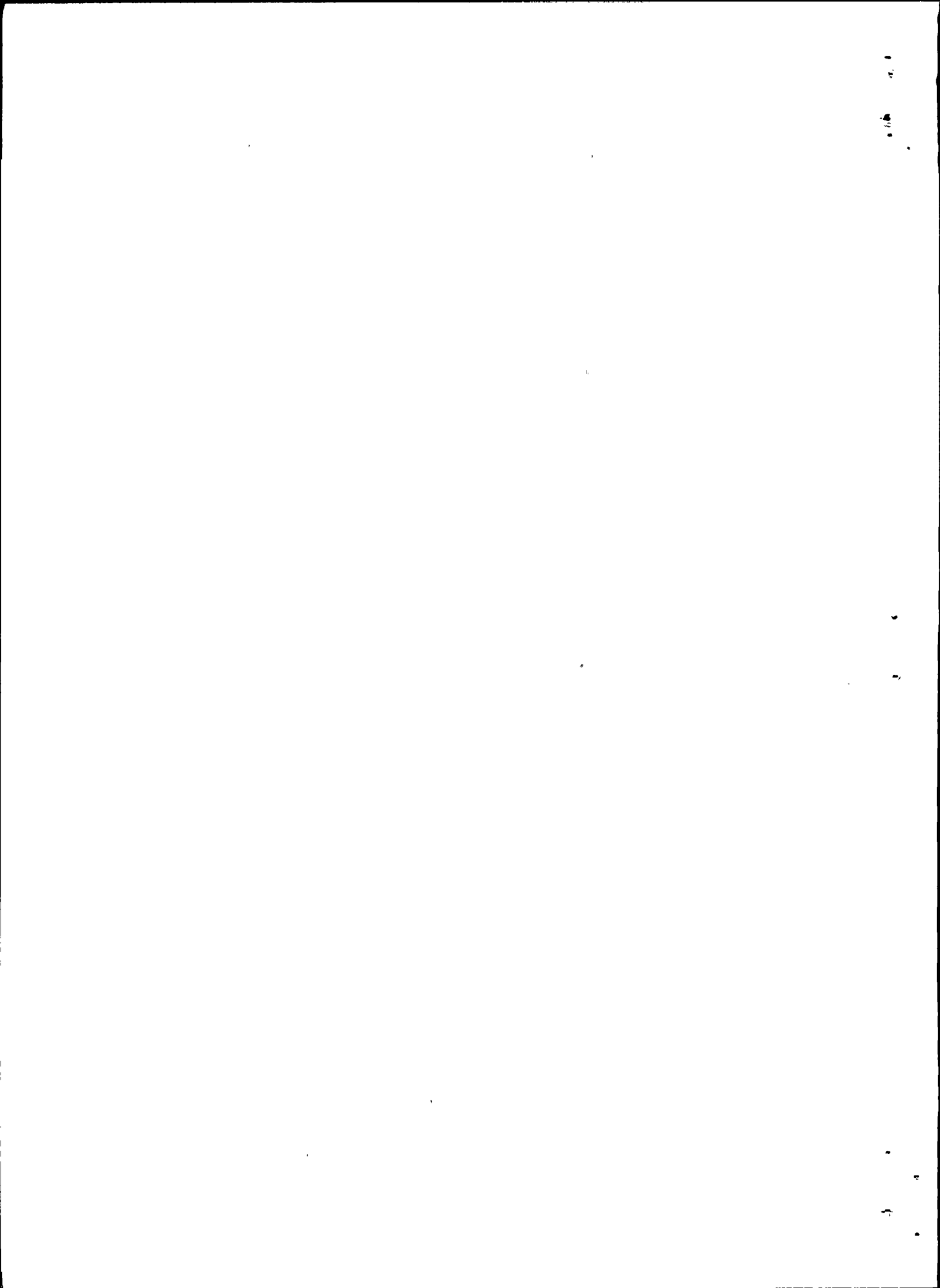


LIST OF FIGURES

<u>Figure</u>	<u>Page</u>
1. Breakwater at Hilo, Hawaii after tsunami of 1 April 1946	6
2. Breakwater and port at Hilo, Hawaii after tsunami of 1 April 1946	7
3. Hilo Harbor, Hawaii plan of improvement	10
4a. Aerial photo of City of Kodiak	14
4b. Southwest breakwater at Kodiak after earthquake	14
4c. Map of Kodiak City & harbor with details of the breakwaters	15
4d. Hypothetical marigram for Kodiak City	16
5. 100- and 500-year elevations vs. location, latitude 35°00' to 35°30', California Coast	18
6. Wave and storm tide conditions, Rossllyn Bay	20
7. Rossllyn Bay breakwater after cyclone damage	20
8. Typical cross section of breakwater, Rossllyn Bay	20
9. Locality sketch, Rossllyn Bay	23
10. Damage to Rossllyn Bay breakwater by Cyclone David	23
11. Praia da Vitoria Harbor: general arrangement	23
* 12. Original and alternate sections, Praia da Victoria	25
* 13. General view of harbor, Praia da Victoria	25
* 14. Views of the damages, Praia da Victoria	27
15. Breakwater sections after storm, Praia da Victoria	27
16. Breakwater sections after storm, Praia da Victoria	28
17. Breakwater sections after storm, Praia da Victoria	28
18. Lay-out of Leixoes Harbour	31
19. Head and cross-section of north breakwater, Leixoes	31
20. Damages caused by overtopping, Leixoes	32
21. Damages at the head, Leixoes	32
22. Damages at the head (cont.), Leixoes	32
23. Damages at the head (cont.), Leixoes	32
24. Damages at the head (cont.), Leixoes	32



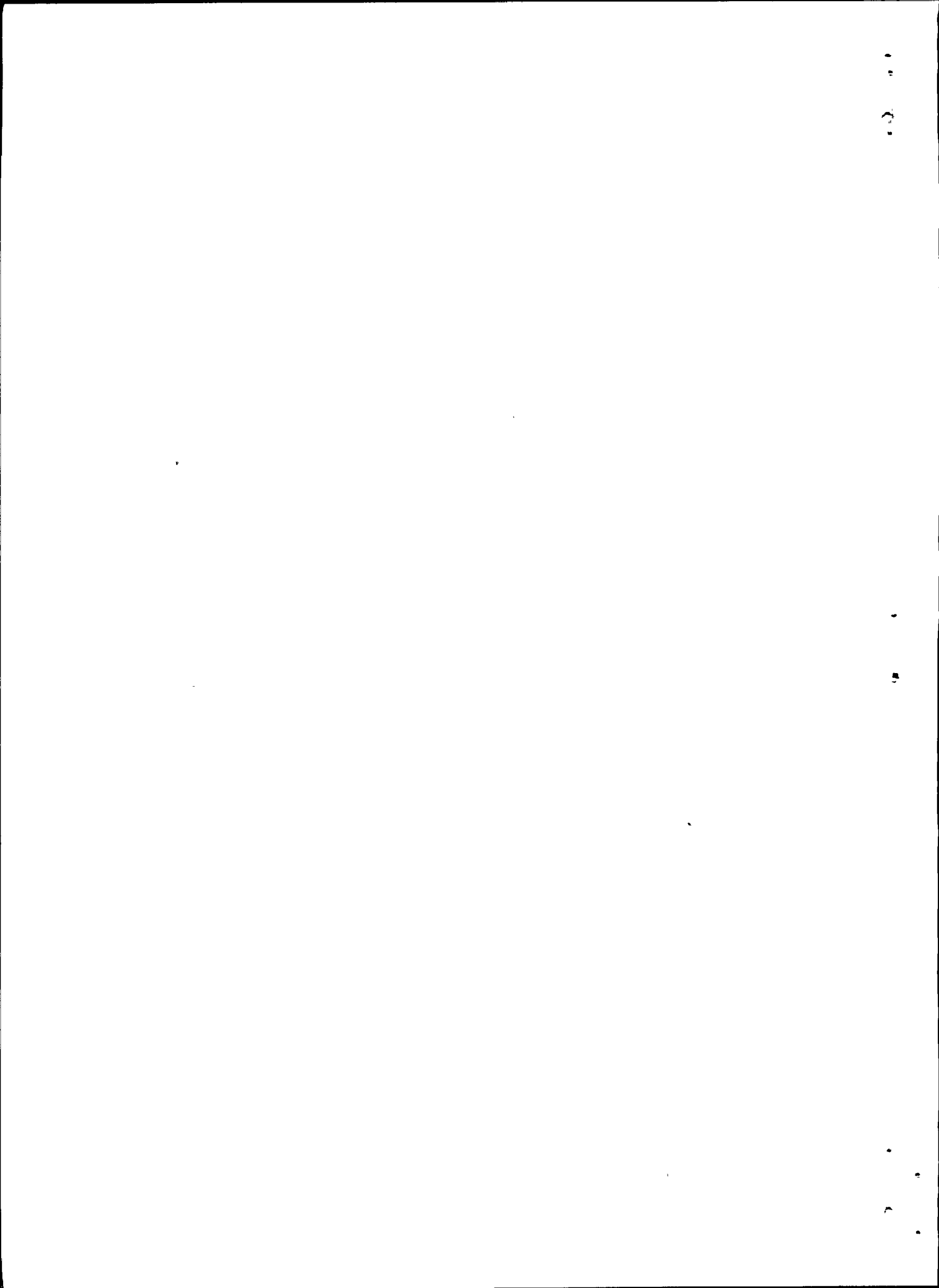
<u>Figure</u>	<u>Page</u>
* 25. The breakwater at the Oil Harbour of Bilbao	34
26. The Oil Harbour site, Bilbao	34
27. Cross-section of the breakwater, Bilbao	34
28. Armour layer profiles recorded 860 m from nearshore end of breakwater, Bilbao	34
29. Wave conditions at the breakwater during December 1976, Bilbao	36
30. San Ciprian breakwater cross-section	36
* 31. Crescent City Harbor, California	37
* 32. Damaged original extension of Crescent City outer breakwater--viewed from station 36+70 with Round Rock in distance (upper right)	37
* 33. Tetrapod repairs to realigned extension of Crescent City outer breakwater	37
* 34. Storm wave displacement of originally submerged tetrapod from repaired area shown on Fig. 9, Crescent City	39
* 35. Wave trench resulting from the overtopping shown on Fig. 9--Crescent City outer breakwater	39
* 36. Humboldt Harbor and Bay, California	39
37. Jettied entrance to Humboldt Harbor and Bay during storm activity of February 1960	40
* 38. Monolithic breakup--Humboldt Harbor and Bay north jetty	40
39. Storm damage of Kahului breakwater	43
* 40. Typical original rubblemound breakwater cross section, Kahului	43
* 41. Typical tetrapod breakwater head cross-section, Kahului	43
* 42. Kahului Harbor	45
* 43. Damage to east breakwater, November 1958, Kahului	45
44a. Typical tribar breakwater head section, Kahului	45
44b. Typical tribar trunk section, Kahului	45
* 45. Photo of damage at Gansbaai breakwater, after storm of December 1977	49
* 46. Comparison of damage to prototype and model breakwaters: 15-ft. waves, San Pedro Bay	51





<u>Figure</u>	<u>Page</u>
47. Comparison of damage to prototype and model breakwaters: 21-ft. waves, San Pedro Bay	51
48. Comparison of wave attack in prototype and corresponding model on breakwater at Silver Bay Harbor, Minnesota	53
49. Final design cross-section of breakwater, Port Sines	56
50. A typical cross-section as observed by divers, Port Sines	57
51. Plan and longitudinal section of main breakwater after February 1978 storm, Port Sines	59
52. Mean profiles after February 1978 storm, Port Sines	59
* 53a. February 1978 storm damage: Main damage area between berth no's 2 and 1, 12/6/78, Port Sines	60
* 53b. February 1978 storm damage: Damage to dolos armour between berth no's 2 and 3, 12/6/78, Port Sines	60
54. Columbia River at mouth, Oregon and Washington	63
55. Mouth of Columbia R., Oregon & Washington, south jetty repairs, 1933	64
56. Mouth of Columbia River, Oregon and Washington, south jetty terminal. Profile and plan	65
57. Mouth of Columbia River, Oregon and Washington, south jetty terminal. Sections	66
58. Mouth of Columbia River south jetty. Profile and top width from Clatsop Spit westward, May 2-3, 1931	68

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# BREAKWATER DAMAGE BY SEVERE STORM WAVES AND BY TSUNAMI WAVES

by

Robert L. Wiegel

5 March 1982

## INTRODUCTION

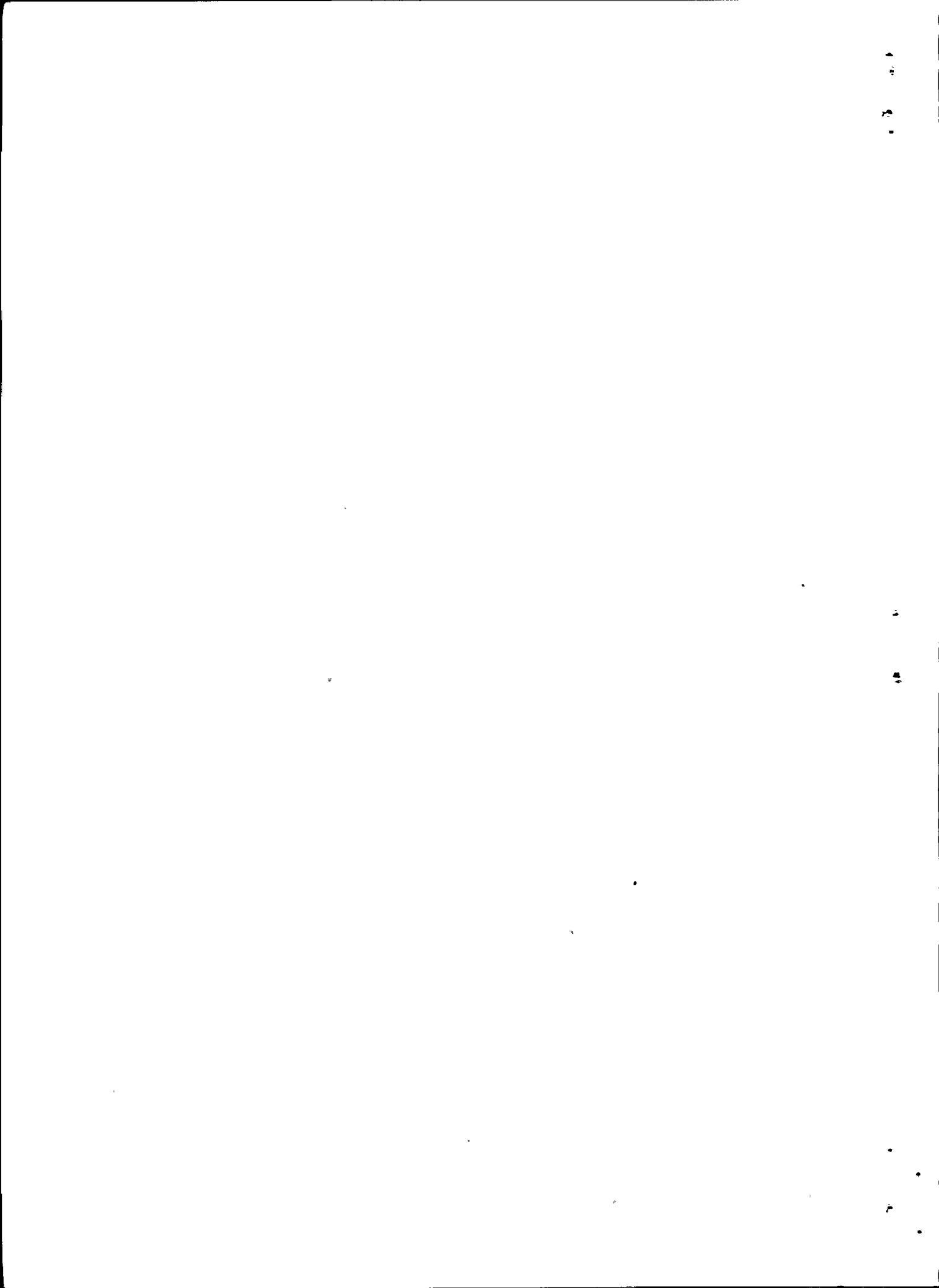
The purpose of this report is to determine the amount and type of degradation that occurs to a breakwater through the action of extremely severe storm waves, or tsunami waves, and to determine the functional usefulness of breakwaters after being severely damaged. Emphasis is given to observations of such effects on actual structures in the ocean.

A number of breakwaters have suffered damage through the action of storm waves.\* Generally the damage as observed from the surface appears to be much greater than it actually is. The material moved by wave action off the above water portion of a breakwater has been moved either over the top of the breakwater and deposited on its lee side, or has been moved down the breakwater slope on the ocean side. On some occasions the rock and whole or broken pieces of cast concrete armor units were moved up the ocean side slope to form a rubble berm near the top of the breakwater. Two important conclusions may be drawn from the observations. The first is that the displaced material still forms a part of the breakwater. The second is that degradation appears to stabilize when the top of the degraded section reaches about the low water level (MLLW).

Although many breakwaters have been damaged by severe storm waves, few have been damaged to any large extent by tsunami waves. Partly this is due to the relative rarity of tsunami waves, and partly it is due to the

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\* A computer search was made, using a number of sources. These were in addition to a large number already known to the writer.



fact that overtopping of breakwaters by tsunami waves does not last long. It is also due to the fact that tsunami waves are very long, the breakwater slope is steep, and usually the water offshore is fairly deep;\* this combination is such that it is unlikely that those waves will break against the structure. An exception to this was the breakwater at Kodiak, Alaska, where a great tidal flat existed and the main tsunami wave was of great height. Apparently it moved as a bore.

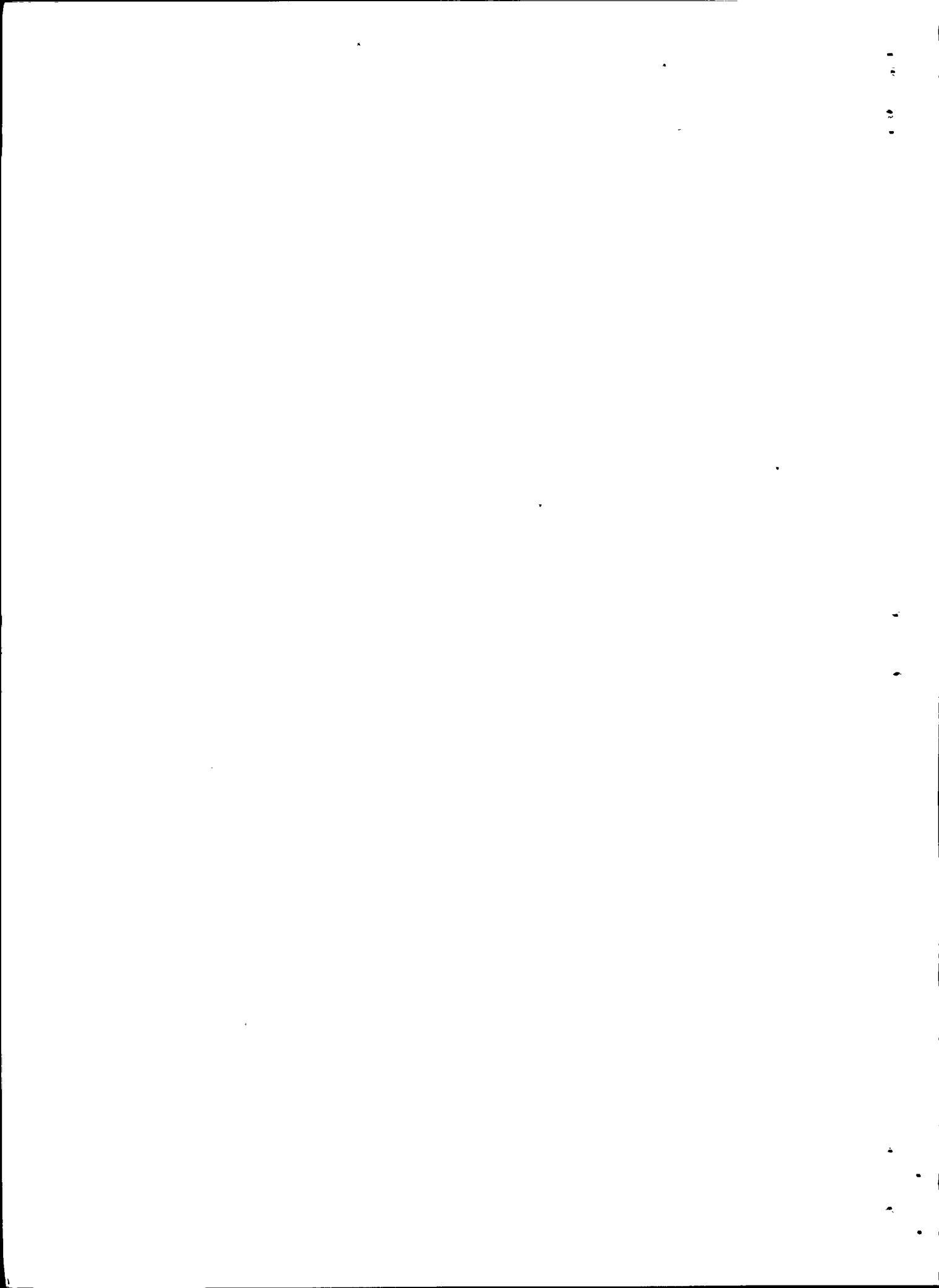
Examples will be given of the type of damage that has been done to breakwaters by severe wave action. Little data are available to determine the effect of a series of extreme events on such degradation. This mostly is because breakwaters are usually repaired within a few years after the damage has occurred. One long-term example is given, that of the south jetty (a breakwater essentially normal to the shore line) at the entrance to the Columbia River. The end few thousand feet had no repair work done to it for several decades and during that time was subject to a number of severe storms.

Almost no information is given in the technical literature on the general problem of capital costs versus maintenance costs. However, the little information on this subject that is given, together with the numerous statements made at technical meetings on breakwater design, construction and maintenance lead to the conclusion that nearly all breakwaters are designed on the basis that it is cheaper to have to repair them occasionally than to design and construct them to withstand some maximum hypothesized event.

Some papers and reports were found that described the relationship between physical model studies and observed damage to breakwaters under severe wave conditions. This information is presented and it emphasizes the

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\*This is the case at Diablo Canyon, CA.



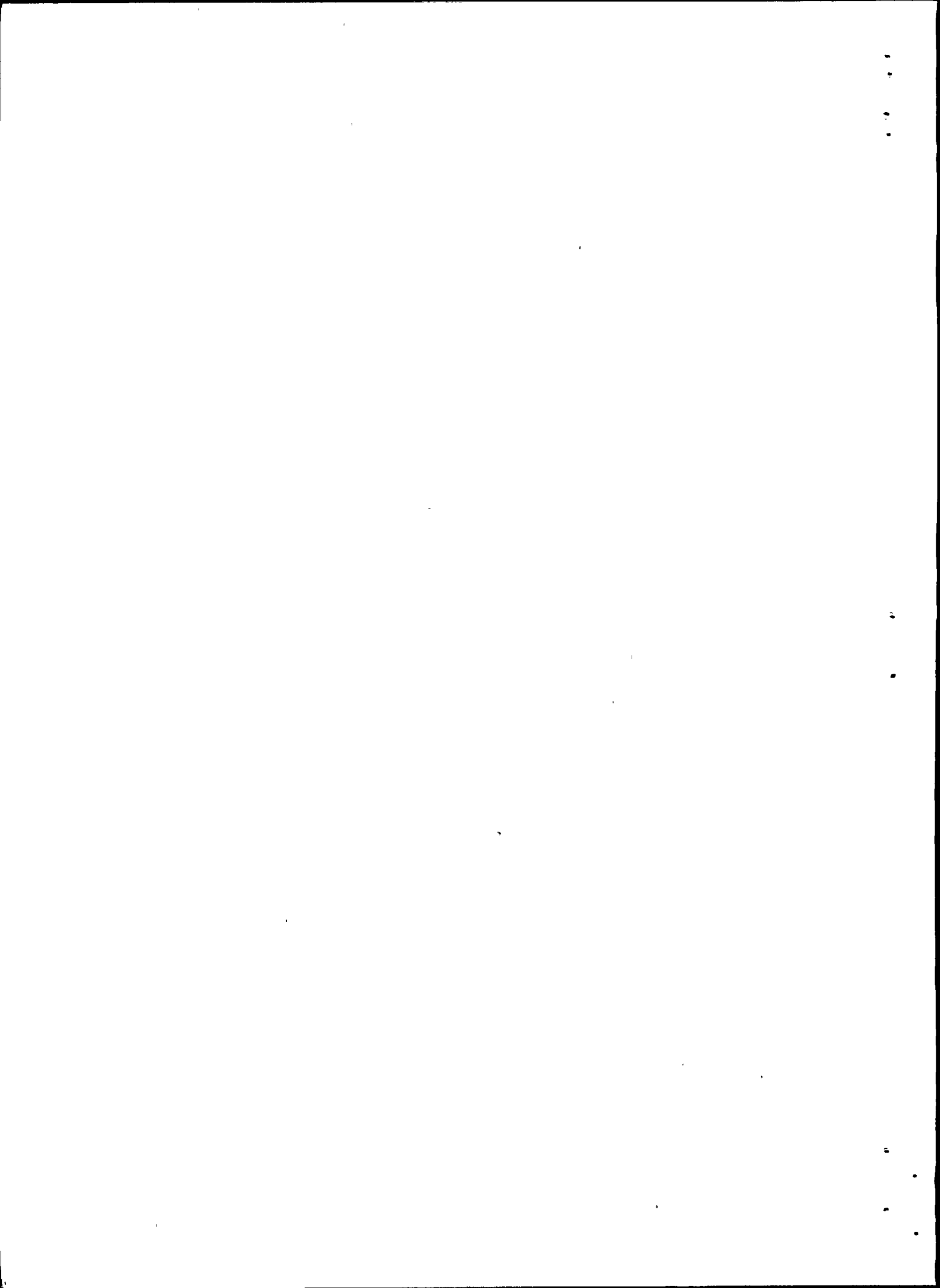
great value of performing appropriate physical model studies.

During the course of the study presented herein, the writer visited Diablo Canyon on two occasions to see the breakwater in its damaged condition, and also visited twice the model basin at Escondido to see the hydraulic model study being made of the breakwater.

### CONCLUSIONS

The following conclusions may be drawn from the available historical data on breakwaters damaged by storm and tsunami waves.

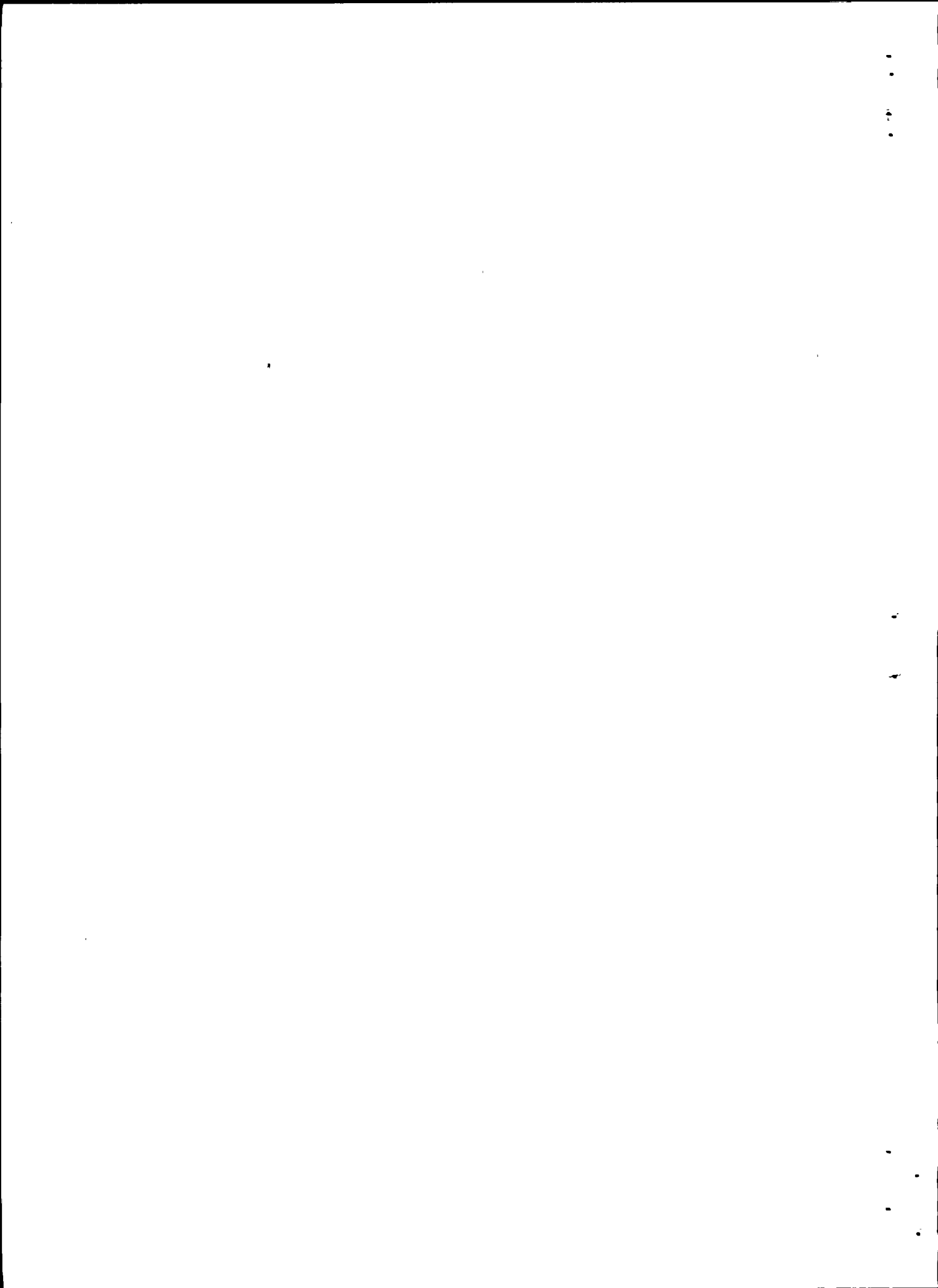
1. Historical evidence shows that breakwaters have not been damaged by tsunami waves unless substantial overtopping occurs.
2. It is extremely unlikely that damage would result to the breakwaters at Diablo Canyon, California, from the action of one of the great tsunamis that might occur in the Pacific Ocean unless the breakwaters were already in a severely degraded condition. That is, the degraded crest of the breakwater (or breakwaters) would have to be so low that the tsunami waves could overtop them.
3. If the breakwaters at Diablo Canyon, California, are not in a storm-damaged condition if and when they are attacked by the hypothesized locally generated tsunami waves, the tsunami waves will not overtop them.
4. The available information on breakwaters that have been damaged severely by great storm waves show that the breakwaters are not degraded by such a storm too much, if at all, below the low water level. This appears to be true even for cases where they have been subjected to additional great storms during the same year or the succeeding year. Thus, the initial degradation of a breakwater tends to become stabilized when the crest elevation reaches about the low water level. Further degradation proceeds slowly over the years. This has been the case at Diablo Canyon since the major damage done on 28 January 1981.





Only one example was found in the technical literature of a breakwater which was subject to long term (several decades) degradation by storm waves. This is the south jetty at the mouth of the Columbia River. It is in an extremely exposed location. Several thousand feet of the end of it have received no repairs for several decades. This section has degraded to about low water, except for the very end. There is conflicting information about the very end, but it slopes into water below the low water level and eventually, of course, to the bottom. The actual depths beneath the low water level are not known, as it appears that no actual survey was made of this portion of the breakwater.

5. Armor stone and cast concrete armor units (often broken) are moved by great storm waves. Depending upon the type and construction of the breakwater, the local bathymetry and the wave characteristics, this material may be moved into the lee side of the breakwater, down the seaward slope of the breakwater, or even up the seaward slope of the breakwater to form a small berm; the important point is that the rock and/or concrete is still at the site and performs a useful part of the function for which the breakwater was built. This appears to be the case with the breakwaters at Diablo Canyon, as was observed during the storm of 28 January 1981 and also during the 1981 - 1982 winter storms.
6. World-wide experience with rubble mound breakwaters has been such that owners must plan on occasional damage occurring to the breakwaters by the action of severe storm waves. This will require expensive repair work. It is usual in the economic planning of breakwater installations to consider maintenance costs and the costs for downtime as well as the initial construction costs.



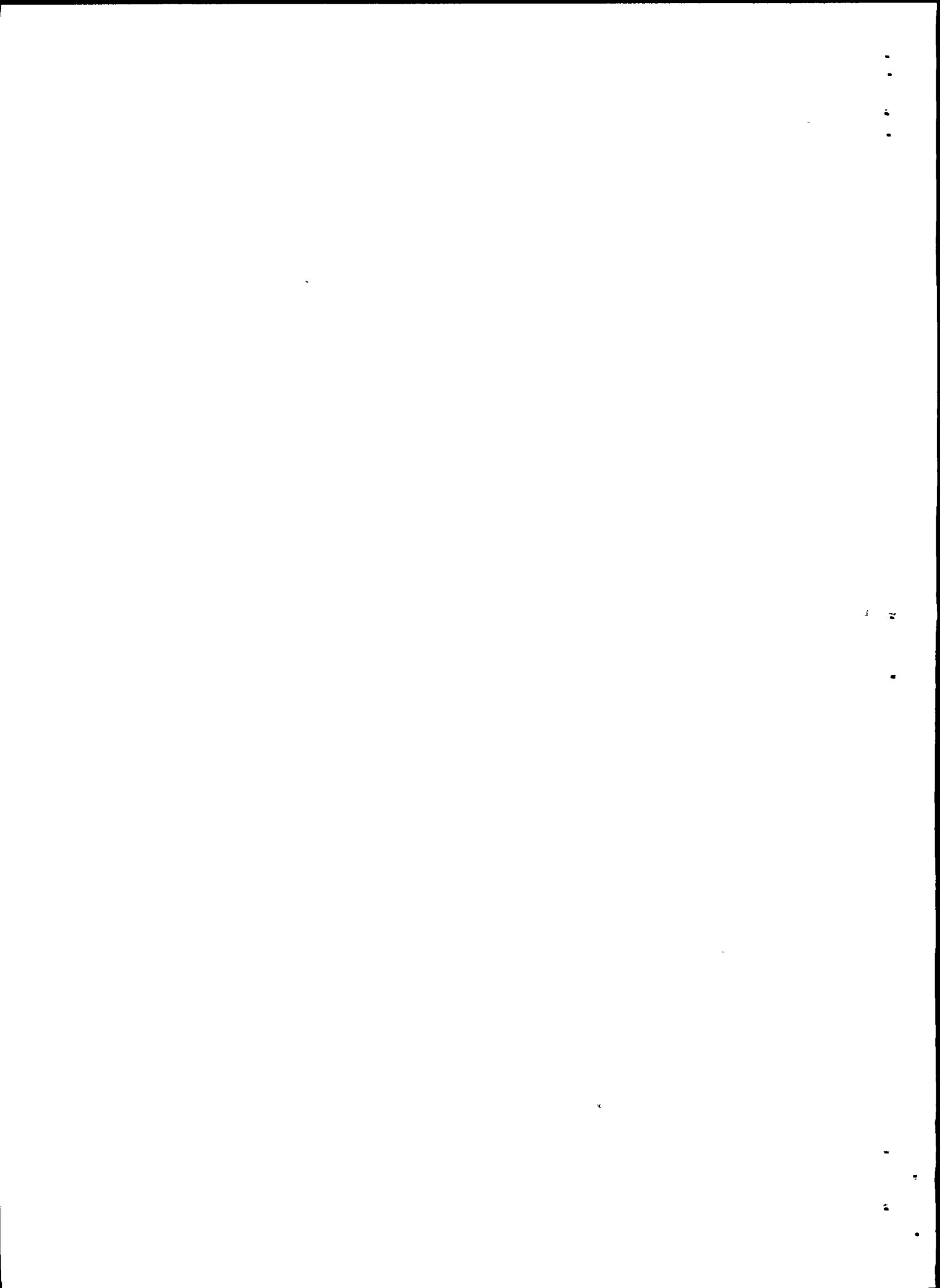
7. Well planned and executed physical model studies have been proved to be very useful, and have been used often in the past. They are a necessary part of the design or redesign for rubble mound breakwaters because the complexity of the physical processes is beyond our ability to formulate them properly by means of mathematical models.

#### TSUNAMI DAMAGE TO BREAKWATERS

Within the U.S., the first detailed study of tsunami damage was the one made just after the great 1 April 1946 tsunami, which originated in Alaska. Considerable damage was caused by this tsunami in the Hawaiian Islands. Shepard, MacDonald and Cox (1950, p. 458) report on breakwater damage as follows:

"Most breakwaters were not damaged, or only very slightly damaged. Only the Hilo breakwater (pl. 27,b),\* which has often been damaged by storm waves, suffered greatly during the tsunami. Of the part of the breakwater above sea level, 6,040 feet, or about 61 per cent, was destroyed. The cap and outside face were composed of rocks weighing 8 tons or more, and the inside face of rocks weighing 3 tons or more. The rocks were thrown both shoreward and seaward by the waves. The average depth of scour in the gaps in the breakwater was 3 feet. The breakwaters at Ahukini and Nawiliwili on Kauai, at Kahului on Maui, and even at Hilo, in spite of the damage, probably reduced greatly the severity of the attack of the waves inside the harbors. The effect is easily shown by the drop in wave heights from 17 and 22 feet outside the breakwaters at Kahului to 7 and 11 feet inside the breakwaters, and from 29 feet outside at Hilo to an average of about 21 feet inside. The effect was similar to that of a coral reef with fairly deep water behind it."

\* A copy of their Plate 27,b is reproduced herein as Figure 1.



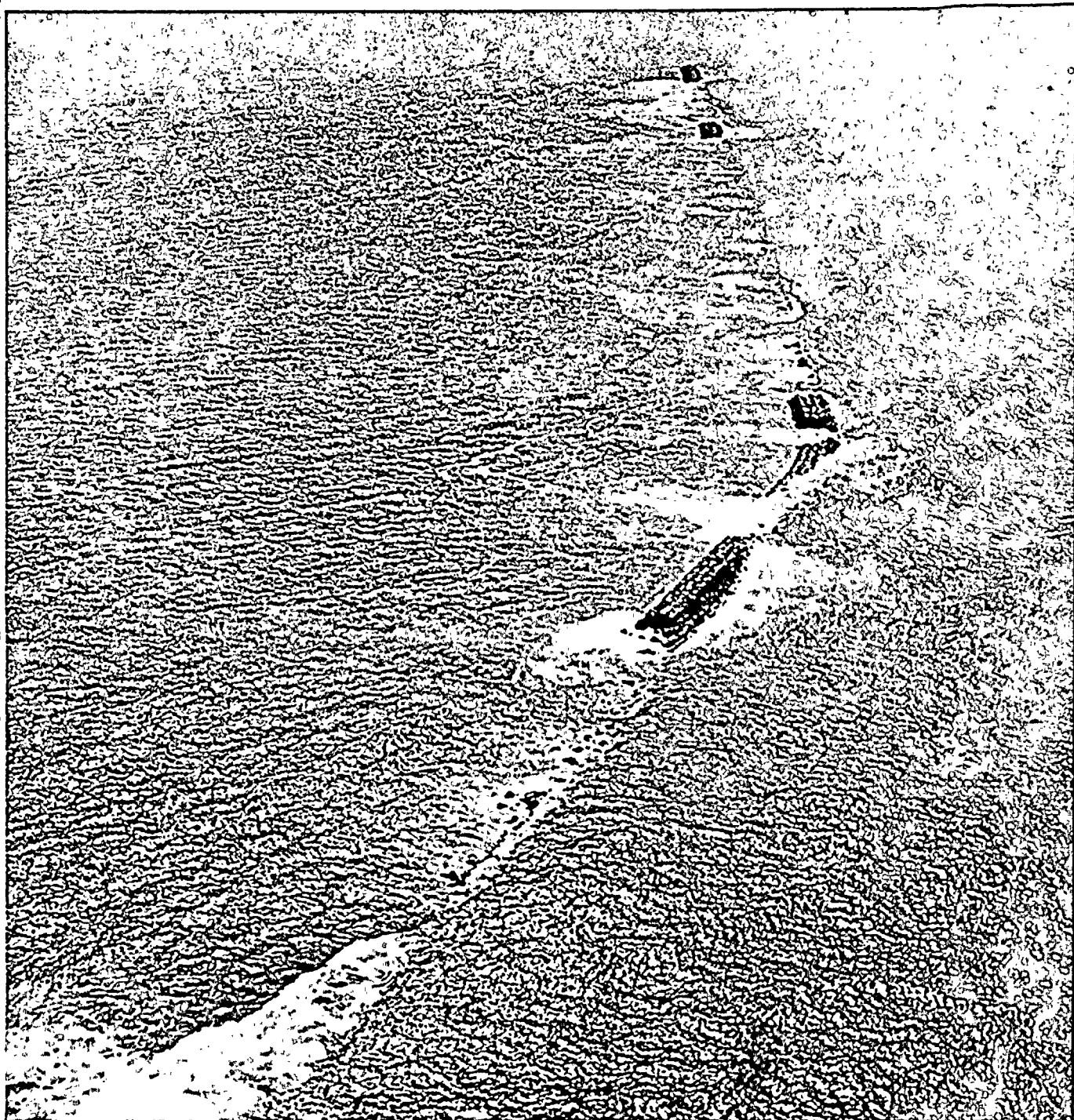
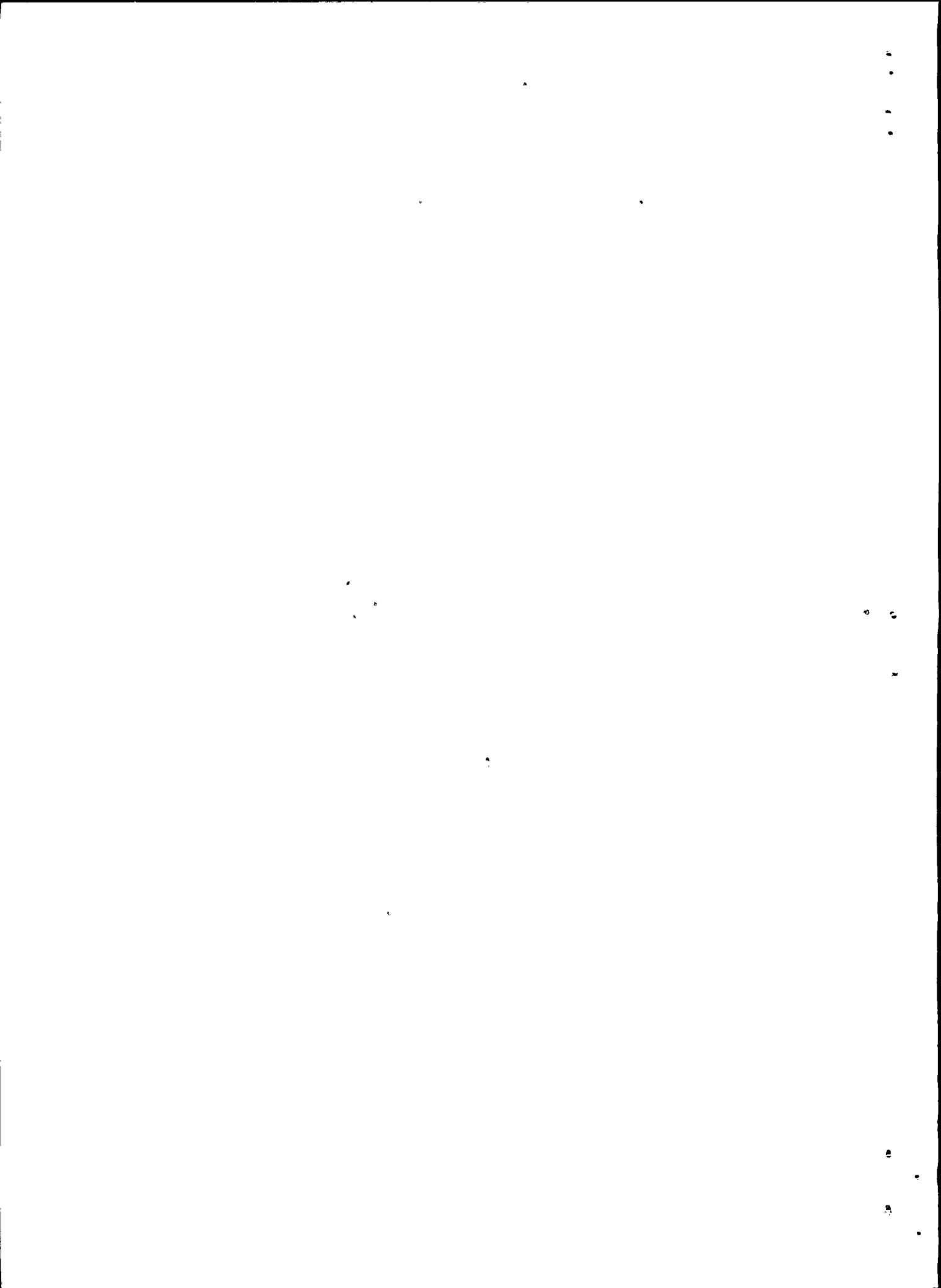


Figure 1. Breakwater at Hilo, Hawaii after the  
1 April 1946 tsunami



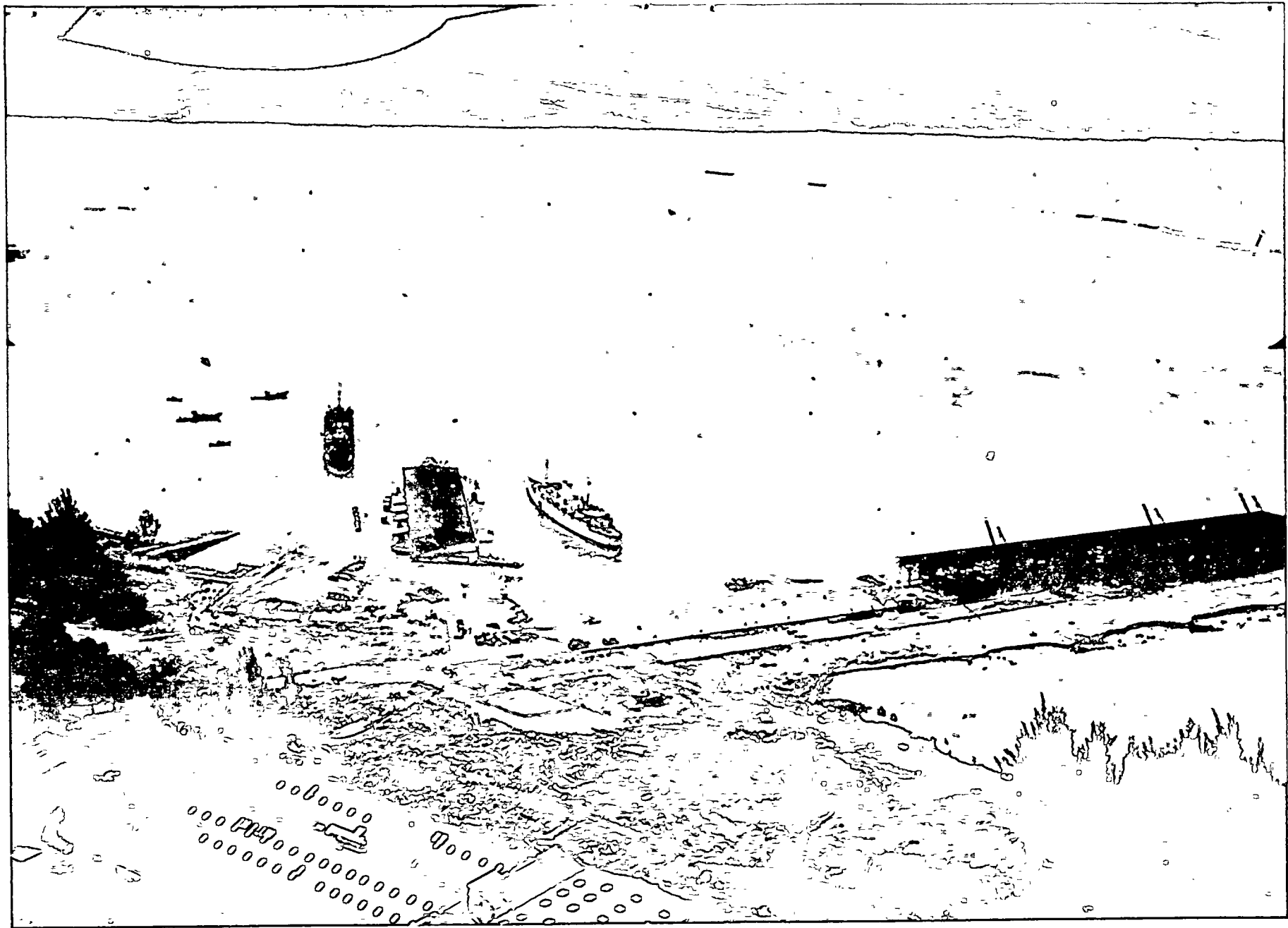


Figure 2. Breakwater and port at Hilo, Hawaii after the 1 April 1946 tsunami

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On page 441 of the same report, they state:

"East of the breakwater, at its landward end, the water rose 29 feet; at Pier 1, just inside the breakwater, it rose 27 feet. Westward from Pier 2 the wave heights rapidly decreased to less than 10 feet in Reed's Bay. This decrease appears to have been at least partly the result of protection of this part of the shore by the landward end of the breakwater and the docks.

"The part of the breakwater which projected above sea level was more than half destroyed (pl. 27,b), but destruction generally extended to a depth of only 2 or 3 feet below sea level; as on the beaches, the force of the waves was exerted mainly at high levels. Although it suffered severely from the wave onslaught, the breakwater undoubtedly played an important part in lessening the severity of the waves in the head of Hilo Bay. Had there been no breakwater, water levels would probably have been several feet higher."

Another view of the damaged breakwater is shown in Figure 2.

A report was made in 1960 by the U. S. Army Engineer District, Honolulu, Hawaii, in which the following statement was made on page 16.

"There is strong evidence that damage to breakwaters located at, or seaward of, the entrance to estuaries is primarily the result of overtopping. The 1946 tidal wave was the only one to severely damage the Hilo breakwater since it was completed in 1930. Even then, about 40 percent of the structure was undamaged, indicating that an increase in the stability and height of the structure would have prevented the damage. The more than 10 other breakwaters in the Hawaiian Islands have been undamaged or only slightly damaged by tidal waves. The breakwaters at Kahului Harbor, Maui, are directly exposed to phenomena originating in the Aleutians but were only slightly damaged by overtopping in April 1946."

On page A-21 of this report it is stated

"Observers at the Wailuku bridge during the last tidal wave indicated that the wave was about 20 feet high at the breakwater location. Studies made of the photographs of breakwater damage in 1946 indicate that overtopping of the breakwater by tidal waves was the major factor in the destruction of the structure. This is evident by the displacement of stone from partially damaged sections of the breakwater. Although the elevation was based on wave analysis, it is probable that this elevation would also be adequate for tidal waves. The elevation of the dike was selected to provide the same degree of protection as for the harbor area."

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"... The existing breakwater would be modified by raising the crest elevation from 13 feet to 20 feet."

Two cross sections of the "existing" breakwater (1960) and proposed modified cross sections are shown in Figure 3 (which is from their Plate A-1, following page A-24).

In Appendix C of this same report (on pp. C-31 and C-32) they mention that the damage to the Hilo breakwater due to the 1 April 1946 tsunami was \$2,090,000 (September 1960 price level). They list no damage to the breakwater by the 4 November 1952, 9 March 1957, or 23 May 1960 tsunamis, although information of damage to other types of structures was given.

In another report by the U. S. Army Engineer District, Honolulu (1967), in regard to the 23 May 1960 tsunami, it is stated that at Hilo, Hawaii

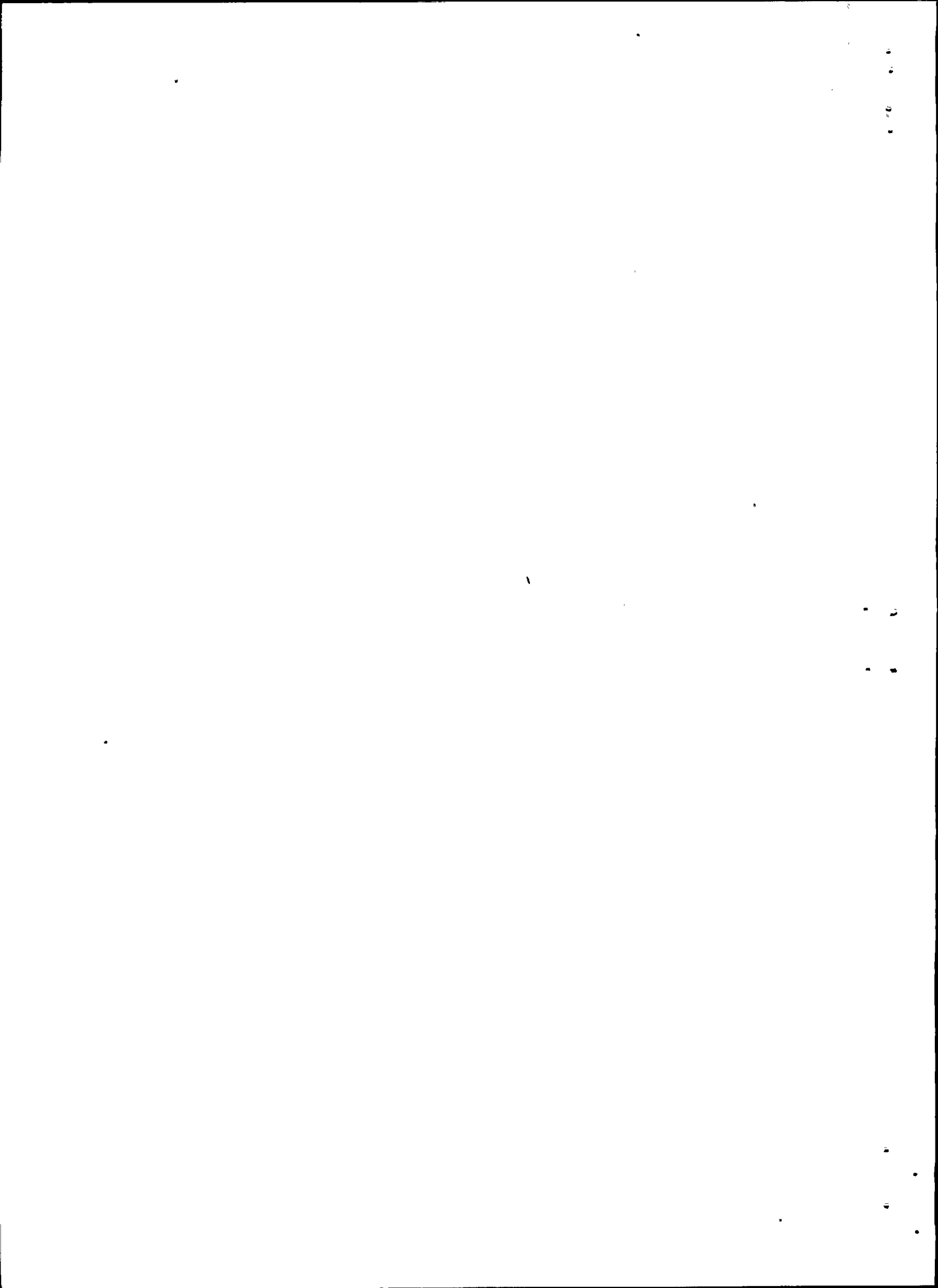
"The breakwater, which was rebuilt after the 1946 tsunami, suffered only minor damage."

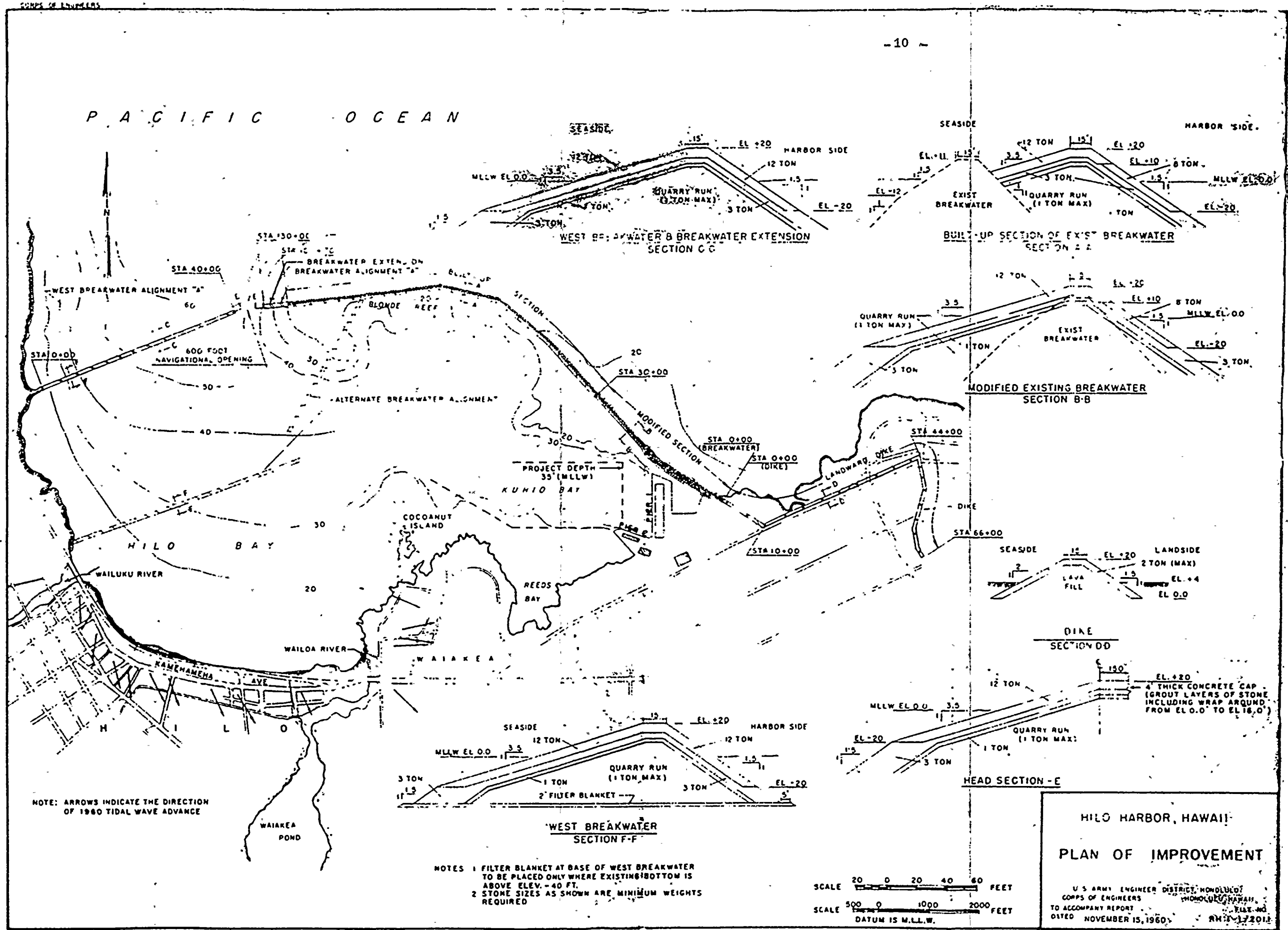
In regard to the 1 April 1946 tsunami, they state on page 19 that at Hilo, Hawaii

"At the root of the breakwater near the pier area, the water rose to +29 feet MLLW, and at pier 1, where a measurement was taken at +27 feet MLLW, the solid water elevation was evaluated to be about +17 feet MLLW. The 2-mile long breakwater was 60 percent destroyed down to a depth of 2 to 3 feet below sea level."

For the record, in the above report, there is a citation to a 1962 report by the U. S. Army Engineer District, Honolulu, on the 1960 tsunami. The writer was not able to locate a copy of this report.

A survey was made of the coast of northern California by Orville T. Magoon after the 1960 tsunami (Magoon, 1962) and after the 1964 tsunami (Magoon, 1965). No damage to breakwaters was cited in either of these two reports. In regard to the 1960 tsunami at Crescent City, CA, Magoon (1962) states:





NOTE: ARROWS INDICATE THE DIRECTION OF 1960 TIDAL WAVE ADVANCE

- NOTES
- 1 FILTER BLANKET AT BASE OF WEST BREAKWATER TO BE PLACED ONLY WHERE EXISTING BOTTOM IS ABOVE ELEV. -40 FT.
  - 2 STONE SIZES AS SHOWN ARE MINIMUM WEIGHTS REQUIRED

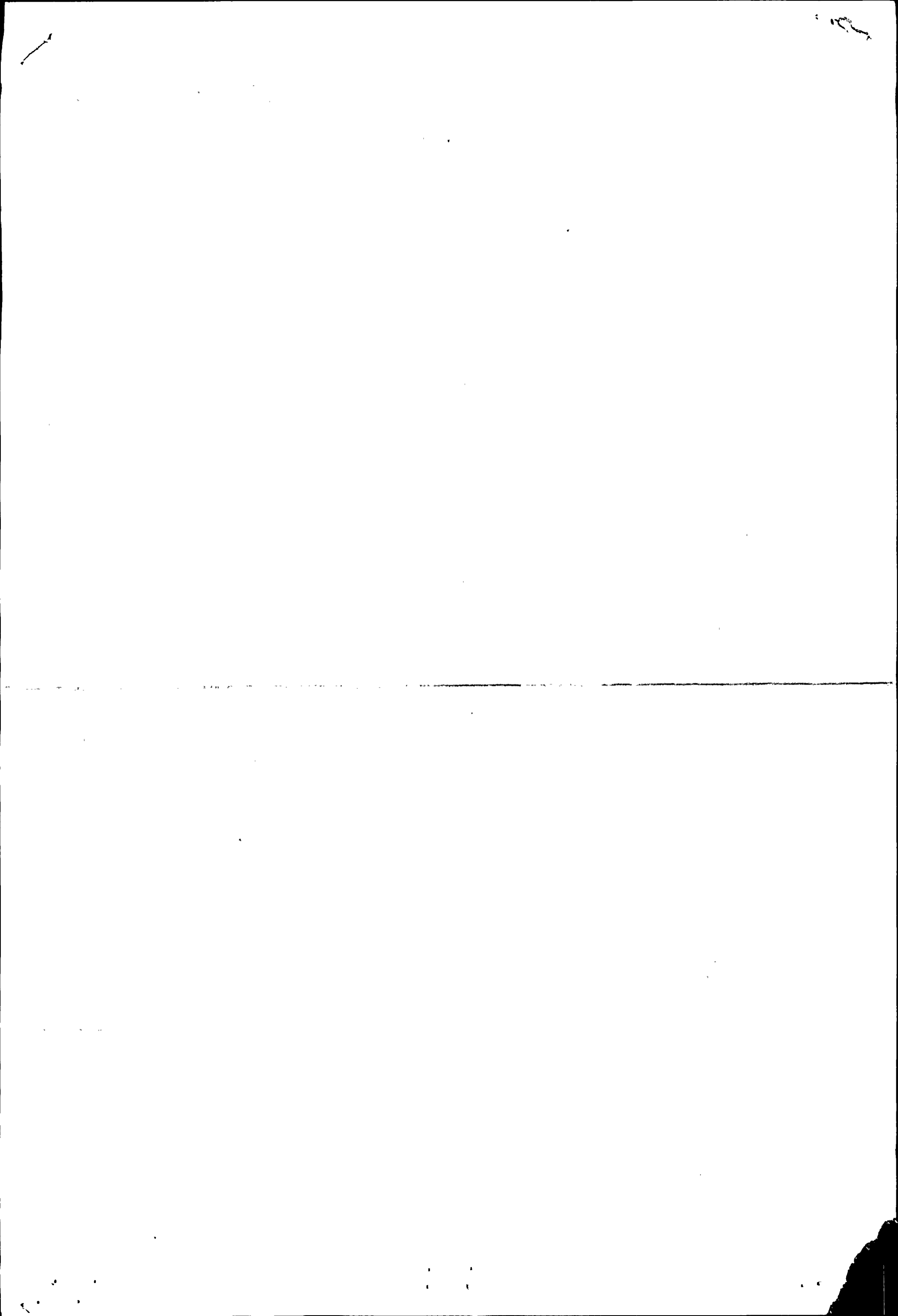
SCALE 20 0 20 40 60 FEET

SCALE 500 0 1000 2000 FEET

DATUM IS M.L.L.W.

(From U. S. Army Engineer District, Honolulu, 1960)

FIGURE 3



"A survey of the harbor taken a week after the initial waves indicated no significant changes in the harbor basins, but a channel was scoured approximately 80 feet wide and 2 feet deep near the seaward ends of the inner breakwater. No damage to the rubble mound structures at Crescent City was observed."

In this same report he mentions that the Half Moon Bay, CA, breakwater was under construction at the time, but does not mention any damage being done to it by the tsunami.

Considerable damage was done to structures in Japan by the 1960 Chile tsunami (Iwasaki and Horikawa, 1960; Horikawa, 1961). The only damage to a breakwater cited was (Iwasaki and Horikawa, 1960, p. 11): "At the port of Hachinoe, a breakwater near the river mouth subsided by 0.5 m; ..."

Tudor (1964) states the following in regard to the breakwater at Crescent City, CA, after the 1964 Alaska tsunami:

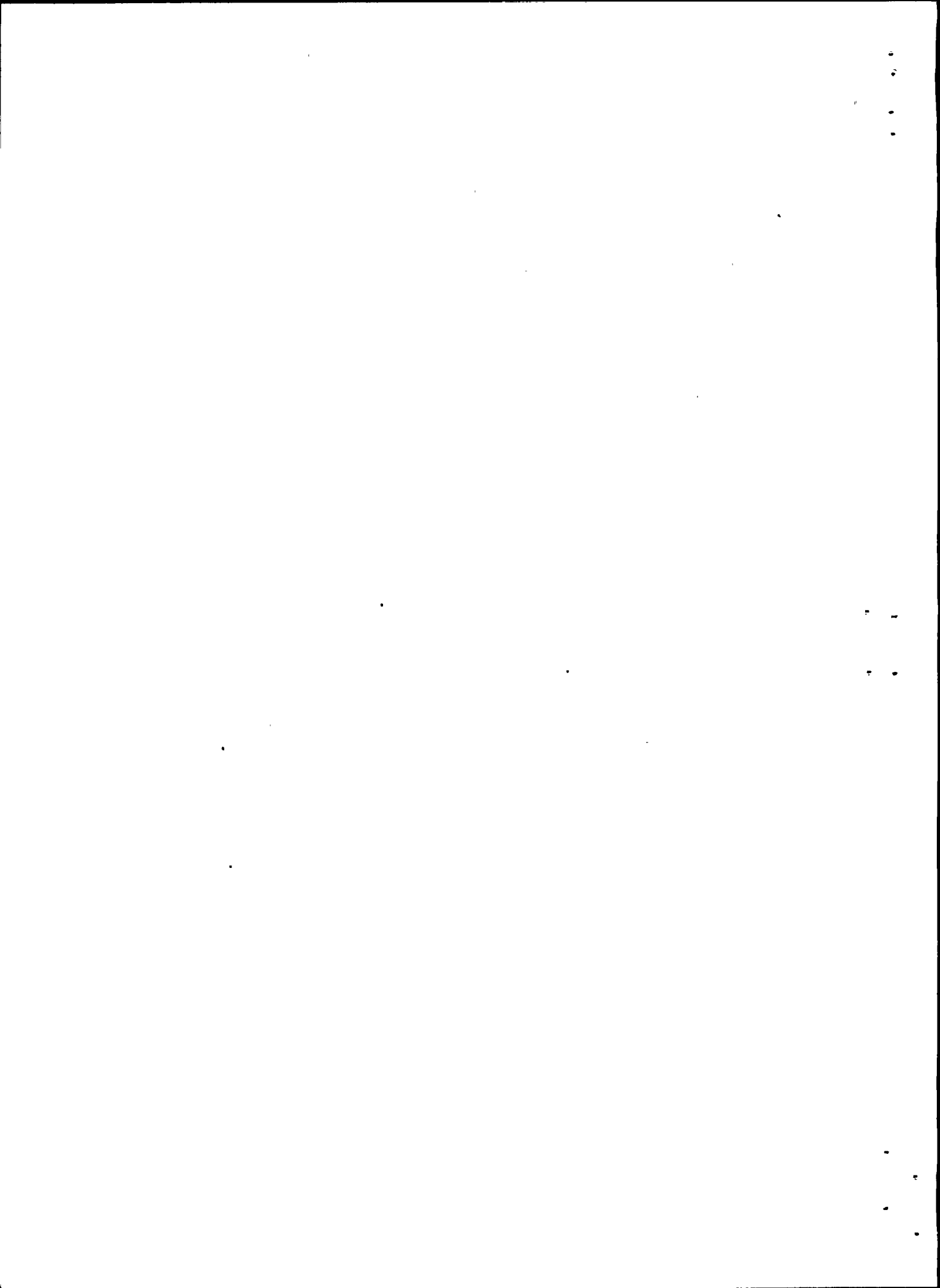
"Some 1975 of these tetrapods were used in construction of Crescent City's breakwater where they were placed on the seaward side of the outer breakwater (Figure 38)\* by the Corps of Engineers in 1956-57 and so interlaced as to keep heavy seas from destroying the breakwater. This breakwater was reportedly overtopped by smooth flowing tsunami water resembling river or weir flow. The surface may have reached 25 feet above MLLW at the outer end of the breakwater. The tetrapods on the breakwater remained intact after submergence by the tsunami waves (Figure 39).\* No visual damage was observed by the author walking along the caps of the outer breakwater, the inner breakwater and the sand barrier."

In regard to the breakwater at Kodiak, Alaska, after the 1964 earthquake, Tudor (1964) states:

"The rubble-rock breakwater is now practically submerged by the higher tides. This breakwater settled 3-1/2 feet from seismic disturbances in addition to the entire Kodiak Island 5-1/2-foot seismic subsidence."

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\* In original report.





A copy of an aerial photograph of the City of Kodiak, Alaska, is shown in Figure 4a.

Wilson and Tørum (1972) make the following comments regarding breakwaters that were subjected to tsunami waves by the 1964 earthquake:

"The breakwaters at Kodiak City were badly damaged, both by settlement during the earth tremors and by erosion from the tsunami. They were built primarily to protect the harbor from locally generated wind waves, and the armor stones were not large enough to resist high-velocity scour. At Seldovia, Kenai Peninsula, the breakwaters, which were also built to protect the harbor against local wind waves, also suffered damage from the earth tremors and the tsunami overtopping. The breakwater at Cordova, Prince William Sound, was apparently not damaged, and at Crescent City, California, the breakwaters, built partly with 25-ton tetrapods in the cover layer to protect against large storm waves from the Pacific Ocean, sustained no noticeable damage during the tsunami attack."

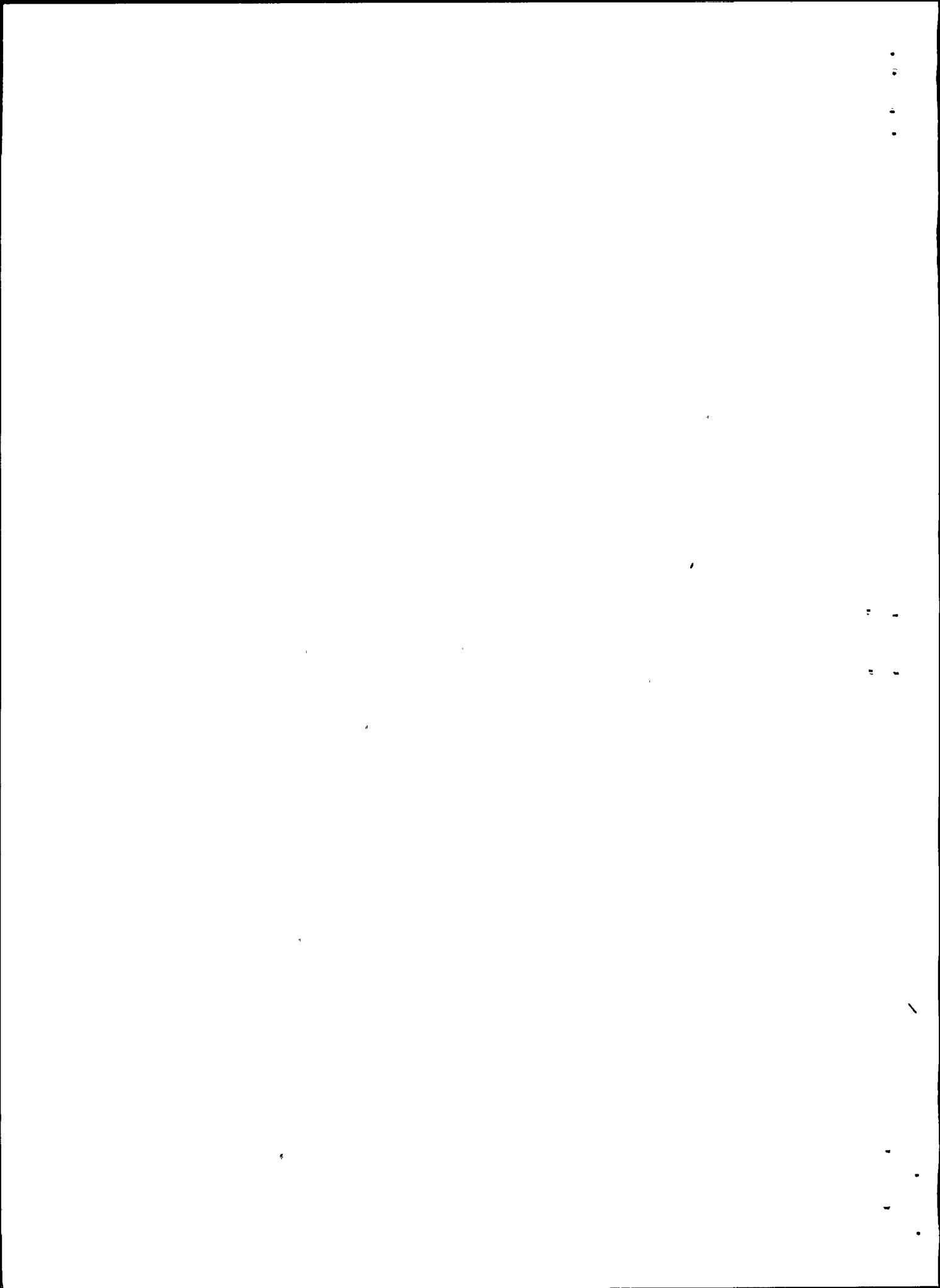
In another part of their report, Wilson and Tørum state:

"The damage to the breakwater was due partly to compaction settlement caused by the tremors and partly to the tsunami. Figure [4c] shows typical sections of the breakwaters as they were measured after the earthquake, as well as cross sections of the rebuilt breakwaters. The weight of the cover-layer stones and the core material of the breakwater are not exactly known. However, as judged from Figure [4b], the armor stones were quite light and would have been incapable of resisting any great degree of overtopping.

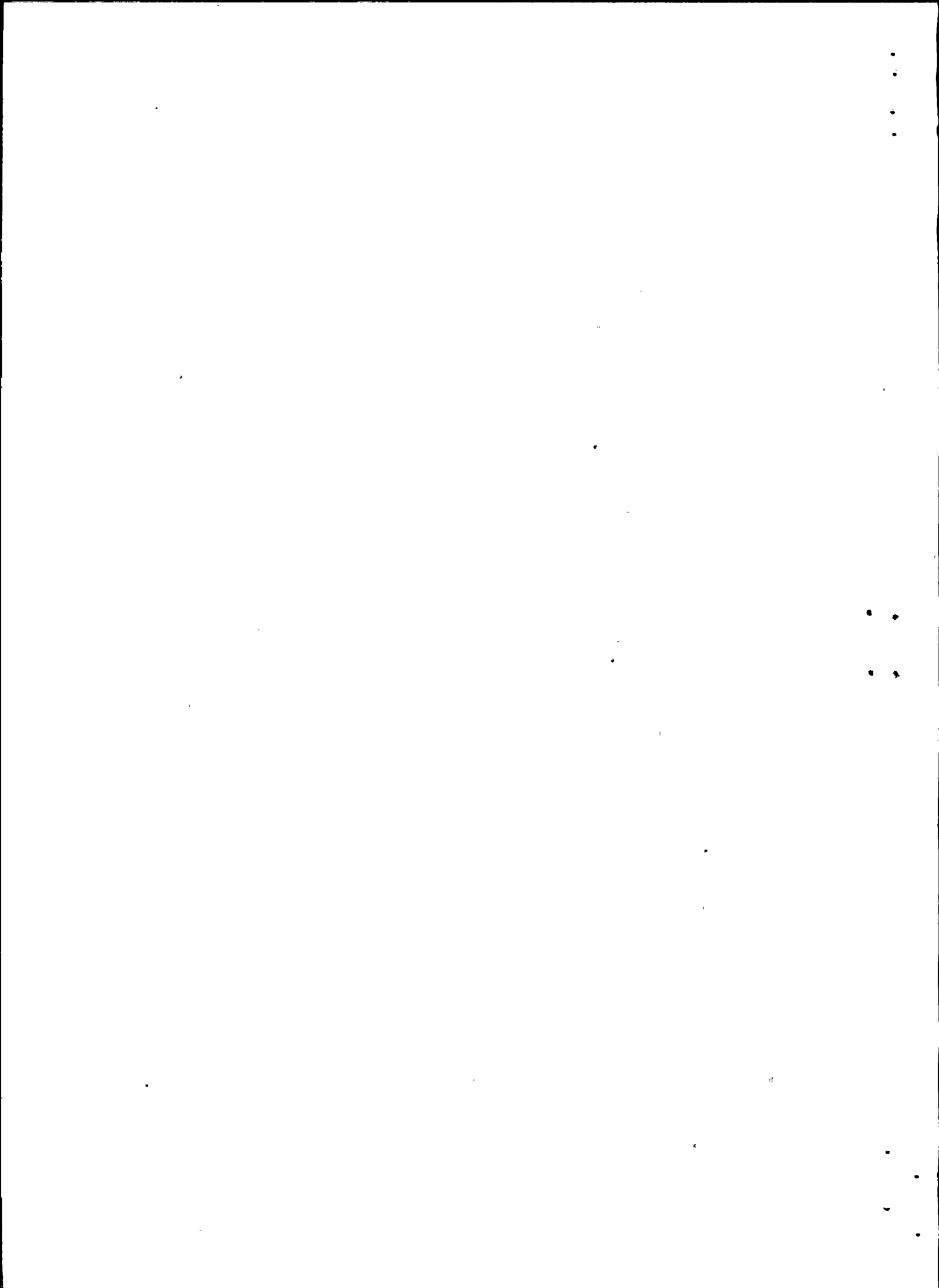
These tsunami waves at Kodiak were far greater than the waves hypothesized for Diablo Canyon, as can be seen from Figure 4d .

Although details of damage to boats, houses, etc., were given for many locations throughout California, no damage to breakwaters in the state was mentioned.

No information on damage to breakwater by tsunami waves has been given in the Corps of Engineers' report "Tsunami Engineering" (Camfield, 1980) other than some of the information cited above. No information on



breakwaters is given in the book "Seismic Sea Waves: Tsunamis" by T. S. Murty (1977). No information on breakwater damage is given in the 397-page book "The Chilean Tsunami of May 24, 1960" (Committee for Field Investigation of the Chilean Tsunami of 1960, Japan, 1961). No information was given in the papers in English in the 519-page (plus numerous plates) report on the 1933 Sanriku, Japan, tsunami (Earthquake Research Institute, 1934), nor were any photos of breakwaters included in the 209 photos in the volume.



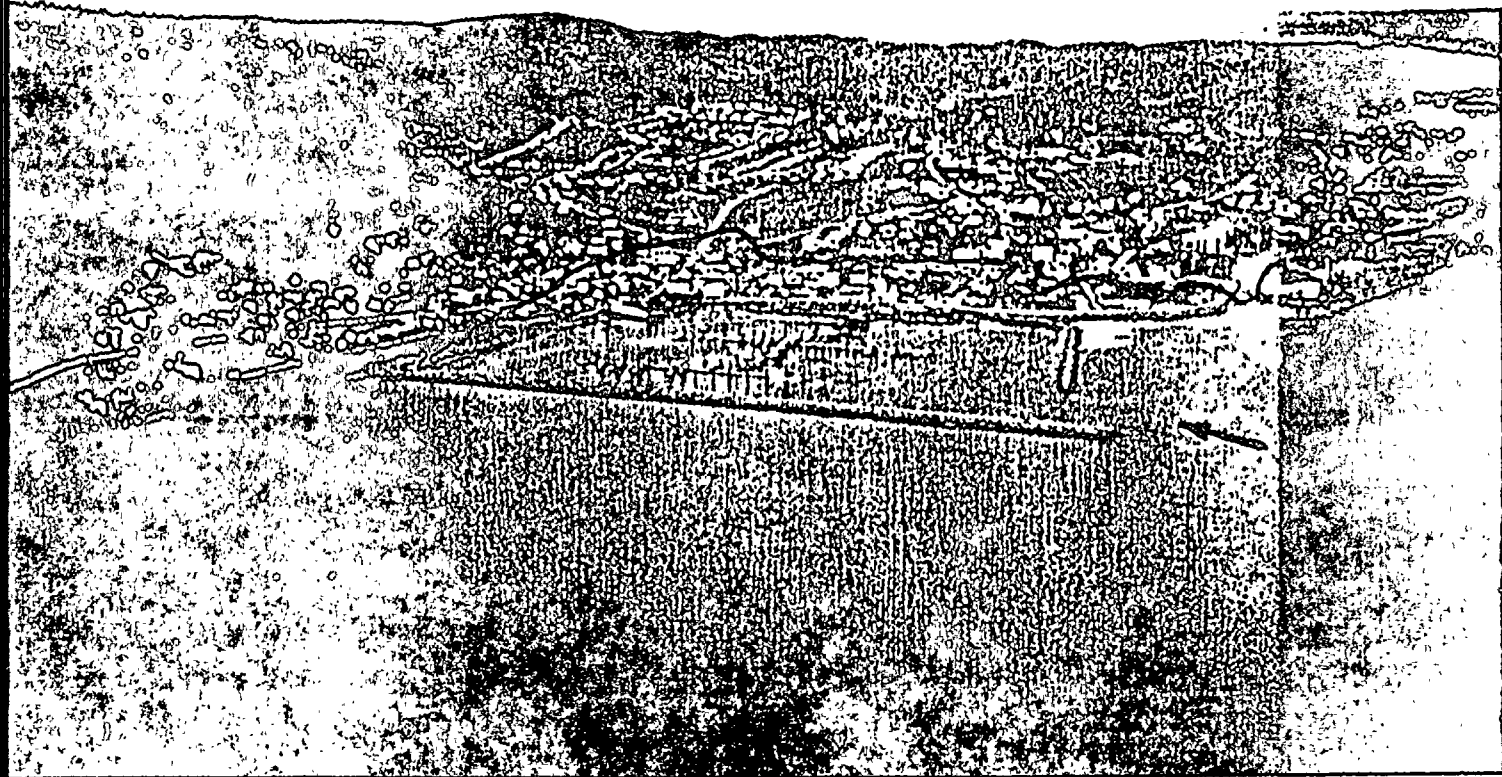
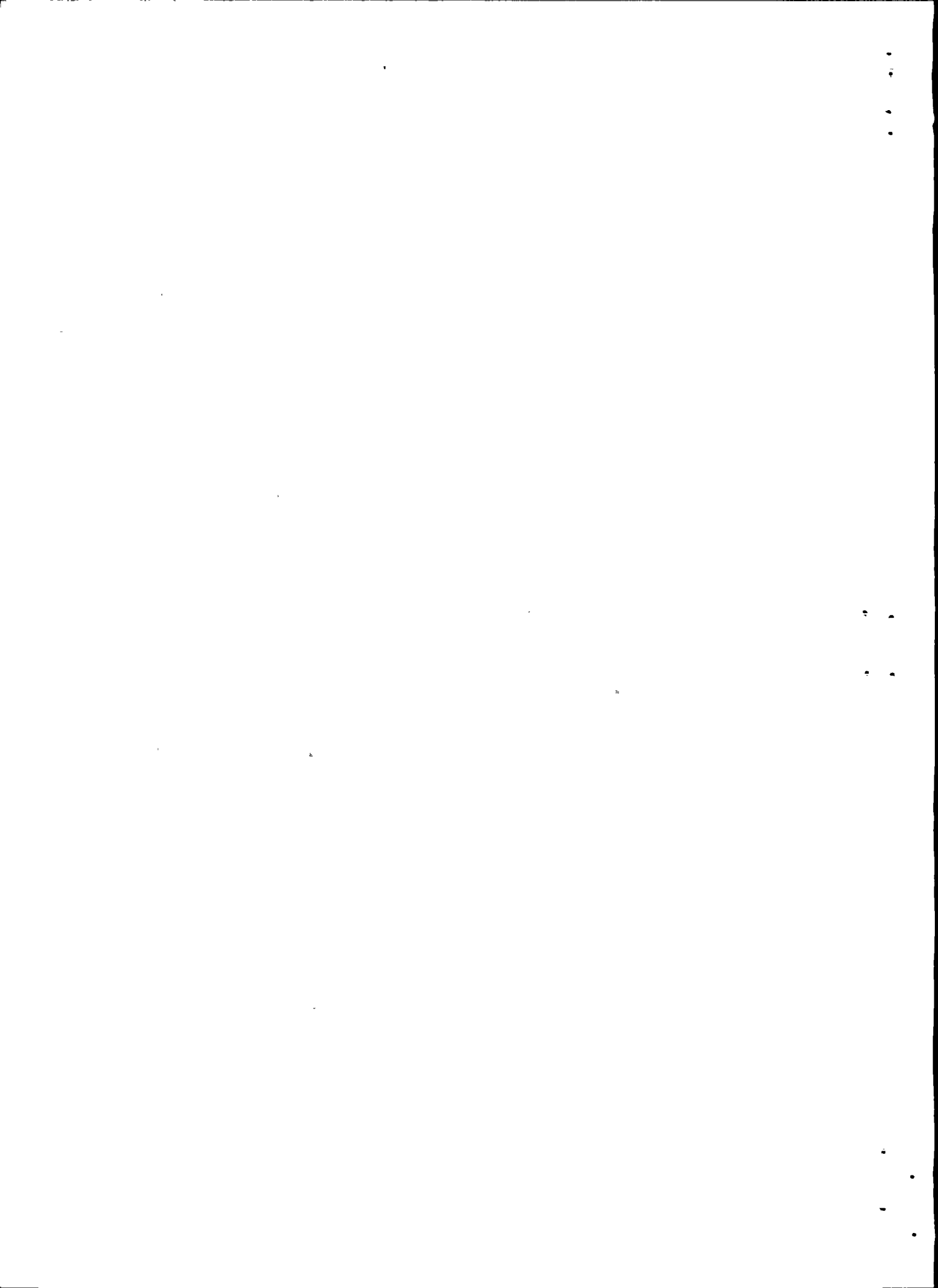


Figure 4a Aerial photo of City of Kodiak, center of one of the largest fishing fleets in Alaska. Solid line shows highest level reached by waves. (Photo by Alf Madsen)  
(From Tudor, 1964)



U.S. Army

Figure 4b Southwest breakwater at Kodiak shortly after the earthquake, with some of the battered remains of the King Crab cannery. (From Wilson and Torum, 1971)



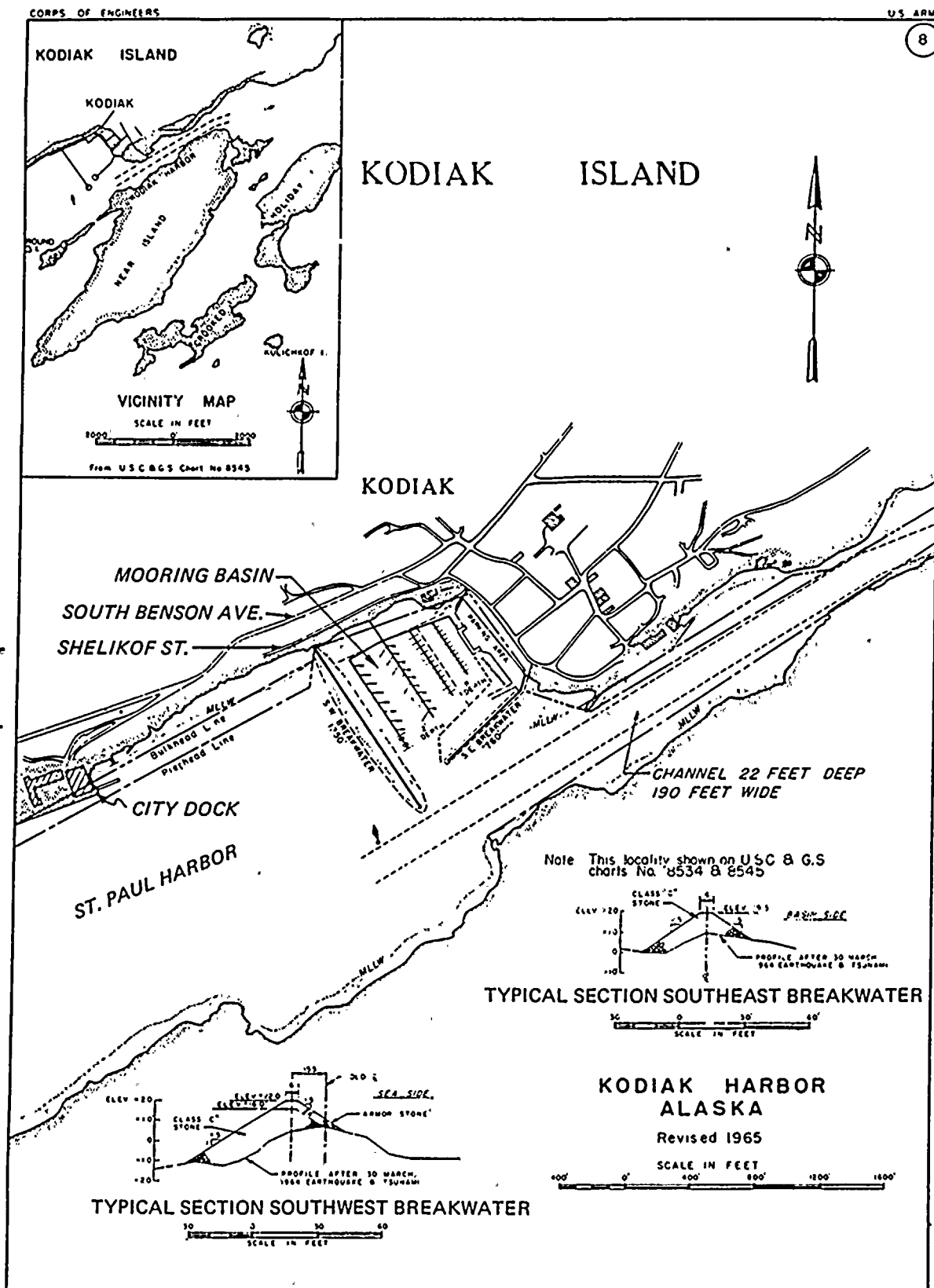
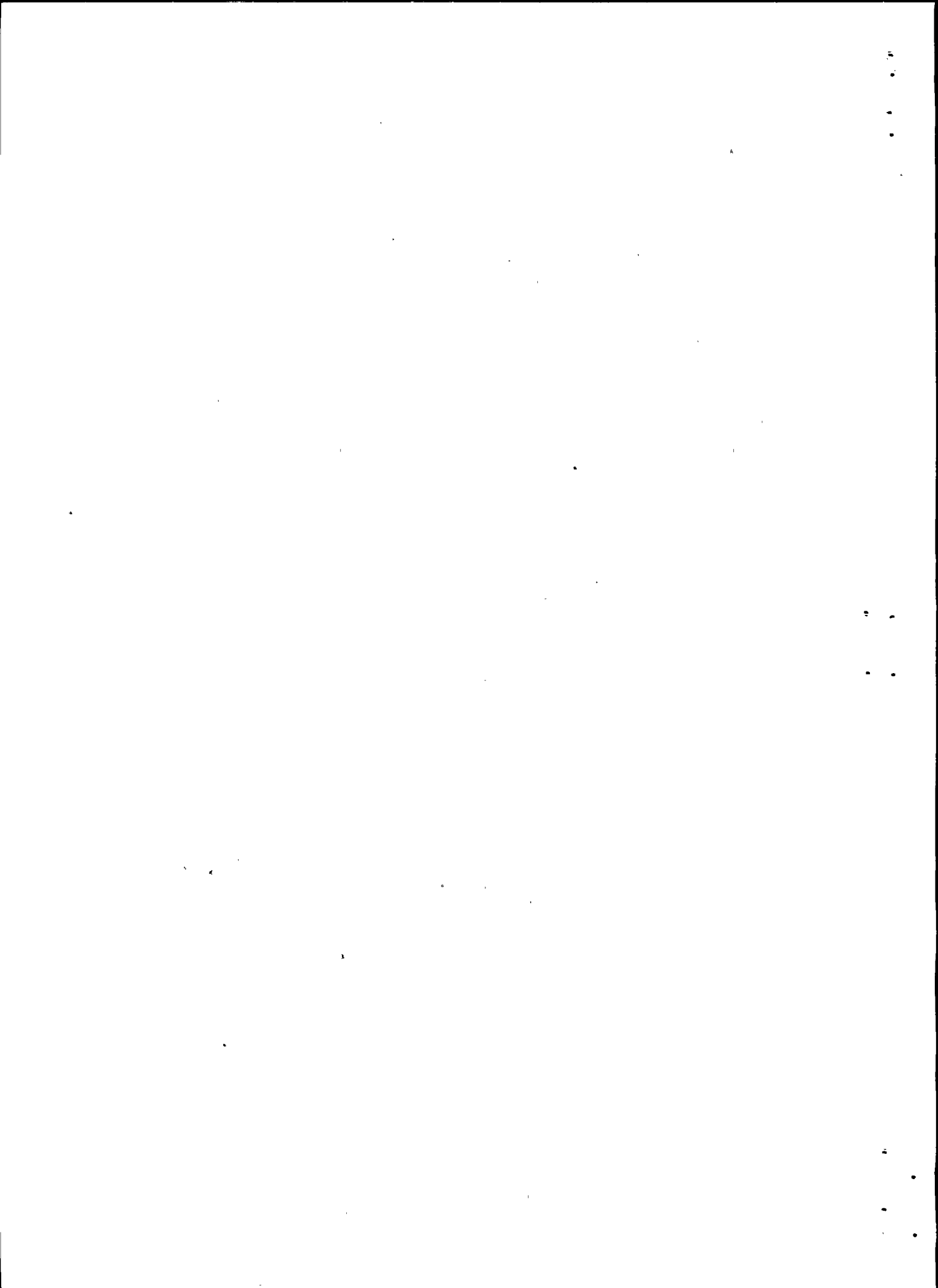


Figure 4c Map of Kodiak City and harbor with details of the breakwaters.  
(From Wilson and Torum, 1972)





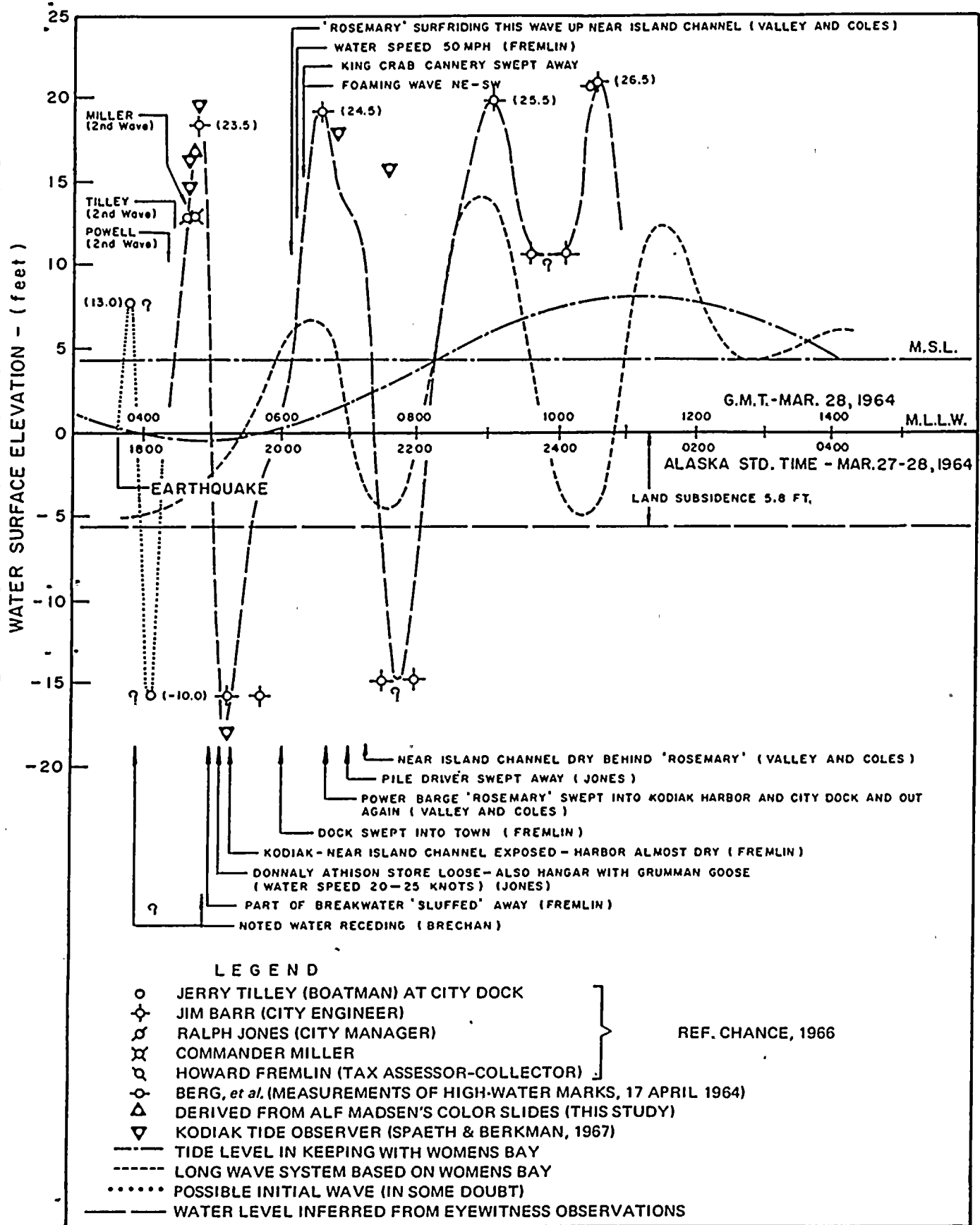
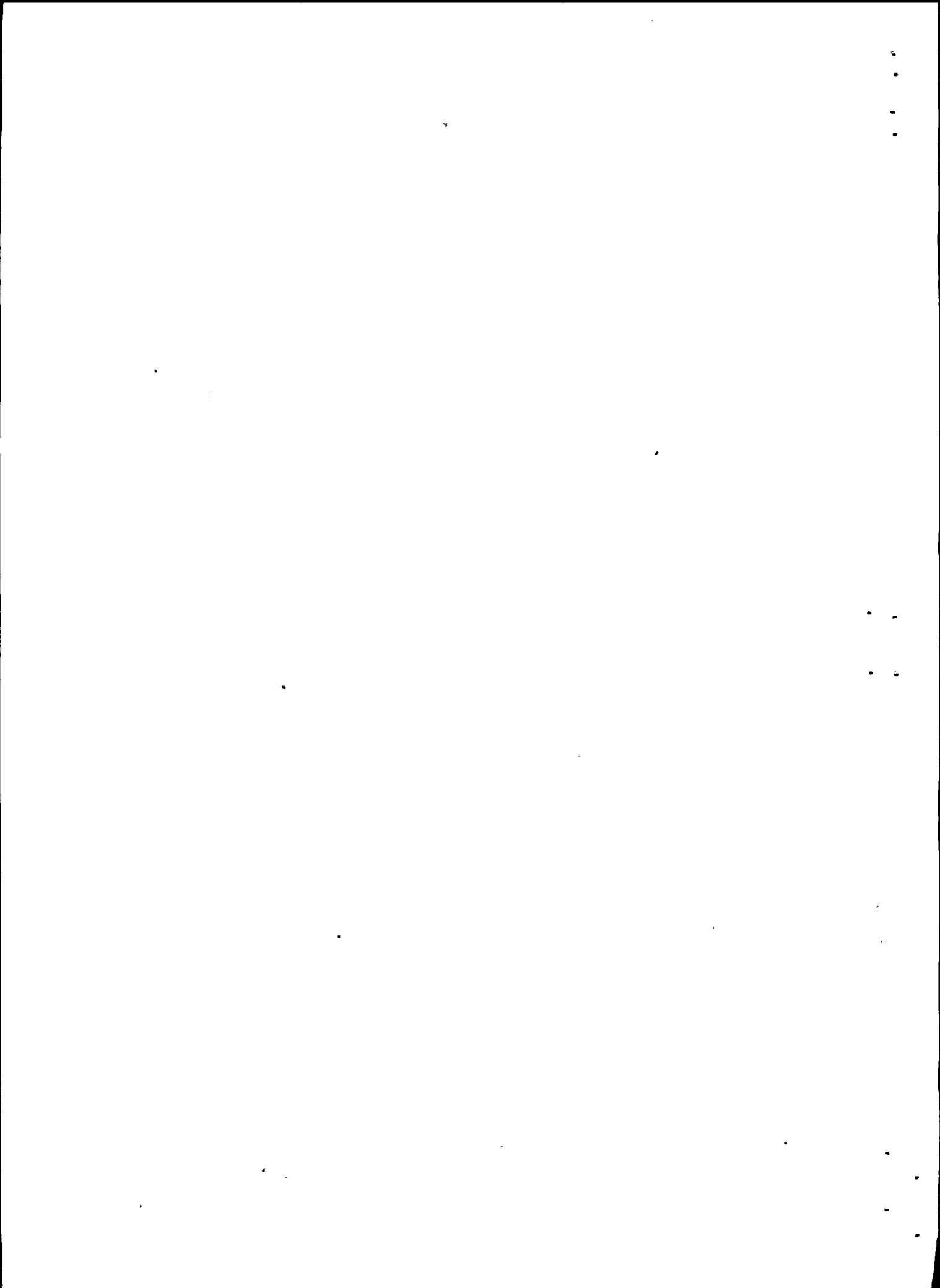


Figure 4d Hypothetical marigram for Kodiak City; based on observations by eyewitnesses.  
(From Wilson and Torum, 1972)



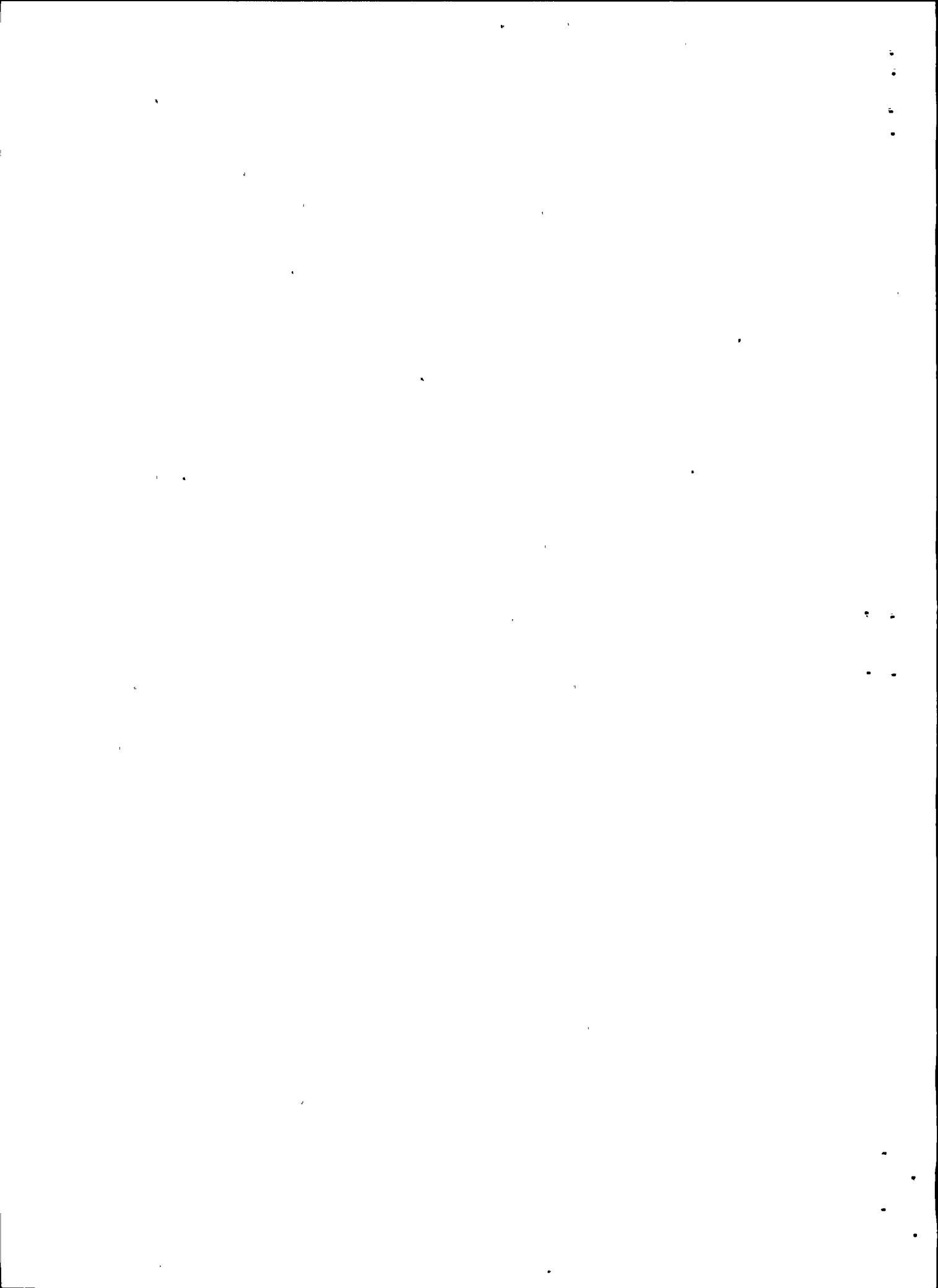
It is extremely unlikely that damage would result to the breakwater at Diablo Canyon from one of the great tsunamis of the type discussed above, owing to the low wave heights which are expected at the site for these waves (see below).

A thorough study was made of the tsunami wave elevations along the west coast of the continental United States by the U. S. Army Waterways Experiment Station (Houston and Garcia, 1978) for the U. S. Federal Insurance Administration. On page 7 of this report, they state:

"Plates 1-30 present predicted 100- and 500-year elevations (in feet) produced by distantly generated tsunamis on the west coast of the continental United States. These elevations include the effects of the astronomical tide; that is, they are maximum elevations due to the superposition of tsunami and tidal wave forms (see PART IV). The lower curves in Plates 1-30 represent the 100-year runup and the upper curves, the 500-year runup. A 100-year runup is one that is equaled or exceeded with an average frequency of once every 100 years; a 500-year runup has a corresponding definition. Runup values in this report are referenced to the mean sea level (msl) datum."

Page 4 of their report is reproduced here as Figure 5. This is a plot of the 100- and 500-year elevations for the section of the coast of California between 35°00' and 35°30' north latitude. Note that at Diablo Canyon (35°12.5' north latitude the 100-year elevation is about 5 feet, and the 500-year elevation is about 8 feet. Thus, the tsunami waves would not overtop the breakwater unless it was in a severely degraded condition at the time the tsunami waves struck it.

Analytical studies were made by Tetra Tech, Inc., for the Pacific Gas and Electric Company of the possible ground displacement at Diablo Canyon and the tsunami waves that might be generated by several hypothesized ground displacements at the Santa Maria Basin Fault and at the Santa Lucia Fault (Hwang, Yuen and Brandsma, 1975). A numerical model was used to calculate the water motion at the entrance



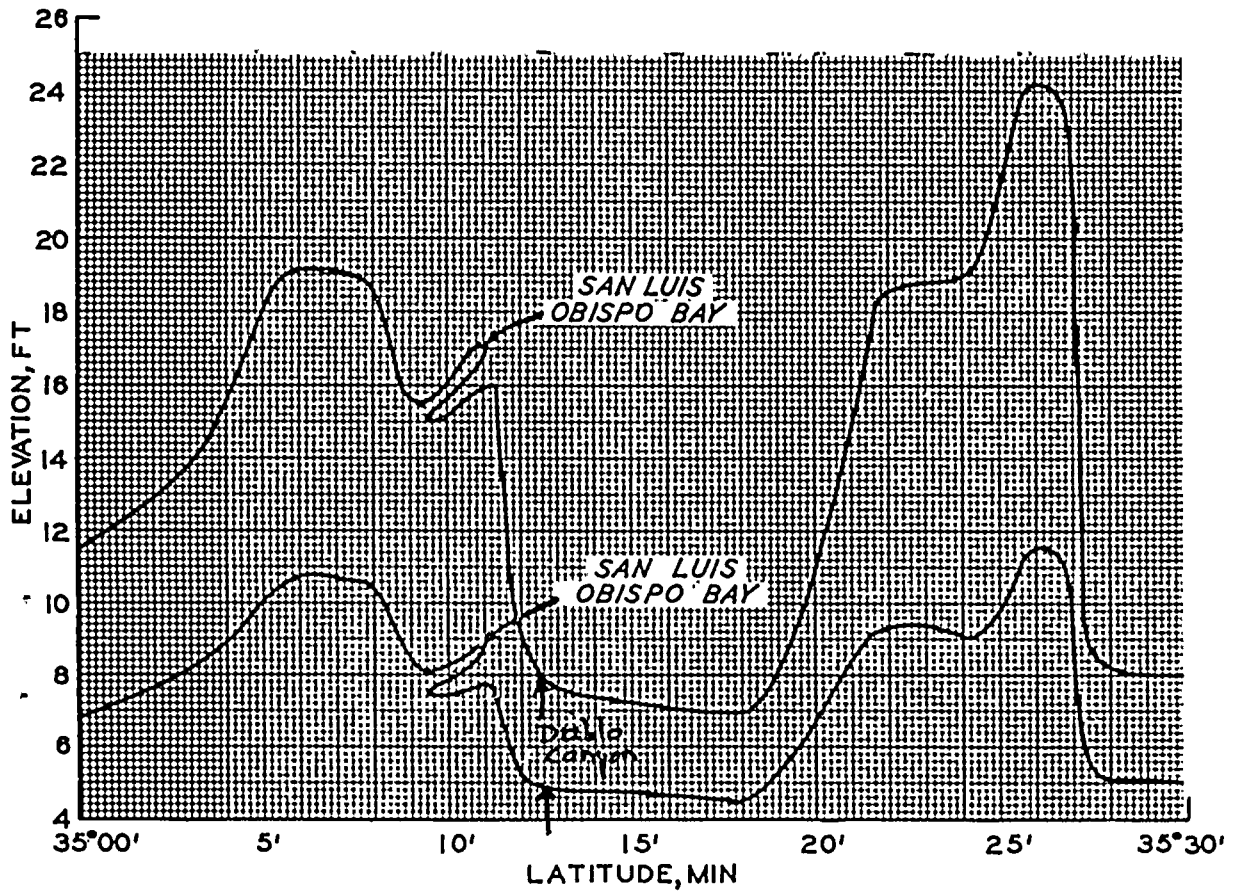


Figure 5. 100- and 500-year Elevations vs. Location  
Latitude 35°00' to 35°30'  
(From Houston and Garcia, 1978)

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to the intake basin. It is important to note that in their report the calculations presented for the tsunami waves are as seen by an observer at Diablo Canyon, including the hypothesized ground subsidence at the site. Their calculations show that with the breakwater in a "slumped" condition, with the slumping caused by the earthquake, the maximum tsunami crest elevation would be 8.7 feet above the tide level at the time of the wave, relative to the new lowered datum. This, combined with an astronomical tide of 5.3 feet above MLLW and a hypothesized storm tide of 1.0 feet, would result in a hypothesized tsunami wave crest of 15.0 feet above MLLW.

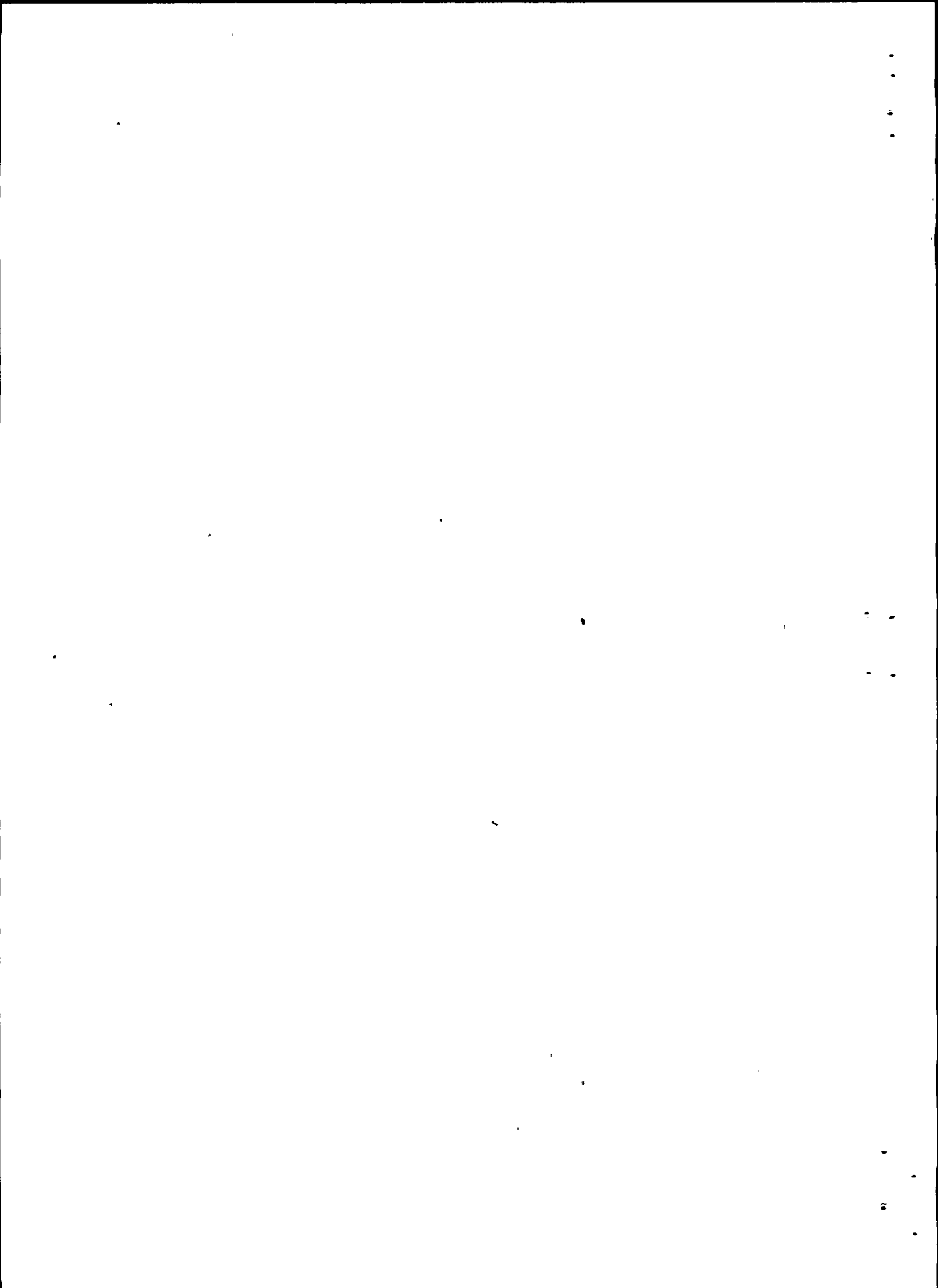
The breakwater crest is 20.0 feet above MLLW which, according to the calculations of H. Bolton Seed (1981), might degrade to a minimum elevation of 17 feet above present MLLW during severe earthquake excitation. Thus, if the breakwaters at Diablo Canyon are not in a storm damaged condition, the tsunami waves would not overtop them. This is also true for the case of the 500-year tsunami run-up by distantly generated tsunamis, based upon the predictions of Houston and Garcia (1978). The results of the literature survey of tsunamis indicate there is not likely to be much damage to a breakwater caused by tsunami waves that do not overtop the breakwater.

#### SEVERE STORM WAVE DAMAGE TO BREAKWATERS

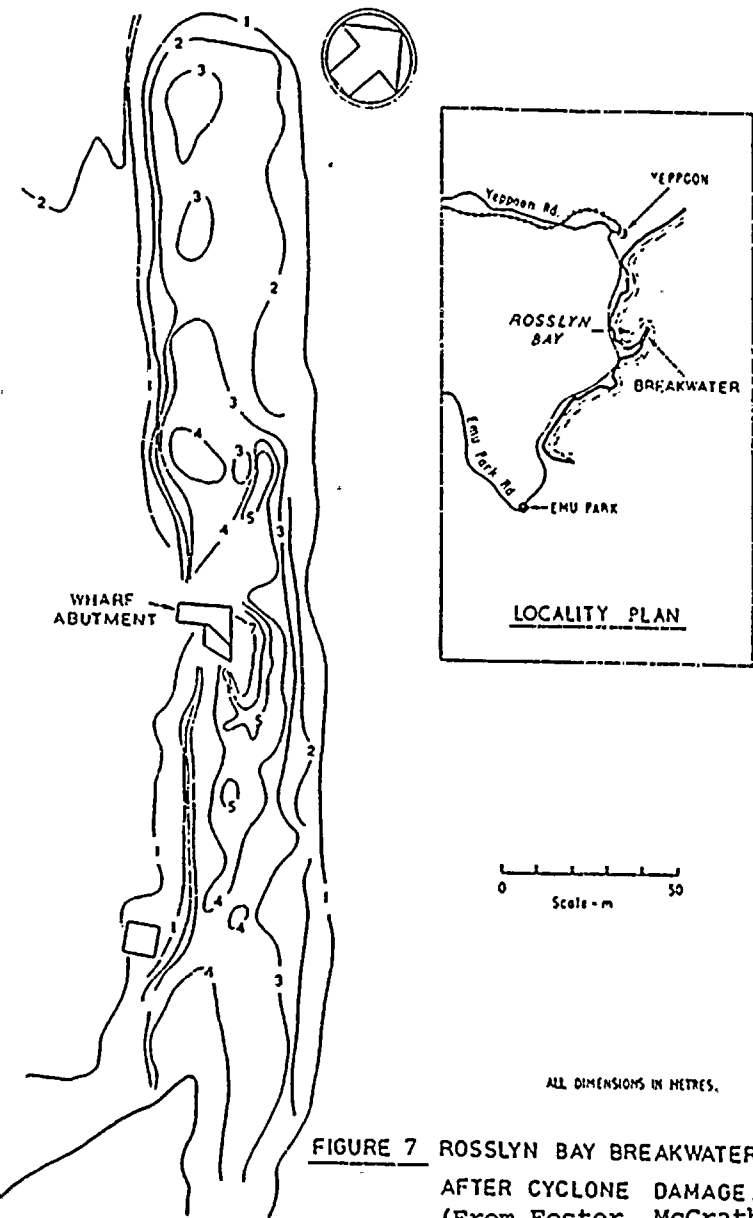
Although a large number of breakwaters throughout the world have been damaged by storm waves, few detailed studies have been made of the damage and the factors causing the damage. This is partly due to the fact that most damage is relatively minor, and is often repaired in a reasonably short time after the damage has occurred. There are some studies in the technical literature, however. These are described below.

##### Rossllyn Bay Breakwater, Queensland, Australia

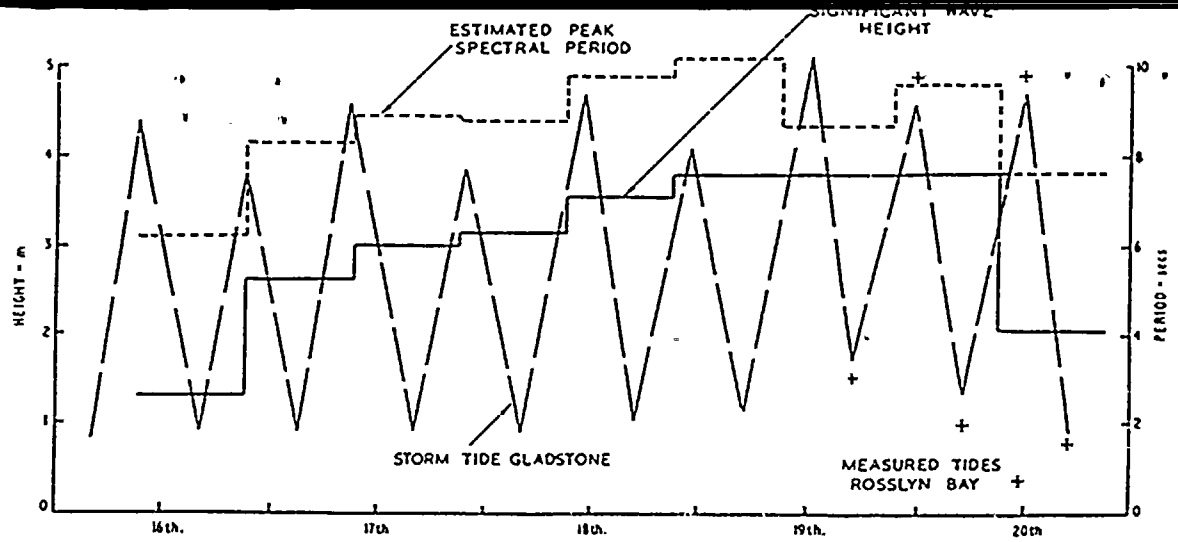
A relatively small breakwater (the Rossllyn Bay Breakwater) located near Yeppoon on the central Queensland coast of Australia, was damaged by severe overtopping by storm waves generated by tropical cyclone (hurricane) "David" during January 1976. Some details of the amount and type of degradation to





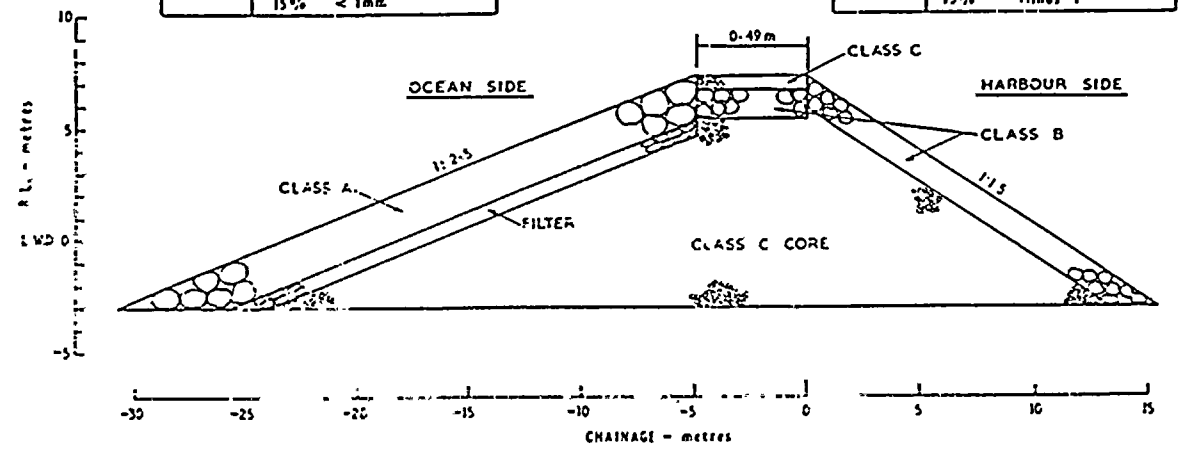


**FIGURE 7** ROSSLYN BAY BREAKWATER AFTER CYCLONE DAMAGE. (From Foster, McGrath and Bremner, 1978)

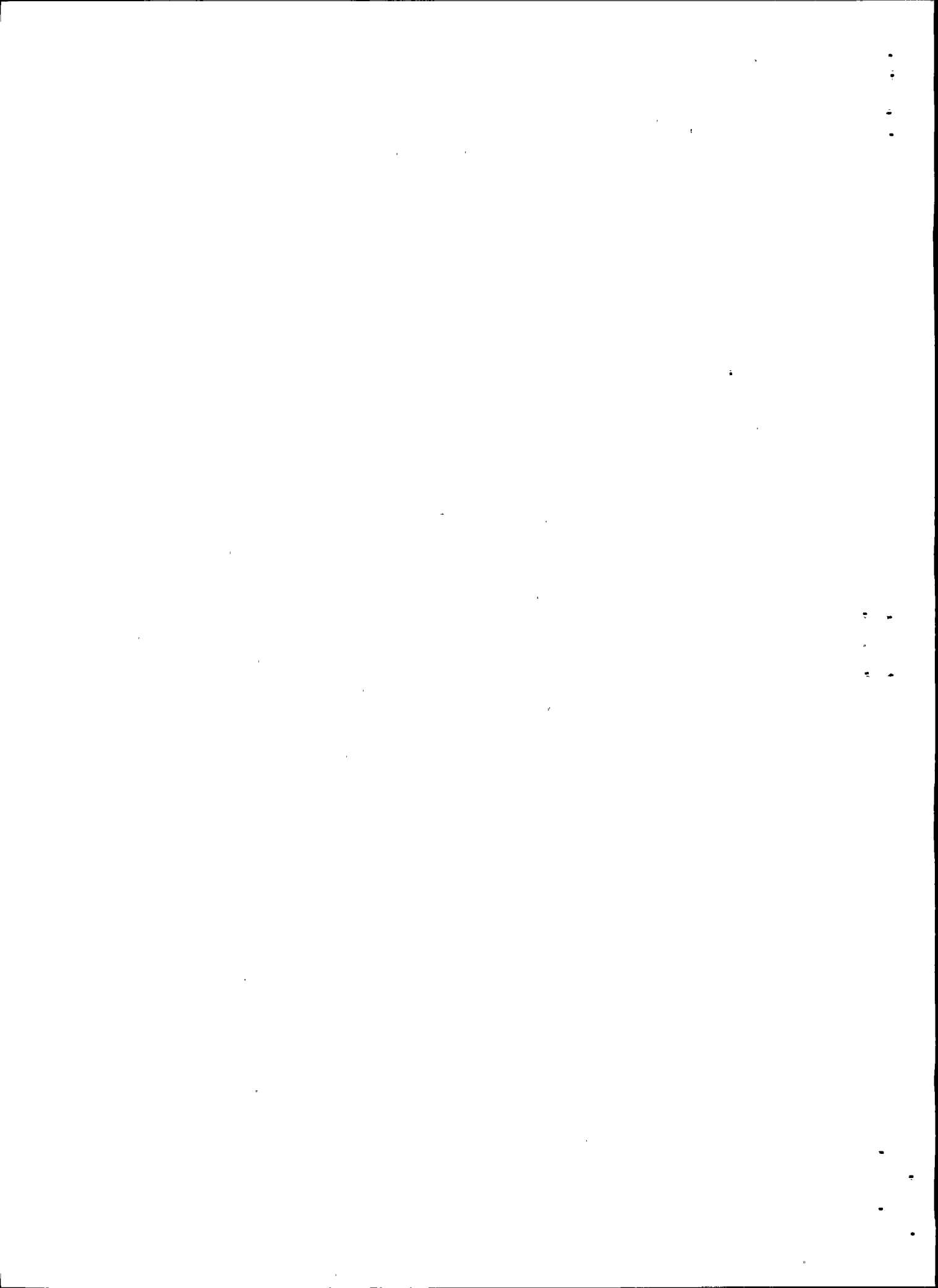


**FIGURE 6** WAVE AND STORM TIDE CONDITIONS (From Foster, McGrath and Bremner, 1978)

MODEL MATERIAL USED			PROTOTYPE MATERIAL		
CLASS A	75%	120 to 140 grams	75%	3 ton	
	20%	60 to 120 grams	20%	1 1/2 to 3 ton	
	5%	20 to 60 grams	5%	1/2 to 1 1/2 ton	
CLASS B	35%	20 to 40 grams	35%	1/2 to 1 ton	
	35%	10 to 20 grams	35%	500 lbs. to 1000 lbs	
	25%	2 to 10 grams	25%	100 lbs to 500 lbs	
FILTER	5%	0.5 to 2 grams	5%	15 lbs to 100 lbs	
	30%	20 to 60 grams	30%	1/2 to 1 ton	
	55%	10 to 40 grams	55%	100 lbs to 1000 lbs	
CLASS C	15%	6 to 3 mm	15%	Coarse Class C	
	60%	6 to 3 mm	60%	6" to 3"	
	25%	3 to 1 mm	25%	3" to 1"	
	15%	< 1mm	15%	Minus 1"	



**FIGURE 8** TYPICAL CROSS SECTION OF BREAKWATER (From Foster, McGrath and Bremner, 1978)



the breakwater that occurred has been reported by Foster, McGrath and Bremner (1978), with additional data being given by Bremner, Foster, Miller and Wallace (1980). Foster et al. (1978, p. 2095) state:

"Inspection of Figure [6] shows that the breakwater was subjected to wave and surge conditions approaching or exceeding the design conditions from 17th to 20th January. Overtopping of the breakwater occurred at high tide with very heavy overtopping being observed during 19th and 20th. The observed history of damage is given in Table [1].

Table [1]: History of Damage

Day Jan. 1976	Time (hours)	Damage
16th	2230	No observable damage
17th	0900	No observable damage
17th	2100	No observable damage
18th	0950	Slight damage
18th	1630	Slight damage
19th	1130	Heavy overtopping - slight damage
19th	2400	Heavy overtopping - minor damage
20th	Early hours	Major failure

"Major damage to the breakwater occurred at or soon after the evening high tide on 19th. At this time the breakwater failed catastrophically with the crest being destroyed and lowered within a few hours by some 4m over most of its length. The majority of the rock was displaced landwards coming to rest immediately on the harbour side. There was little damage to the seaward face.

"A survey of damage to the breakwater was undertaken 20 days following the failure as shown in Figure [7].

"Some comments on the damage are worthy of note. The damage resulted from wave overtopping, the majority of the rock being displaced landwards. When damage occurred it was catastrophic, taking place over a few hours. Despite high combinations of wave and storm tide levels and heavy overtopping little damage was noted prior to final collapse. It is difficult to believe that the class C or B material at the crest (Figure [8]) would withstand any significant overtopping as was observed to be the case (see section on model simulation of failure). It is possible that as a result of compaction under road traffic the crest acted as an impervious scour blanket giving protection until incipient failure occurred, after which total failure followed quickly. Damage was very uniform over the entire length of

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the breakwater except at one section where a wharf abutment on the harbour side acted as a buttress. At this location damage was slight. Little damage occurred to the seaward face for deep water wave heights of up to 3.8 m and estimated wave heights at the structure of between 3.0 and 3.4m. Equivalent damage coefficients in Hudson's equation are approximately 2.7 and 3.9 and wave conditions varied between breaking at low tide and non-breaking at high tide."

Additional information as given in Bremner et al. (1980, p. 1899) can be seen in Figures 9 and 10.

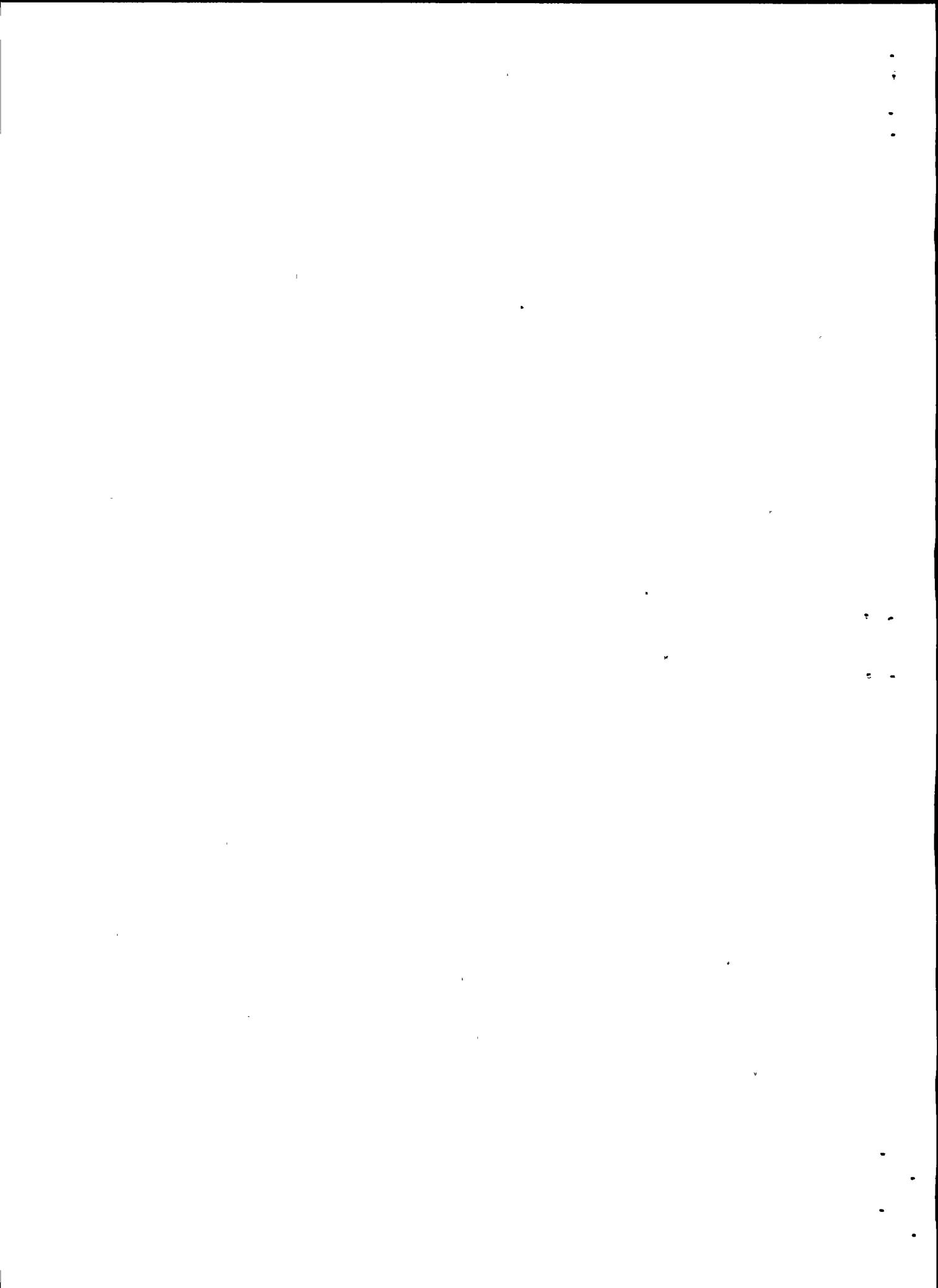
Of particular interest are their statements regarding the functional usefulness of the breakwater after the damage had occurred; Foster et al. (1978, p. 2097) state:

"After failure the structure continued to give substantial protection by acting as a partially submerged breakwater, significantly reducing damage to the harbour infrastructure during the storm and enabling the harbour to be used for its design function under the more common weather conditions that followed. The action of prudent yachtsmen in removing their vessels from the harbour to nearby natural shelter at the onset of the cyclone resulted in the damage to moored vessels being not too severe."

Bremner et al. (1980, p. 1900) state:

- "(i) Failure occurred in a controlled manner with material being displaced from the crest and deposited on the leeward slope, forming a widened and lowered profile. The breakwater was composed of 3t nominal armour rock on the seaward face, 1/2 to 1 tonne filter rock and less than 150mm core material.
- (ii) Failure to the breakwater was closely simulated by model studies.
- (iii) Reconstruction of the breakwater was not commenced until 2-1/2 years after failure. During this period the breakwater acted as a submerged breakwater and continued to provide substantial wave protection within the harbour. Over this period waves of up to 1.8m height were experienced. All small boats continued to stay on their piled and bottom moorings and there was no damage to boats, moorings or shoreline revetments.

"These observations triggered the idea that if a controlled submerged breakwater could successfully be built then a significant reduction in costs may well be possible. The interesting design



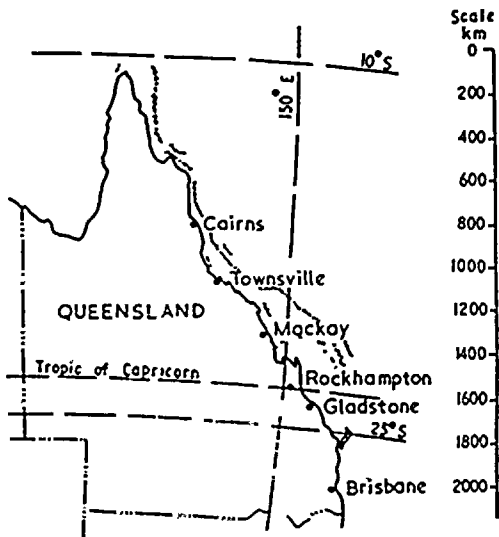
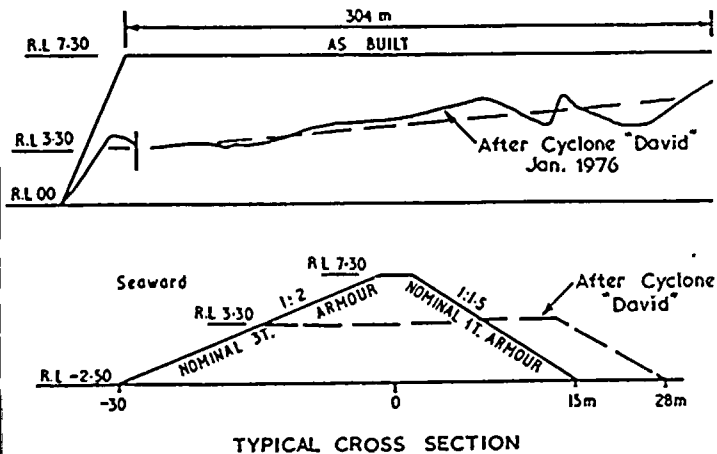


FIGURE 9 LOCALITY SKETCH

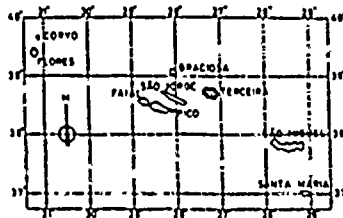
(From Bremner et al., 1980)



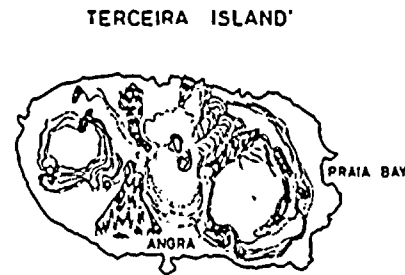
TYPICAL CROSS SECTION

FIGURE 10 DAMAGE TO ROSSLYN BAY BREAKWATER BY CYCLONE DAVID

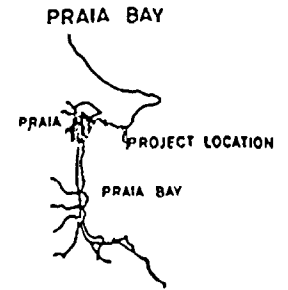
(From Bremner et al., 1980)



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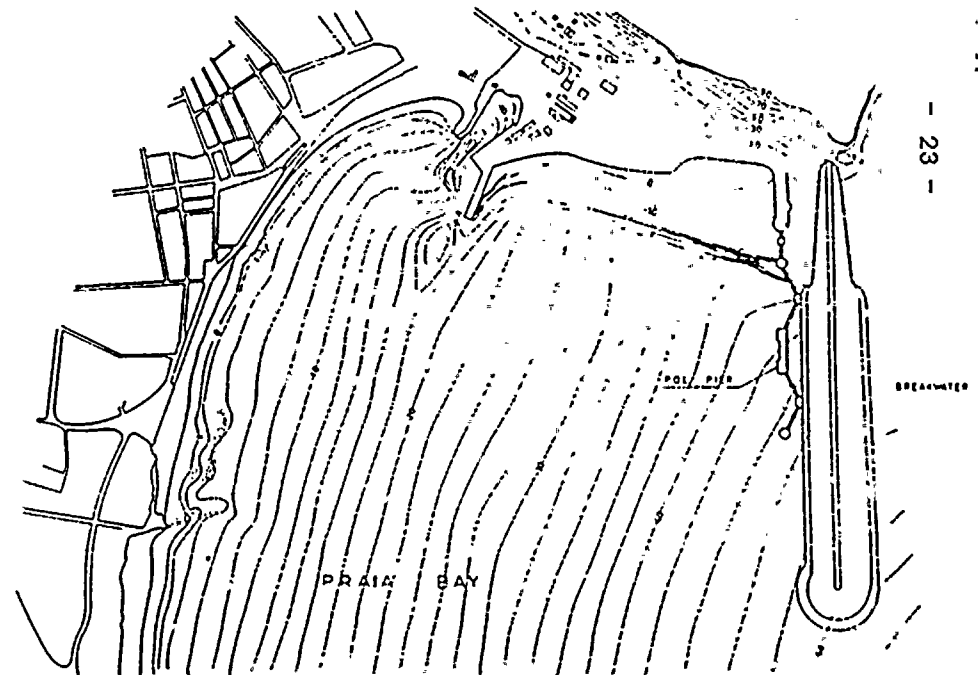


FIG. 11 PRAIA DA VITORIA HARBOR: GENERAL ARRANGEMENT

(From Reis de Carvalho, 1964).

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problem was then posed of building a breakwater of sufficient initial stability to allow safe construction but when subjected to the designed forces, due to carefully chosen extreme and fairly rare weather events that result from cyclones, it would be reshaped in a controlled and predictable manner to become a stable and partially submerged structure and retain its wave attenuation ability."

Praia da Vitoria Harbour, Azores

During 25-28 December 1962, a violent storm generated waves that attacked the breakwater at Praia da Vitoria, Azores. The breakwater

"... was continuously overtopped between the afternoon of 25th December and the morning of the 28th, notably during the high water."

The waves were estimated to be 8 to 9 meters high with a period of 13 to 14 seconds. The location of the breakwater, details of the breakwater, and a copy of an aerial photograph (after the storm) are shown in Figures 11-13.

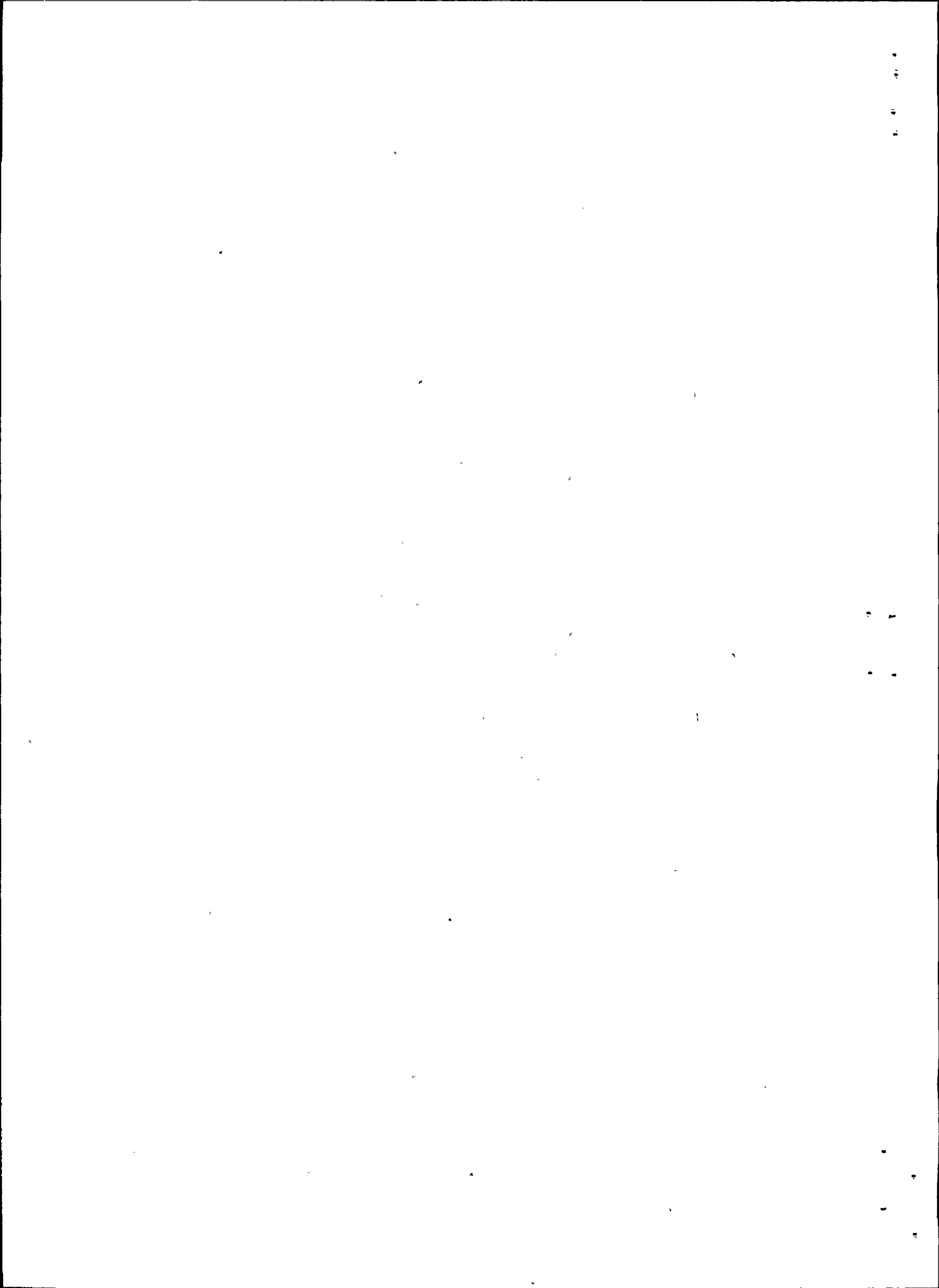
In the paper by José Joaquim Reis de Carvalho (1964, p. 569) the following extensive information was presented.

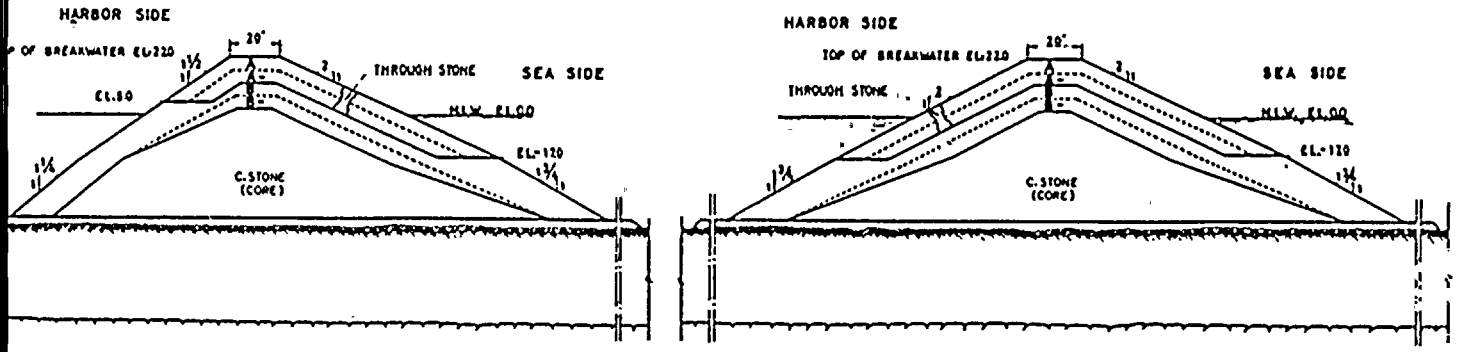
"At the date of the storm the breakwater was practically completed save for armour stone A which had still to be placed between elevations (-12!00) and (-22!0), between the root and the profile 175 metres.

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

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

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





TYPICAL SECTION

HEAD SECTION

BREAKWATER - STONE SIZES

A STONE (ARMOR)	12-18 tons and larger with 80% of stones larger than 14 tons
A- STONE (ARMOR)	10-12 tons with 80% of the stones larger than 11 tons
B STONE (SECONDARY ARMOR)	4-8 tons and larger with 80% of stones larger than 8 tons
B- STONE (SECONDARY ARMOR)	2-4 tons with 80% of the stones larger than 3 tons

Figure 12. Original and alternate Sections  
(From Reis de Carvalho, 1964)

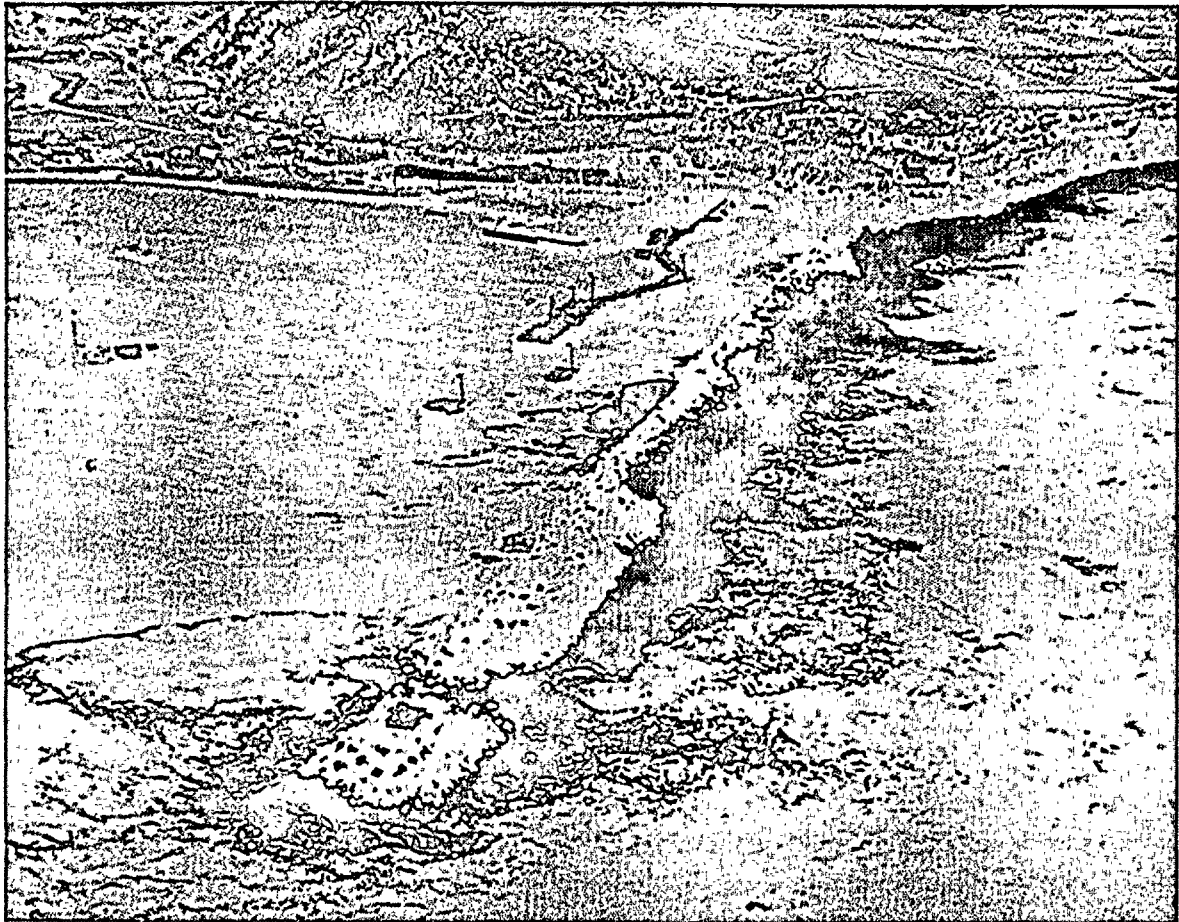
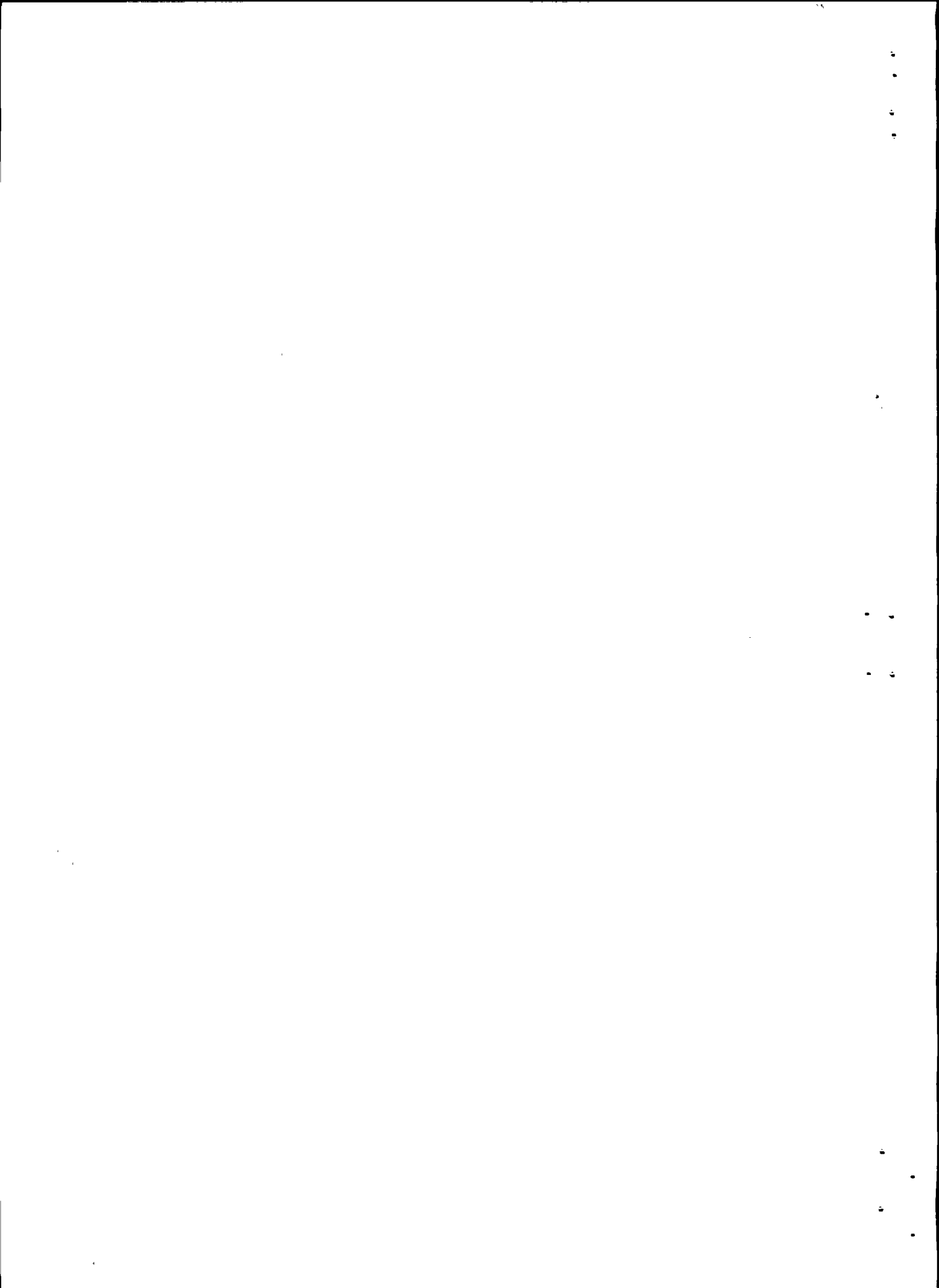


Figure 13. General View of Harbor  
(From Reis de Carvalho, 1964)



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

"The damages undergone by the breakwater during the storm can be summed up as follows [Figs. 13, 14, 15, 16 and 17].

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

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

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

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

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

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

"The first conclusion to be drawn from the preceding analysis of the damages observed along the breakwater, is their extremely irregular distribution: the head remained practically intact, the

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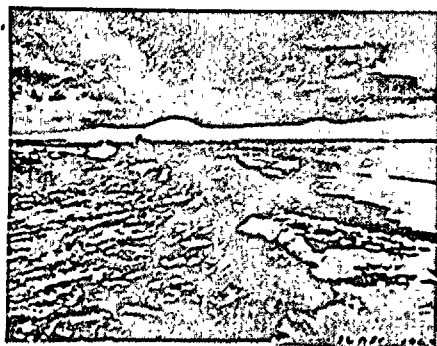


Figure 14. Views of the Damages  
(From Reis de Carvalho, 1964)

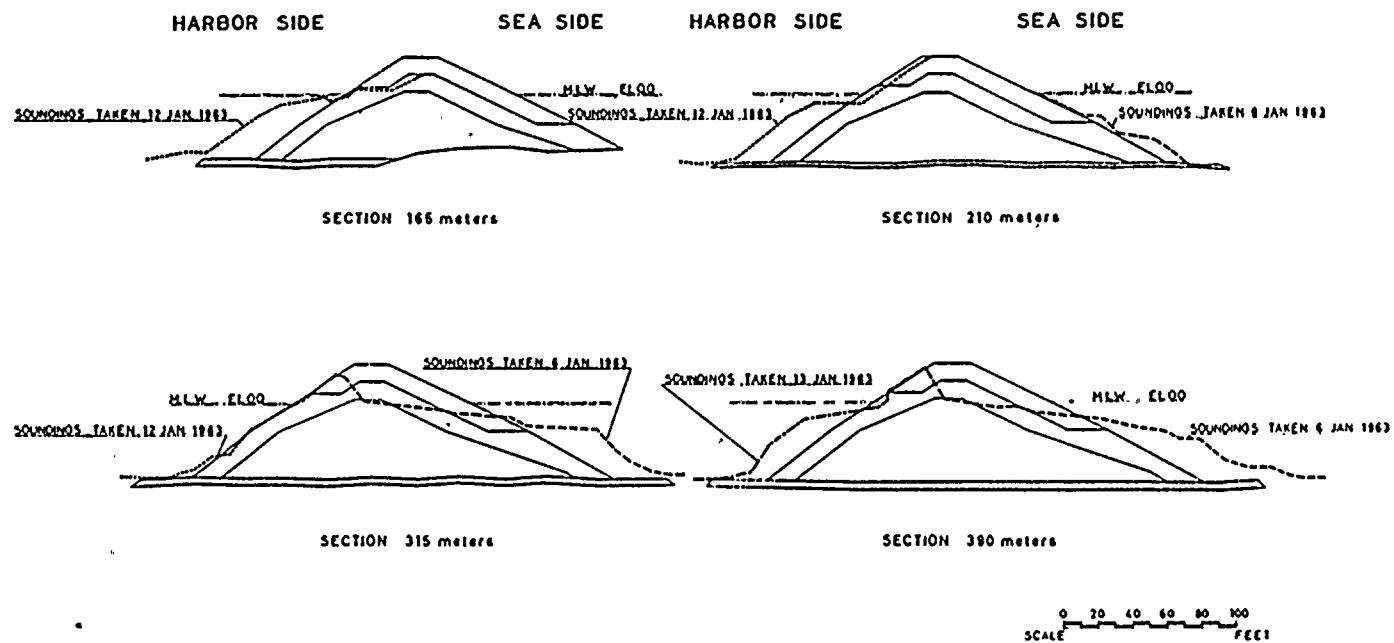
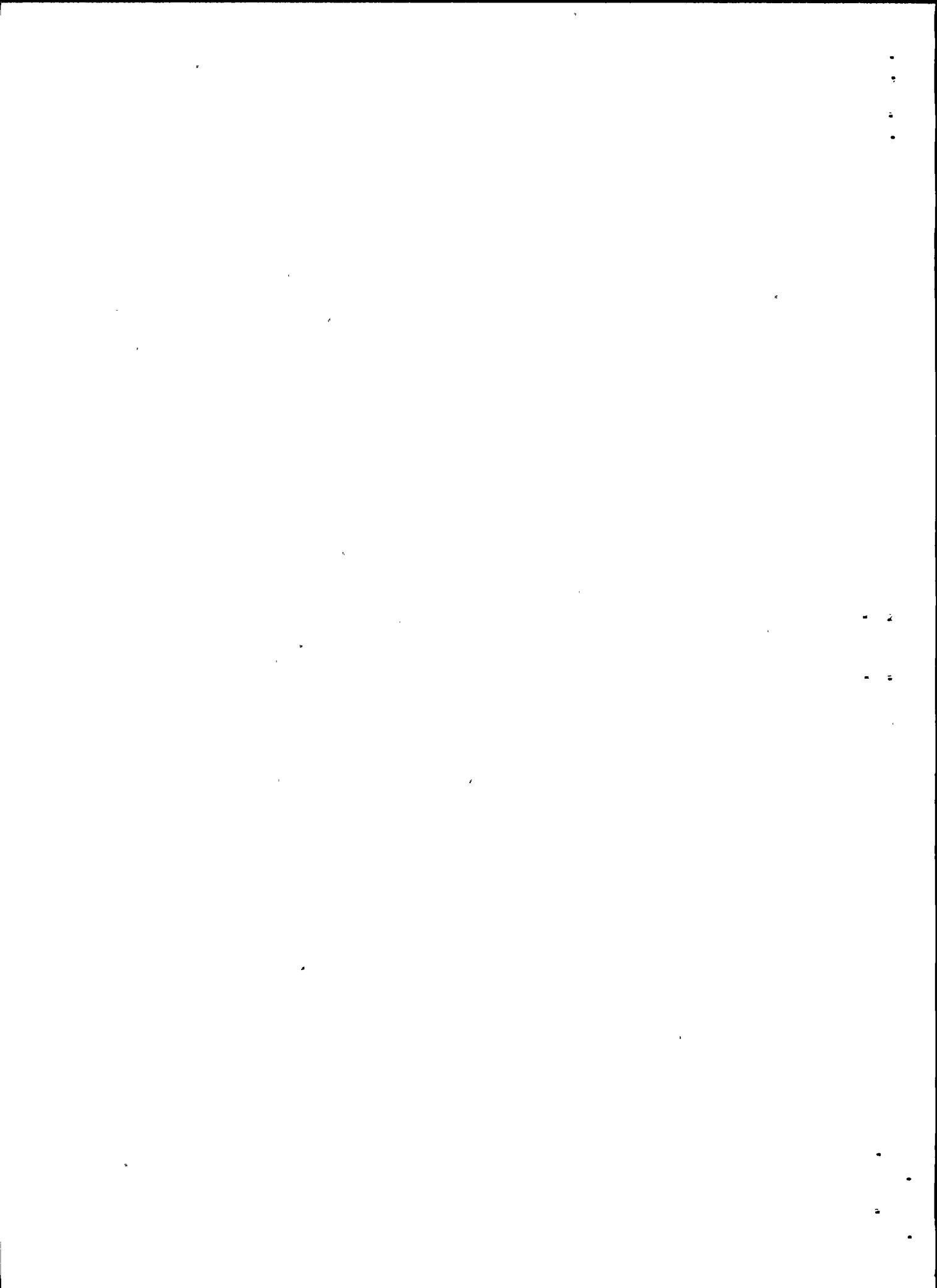


Figure 15. Breakwater sections after storm  
(From Reis de Carvalho, 1964)





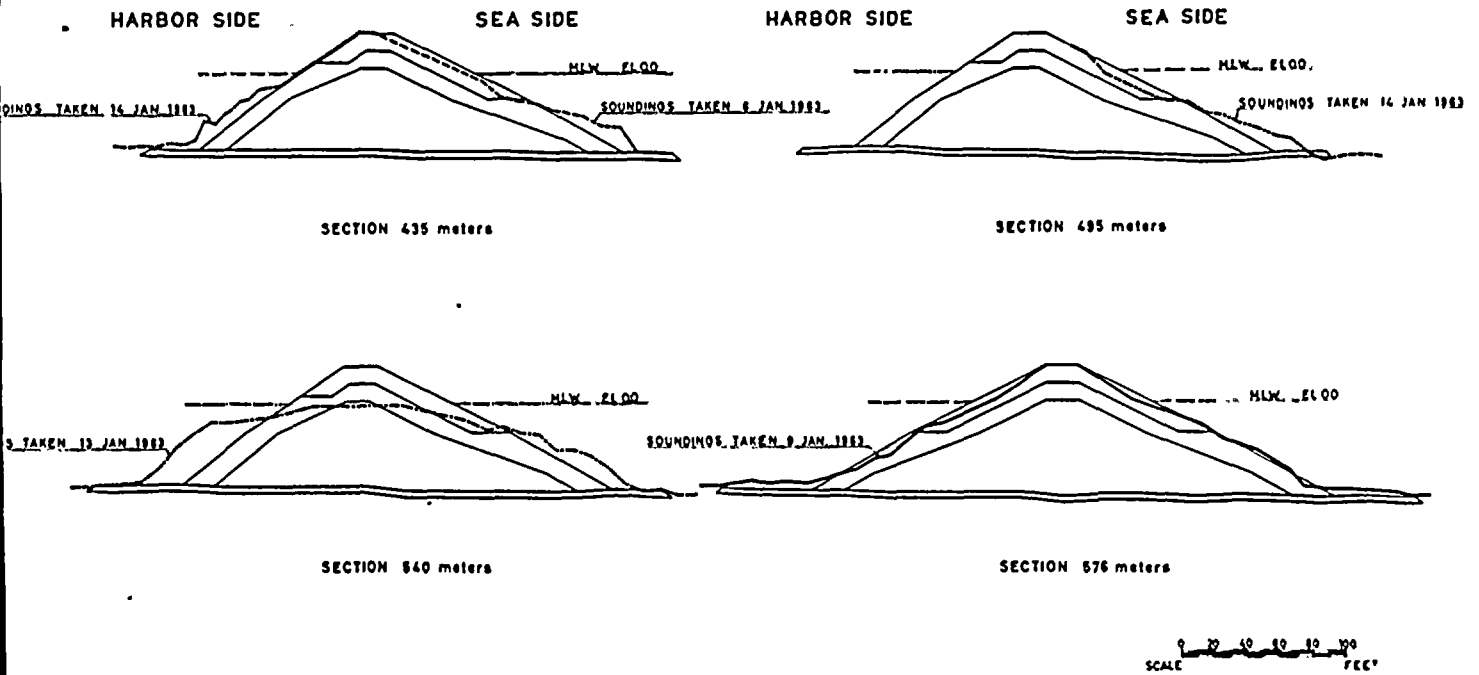


Figure 16 Breakwater sections after storm  
(From Reis de Carvalho, 1964)

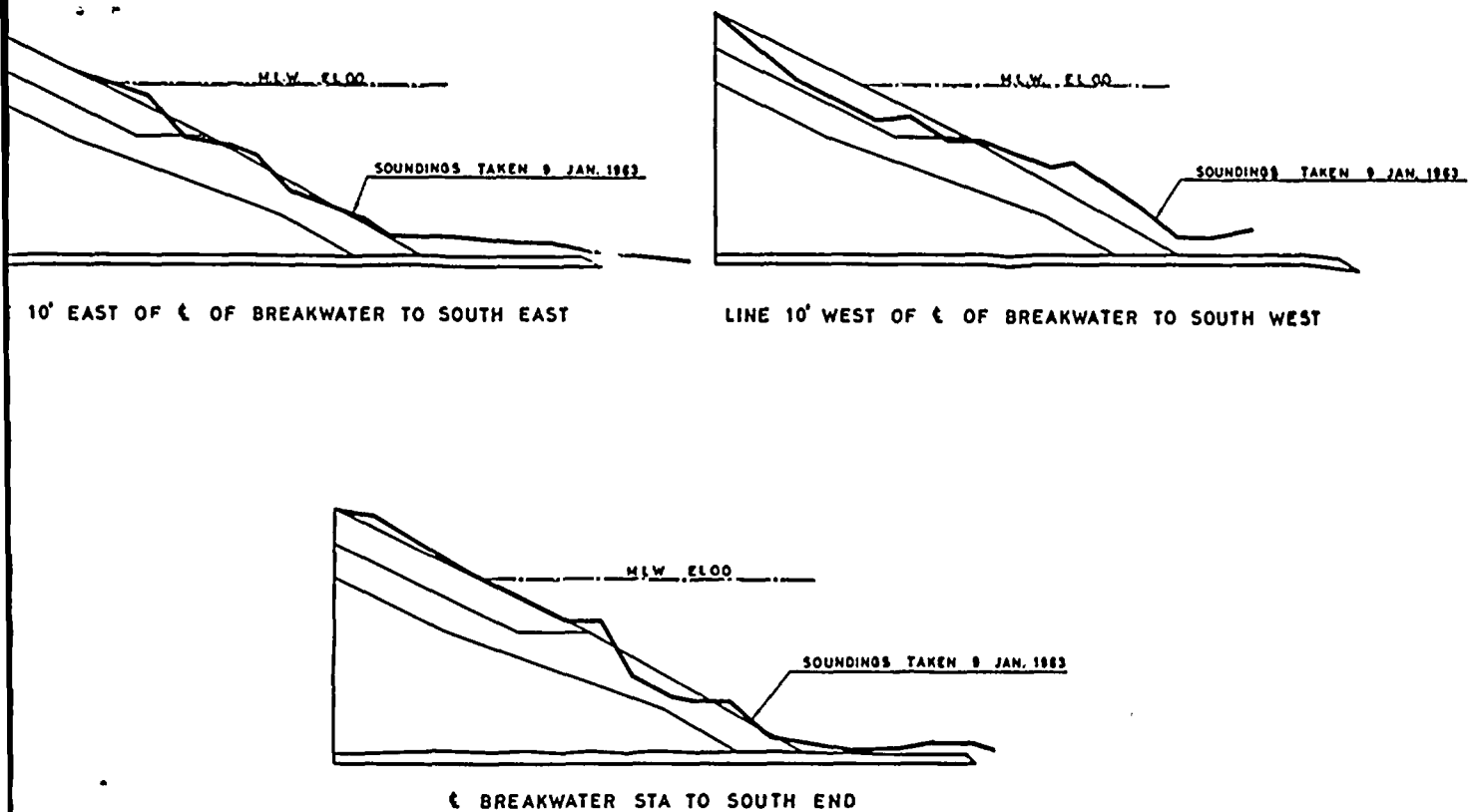
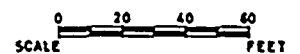


Figure 17 Breakwater sections after storm  
(From Reis de Carvalho, 1964)



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adjoining section presents a breach, then a length of about 90 metres underwent only slight damages, but just beyond the breakwater was severely hit in an extent of 150 metres.

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

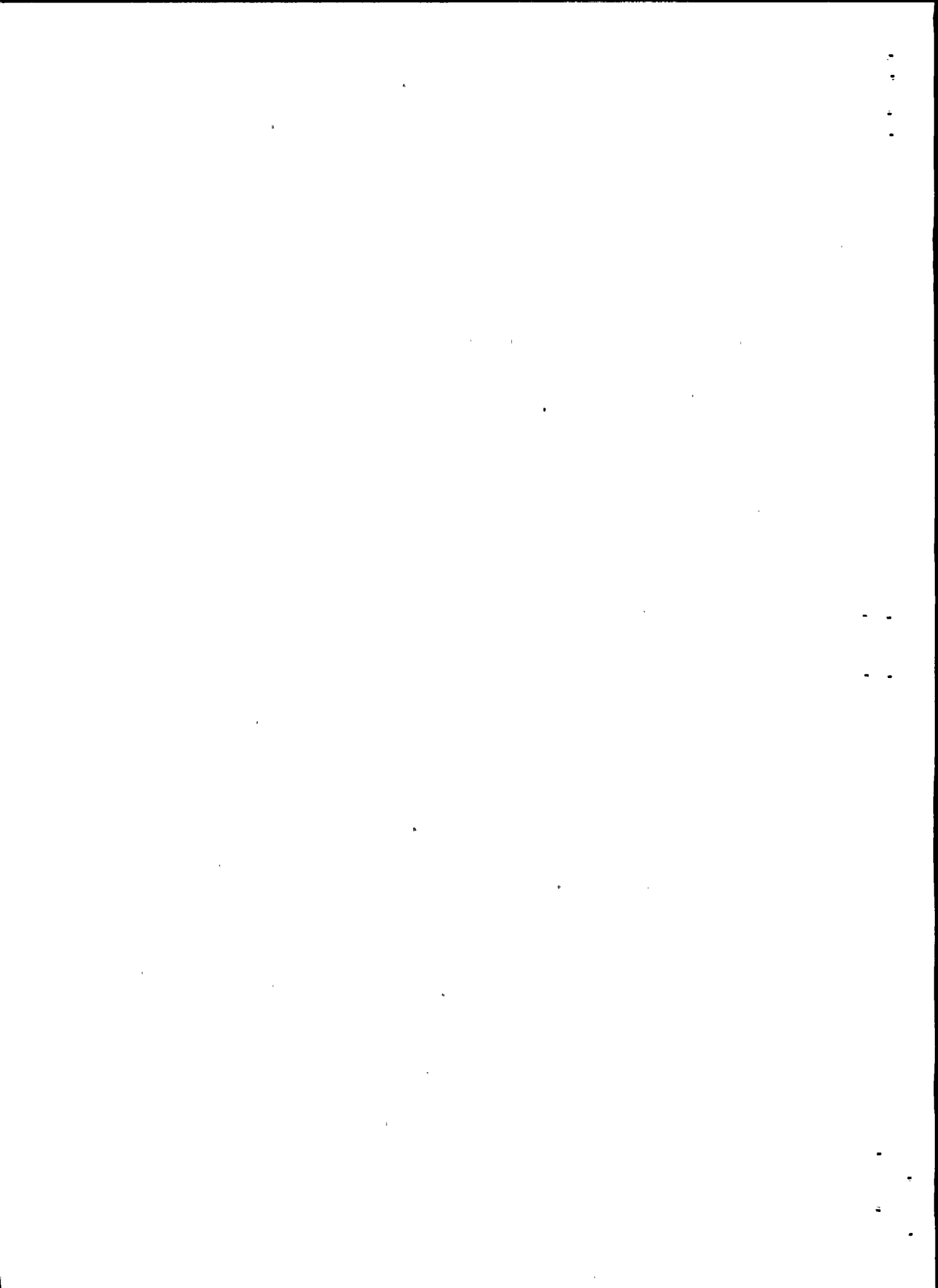
"In the harbour-side slope, overtopping waves displaced some B armour stones alone.

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

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

"For the latter direction, the angle of the sea attack with the structure was small, without any apparent variation of the wave height along the breakwater. The analysis of the (fig. [14]) photographs taken during the storm shows that the attack was always practically frontal.

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



Leixões, Portugal

An interesting fact about the breakwater at Leixões, Portugal, is that the outer breakwater was built between 1937 and 1941 as a submerged breakwater with its top 1 meter above datum (the datum being arbitrary, a few centimeters below the minimum low water level, astronomical spring tides). In 1971 the outer breakwater was raised to +15 meters, Figures 18 and 19. This breakwater was damaged during storms of 16-17 January 1973 and February 1974 (Morais and Abecasis, 1974). They state that during the storm of 16-17 January 1973 (p. 100) the following occurred.

"During the storm the outer breakwater was severely overtopped. As a consequence of this overtopping the structures of the oil terminal itself, mainly the steel ones, and the oil leading pipes were bent.

"The looking landwards steel doors of a transformer station located inside the concrete superstructure of the breakwater were carried away, apparently by suction action [Fig. 20]. Some tetrapods of the cover layer of the breakwater were broken. But the most severe damages occurred in the head of the breakwater that was practically destroyed: many tetrapods were broken or carried away; the end concrete block of the superstructure supporting the lighthouse and the neighbouring one fell down by undermining action of the waves, that carried away the small rock blocks of the underlayer over which they were placed, and the following concrete block was slightly displaced laterally [Figs. 21, 22, 23, 24]. To this effect strongly contributed the presence of the cylindrical concrete monolith that supported the lighthouse of the submerged breakwater before the raising operation, though the upper part of this cylindrical block was mechanically destroyed during the raising works, as this block originated a concentration of strong currents from the breaking waves on the damaged zone."

In February 1974 another severe storm occurred. By then, the head of the outer breakwater had been repaired, and during this storm only a few tetrapods near the head of the breakwater were broken or carried away.

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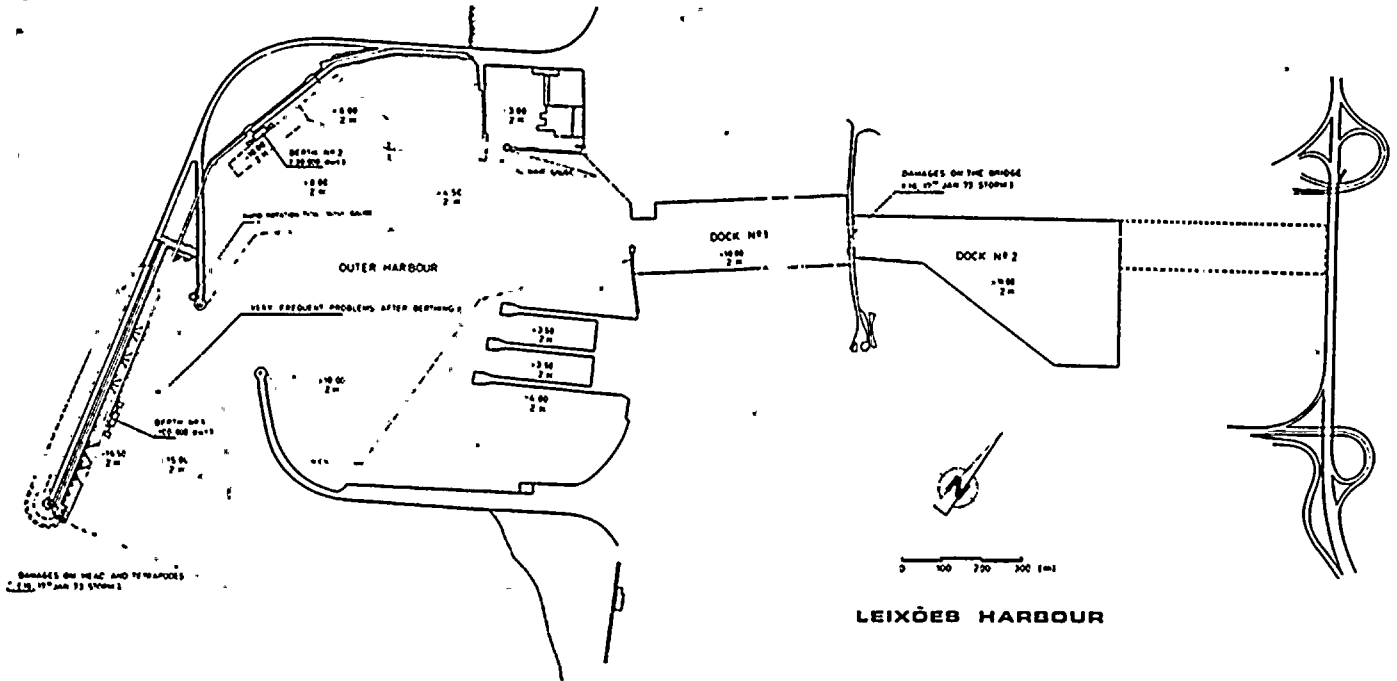


Fig. 18. Lay-out of Leixões Harbour  
(From Morais and Abecasis, 1974)

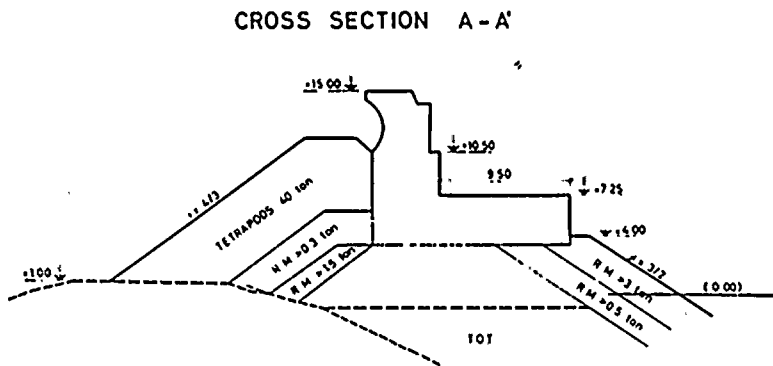


Fig. 19. Head and cross-section of North breakwater  
(From Morais and Abecasis, 1974)

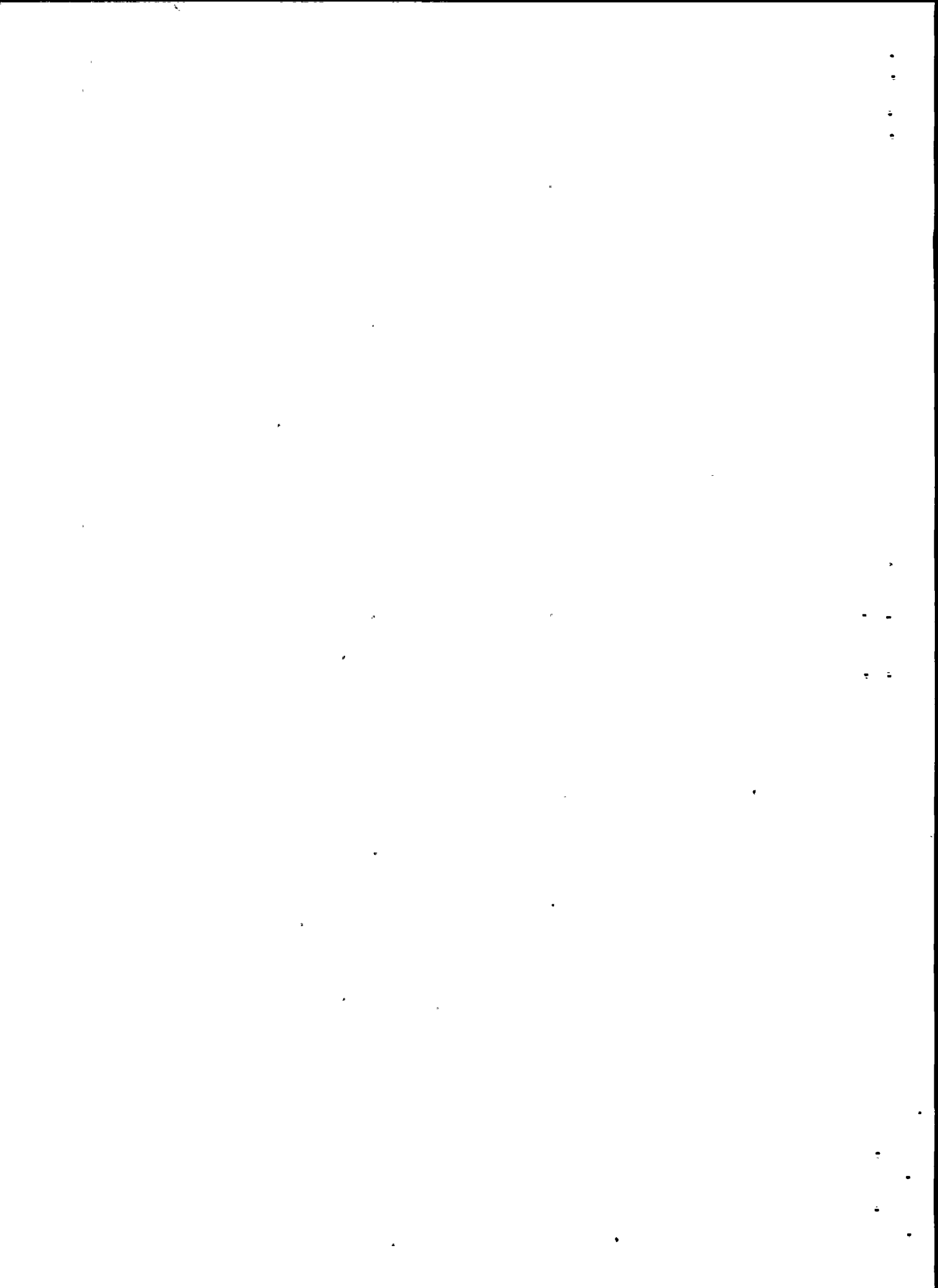




Figure 20



(All from Morais and Abecassis, 1974)

Figure 20. Damages caused by overtopping (Cont.)

Figure 21. Damages at the head

Figure 22. Damages at the head (Cont.)

Figure 23. Damages at the head (Cont.)

Figure 21

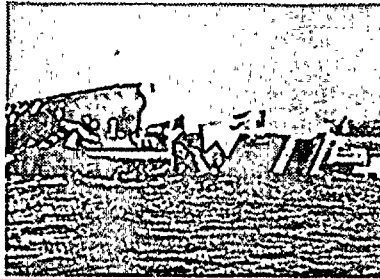


Figure 22



Figure 23

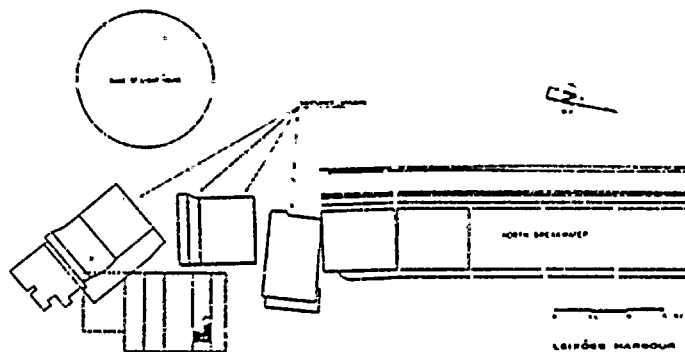
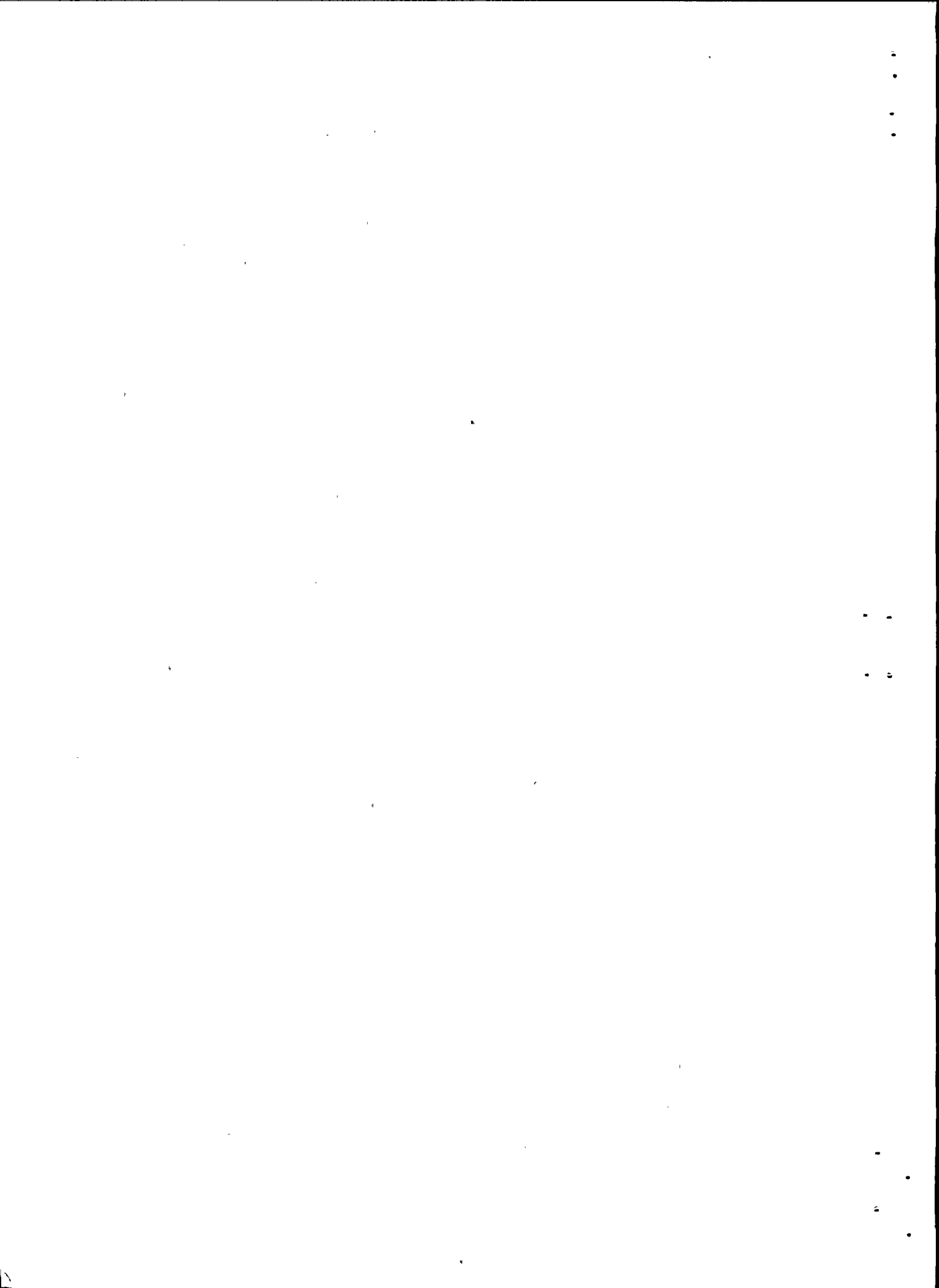


Figure 24. Damages at the head (Cont.)  
(From Morais and Abecassis, 1974)



Bilbao, Spain

Tørum, Mathiesen and Escutia (1979) present the following information on the breakwater at Bilbao, Spain (Figure 25).

"The construction of the breakwater was completed in 1976. The total length is approximately 2,400 m and the orientation is shown in Fig. [26].

"The theoretical cross section is shown in Fig. [27]. The front armour layer consists of 65 t concrete blocks. The porosity of this layer is 40-45%. The blocks were placed by means of a crane running on the wave screen.

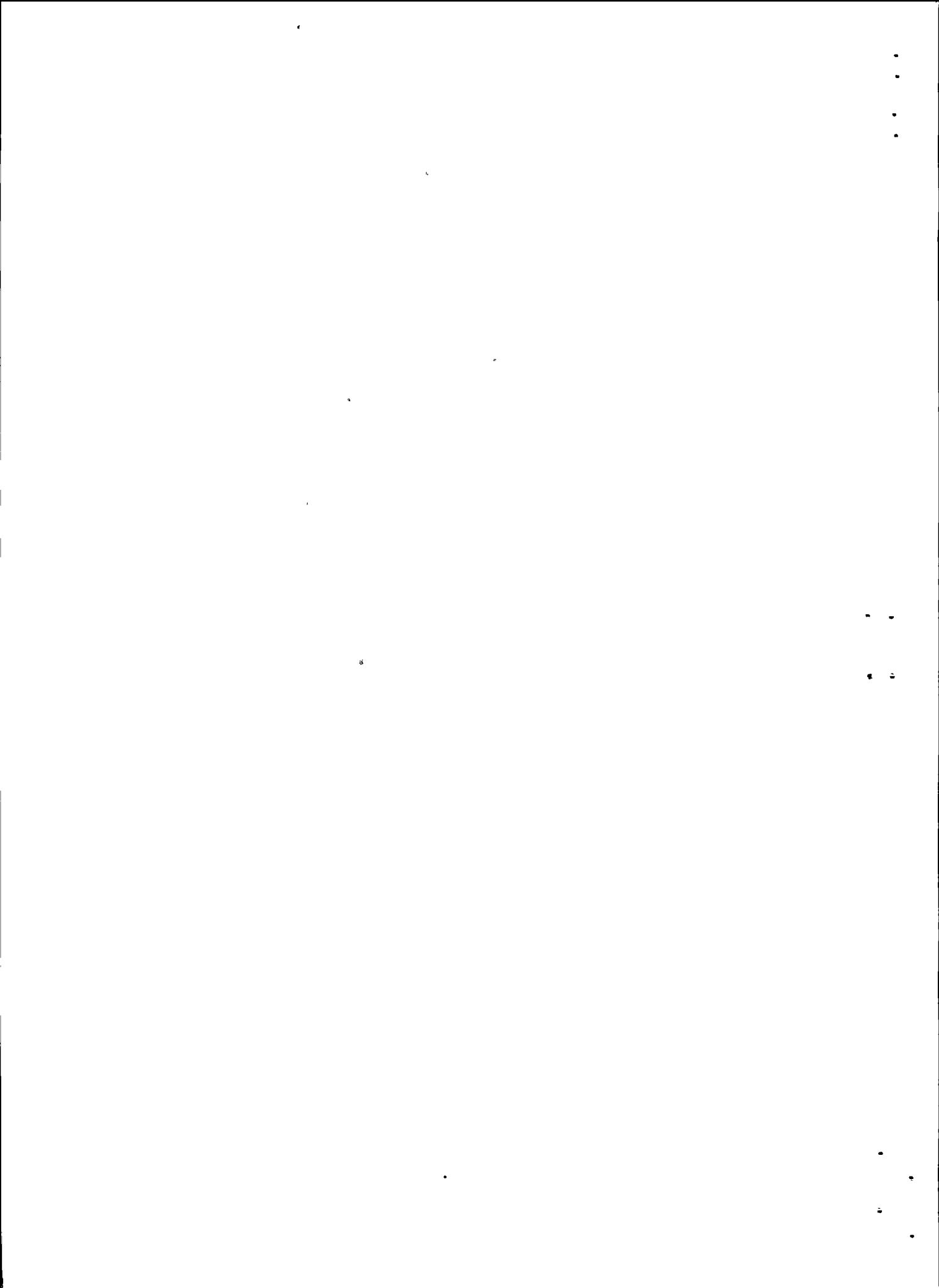
"The armour layer has been damaged twice since its construction. No repairs were made after the first damage in March 1976. During the storm 1-4 December 1976 the armour was further damaged and the wave screen was in places broken down. Profiles showing the surface of the armour layer before and after this second damage has been recorded. 14 different profiles recorded at 860 m to 2100 m from the nearshore end of the breakwater were recorded. In Table [2] the damage to the armour layer during the storm as well as the damage before the storm is presented for the individual cross sections. The definition of damage is given later on. Fig. [28] shows an example profile recorded 860 m from the nearshore end.

Profile (no.)	860	900	1000	1100	1200	1300	1400	1500	1600	1700	1800	1900	2000	2100
Damage before storm	39	28	38	49	38	79	25	45	28	53	37	52	37	56
Damage during storm	41	80	75	47	67	0	60	12	76	57	20	39	17	15

TABLE [2]. Prototype damage in percent.

"THE STORM AT BILBAO 1-4 DECEMBER 1976

"Waves were recorded during the storm by means of a Wave Rider Buoy positioned near the breakwater head as shown in Fig. [26]. Recordings were made over a 10 min period every 4 hours. This is a shorter sampling interval than normally required. The time series are spectral analysed using a Fast Fourier Technique on a 409.6 sec part of each record (1024 data points). 512 spectral density estimates are calculated in the frequency range 0-1.25 Hz giving a frequency resolution  $f = 0.00244$  Hz. Smoothing is made by taking a moving average of 5 "raw" spectral density estimates.



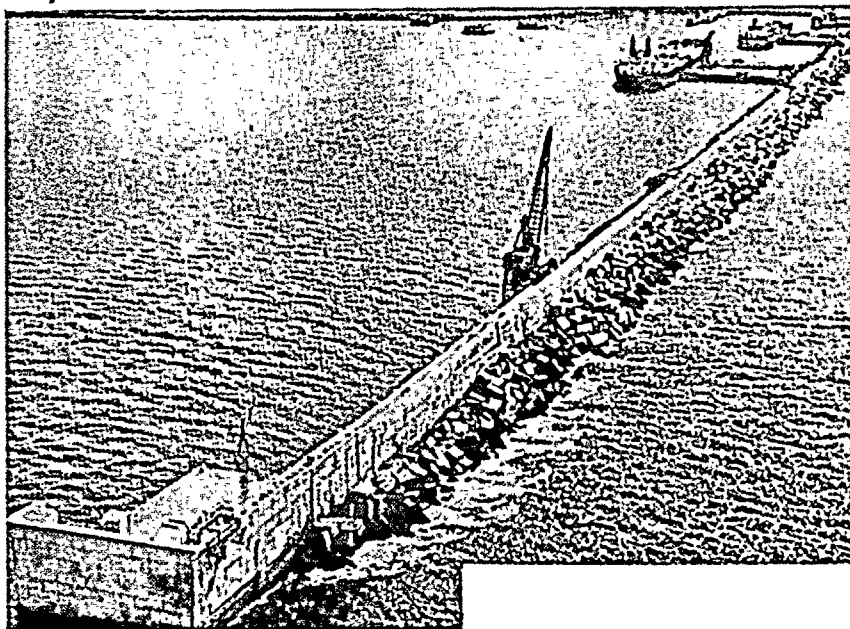


Figure 25. The breakwater at the Oil Harbour of Bilbao.

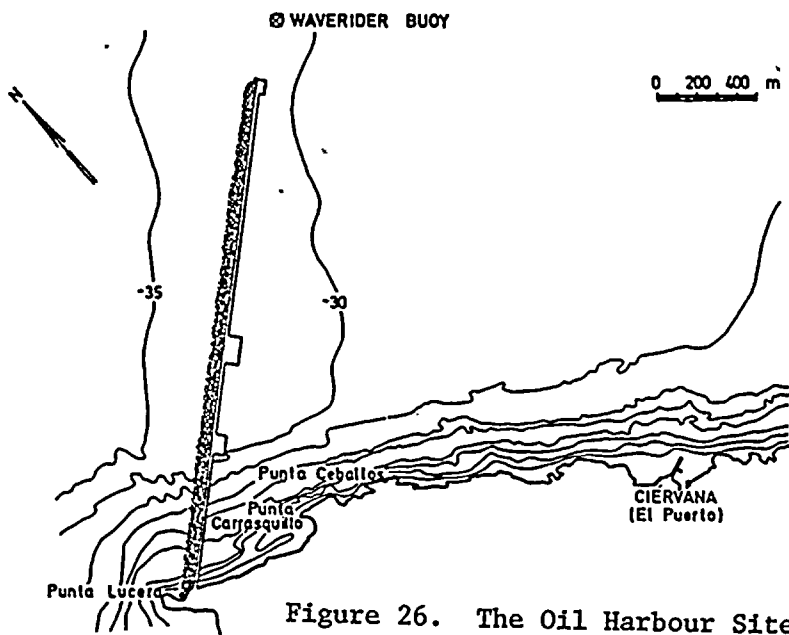


Figure 26. The Oil Harbour Site

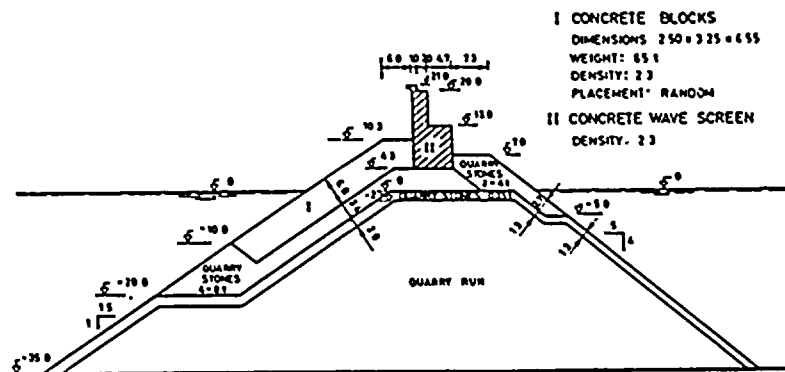


Figure 27. Cross-section of the breakwater

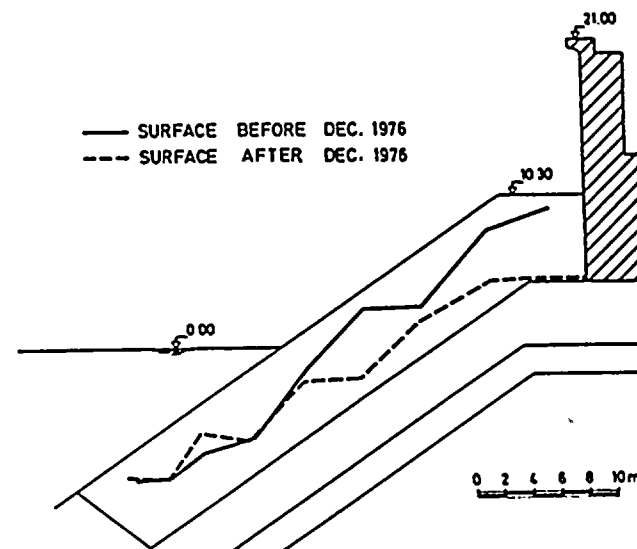
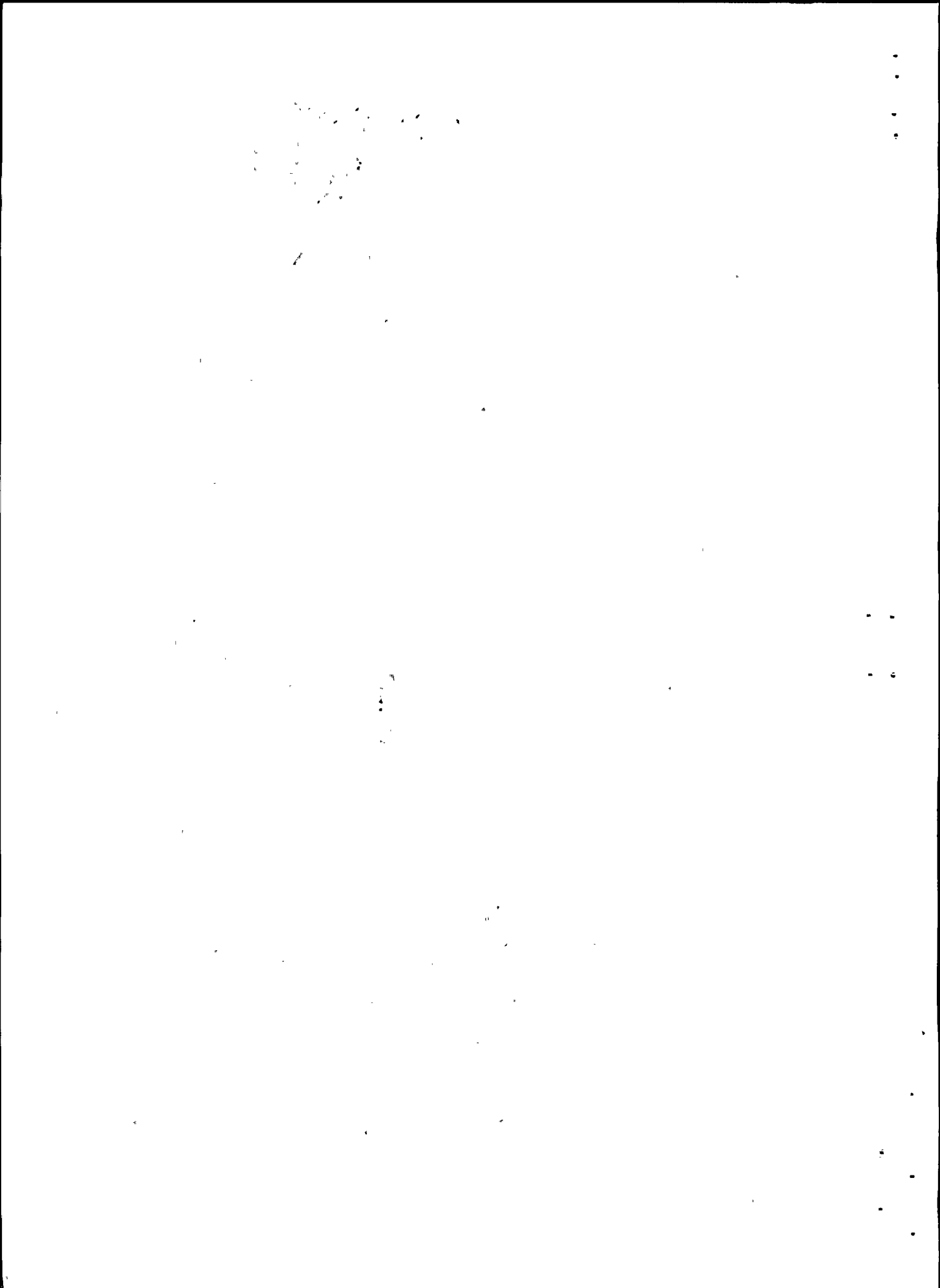


Figure 28. Armour layer profiles recorded 860 m from nearshore end of breakwater



"In Fig. [29] is shown some parameters derived from the obtained spectra. The wave height  $H_{4rms} = 4\sqrt{m_0}$ , where  $m_0$  is the spectral area, and the peak period  $T_p$  are presented for the spectra obtained from time series 1 December at 0600 to 4 December at 0600. The wave heights  $H_{4rms}$  range from 3.10 to 7.86 m while the peak periods lie in the range 10.3 sec to 17.8 sec.

"On 2 December the sea was growing rapidly resulting in the low wave periods. The decay from 1800 to 2200 gave swells and wave periods around 17-18 sec.

"The very rapid growth to the maximum sea state at 0200 on 3 December results in a "new" sea dominated by wind generated waves with quite low periods, around 12 sec. At the end of the storm swells with periods around 18 sec dominated."

#### San Ciprián Harbor, Spain

Losada and Giménez-Curto (1981) report that important damage to the breakwater at San Ciprián, Spain, was found after the storm of March 1980. Numerous dolos in the North Breakwater were fractured, but did not move out of the section. It was estimated that during the peak of the storm the significant wave height was 7 meters, and the mean zero upcrossing period was 14 seconds. The breakwater has been repaired with steel reinforced dolos ( $100 \text{ kg m}^3$ ). The slope of the breakwater has been decreased to 1:2.5. The original cross section is shown in Figure 30.

#### Crescent City Harbor, California

Magoon, Sloan and Foote (1974) report on damage by storm waves to the breakwater at Crescent City, California. They state:

"Crescent City Harbor is protected by a rubble-mound outer breakwater extending S 27° E for approximately 3,700 feet and S 80° E for approximately 1,000 feet (see Figure [31]). This latter portion is called the realigned extension. The outer portion of the Crescent City main breakwater was built of 12 ton per average (armor stone) with slopes of from 2-1/2 to 1 through 4 on 1.

"Originally, it had been planned to extend the structure along the S 27° E alignment toward Round Rock. However, as shown in Figure [32], about 500 feet of breakwater was extensively damaged during the winter of 1950 to 1951 and a realigned 1,000 foot extension was constructed during 1954 to 1957, as shown in Figure 5.\*

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\* In original paper.

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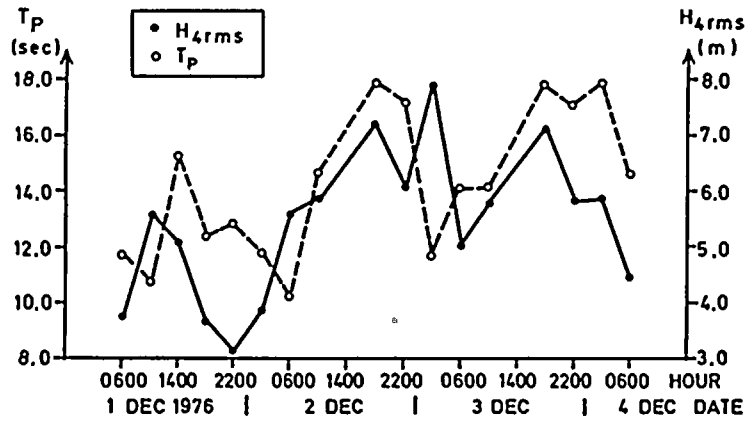


Fig. 29. Wave conditions at the breakwater during the storm December 1976.  
(From Tórur, Mathiesen and Escutia, 1979)

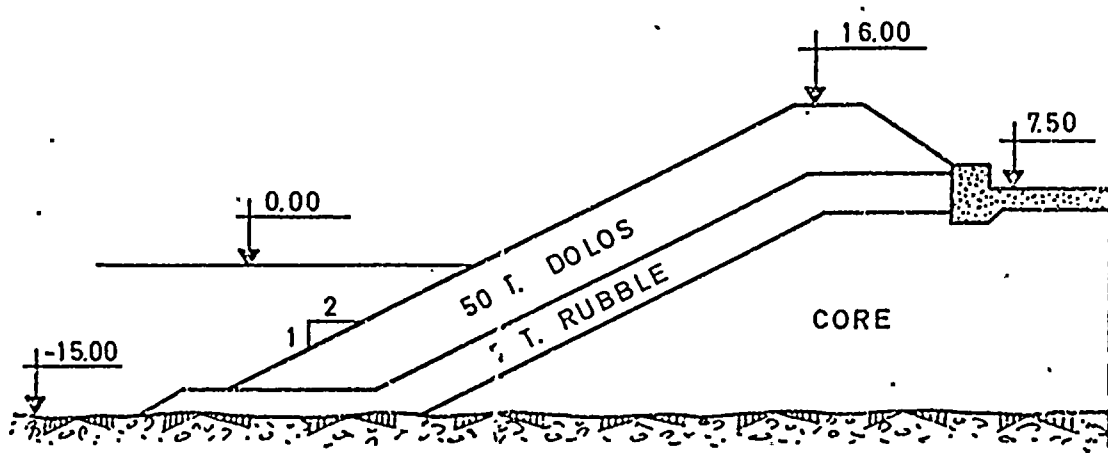
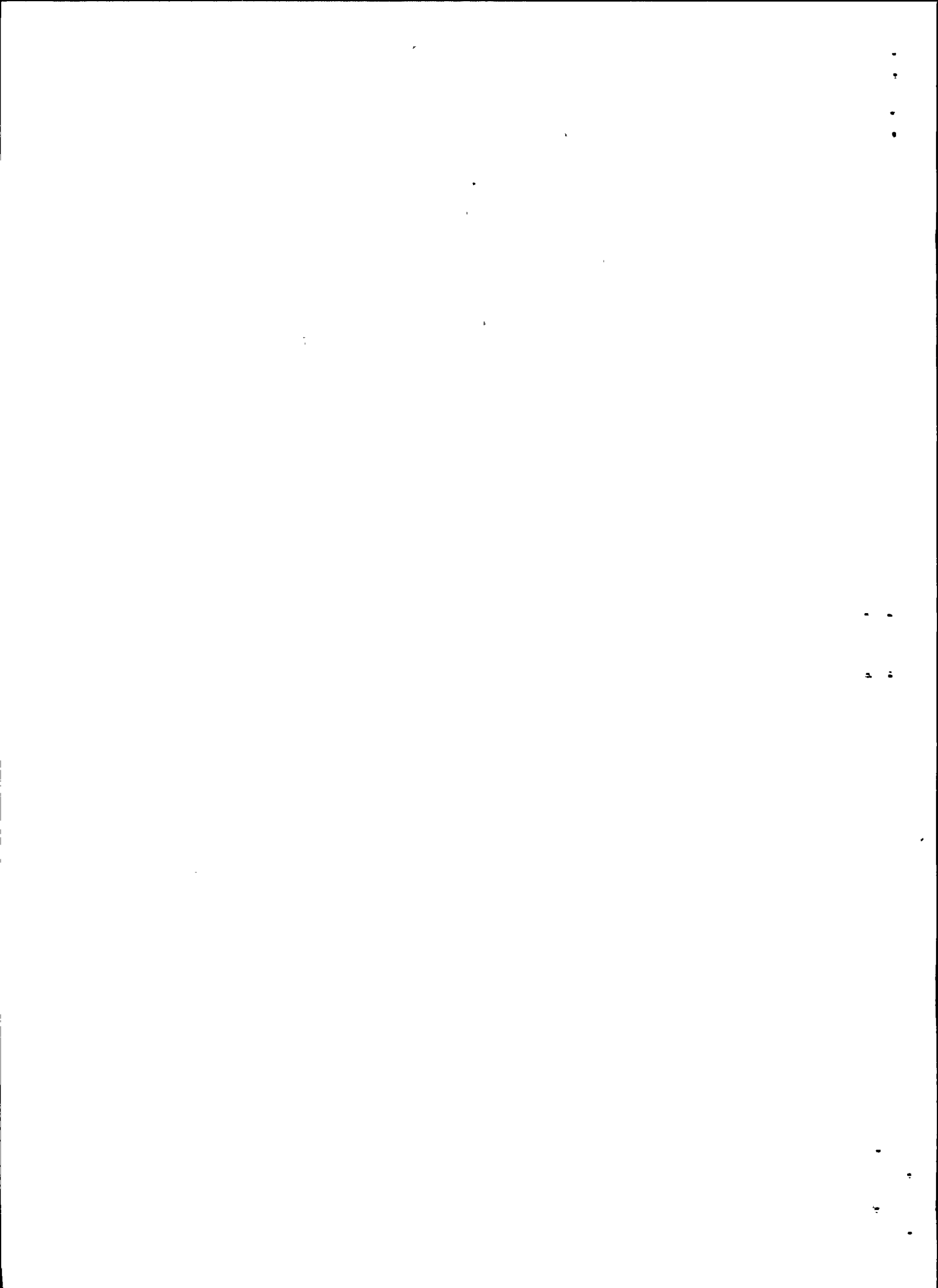
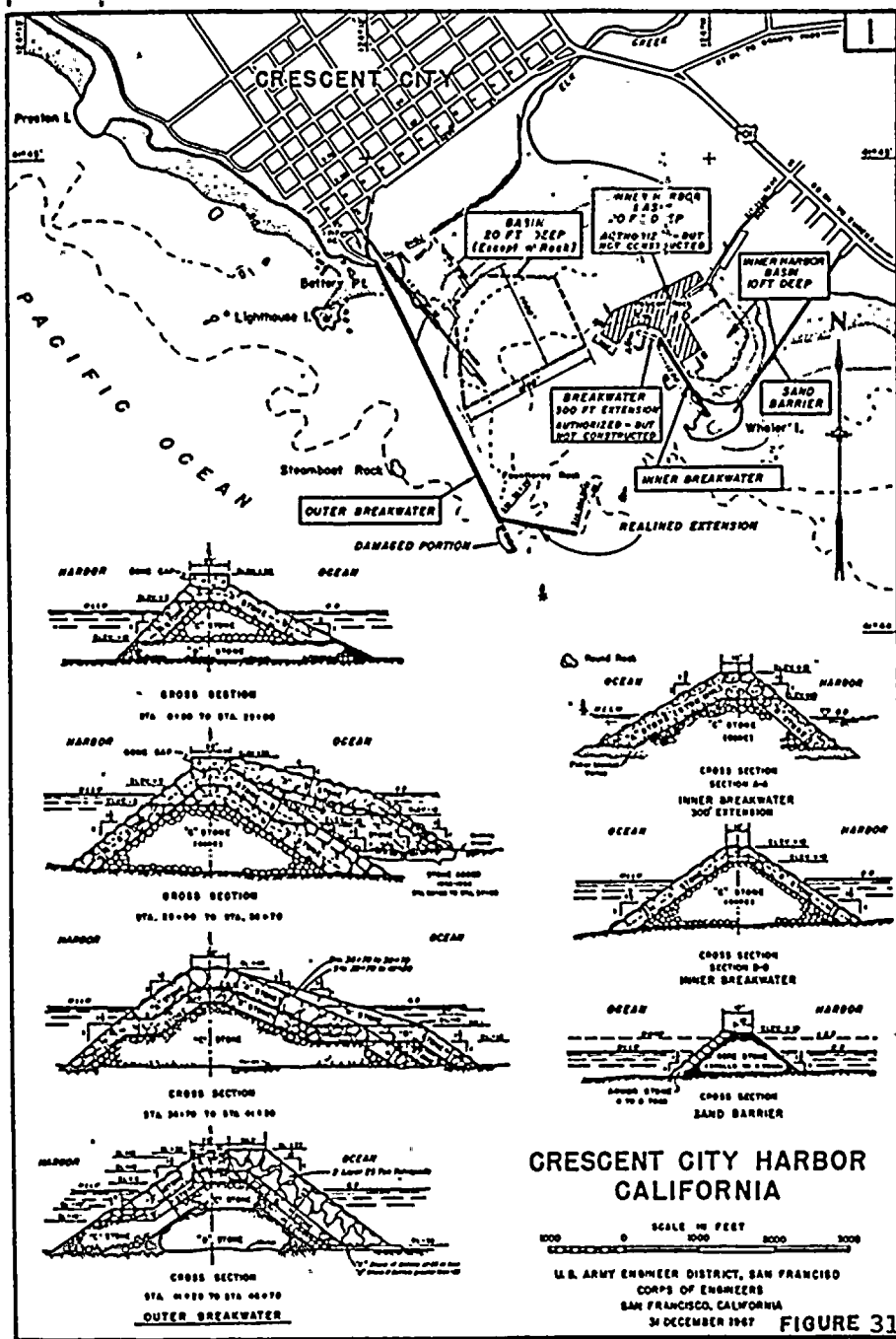


Fig. 30. San Ciprian Breakwater. Cross Section  
(From Losada and Giménez-Curto, 1981)





(From Magoon, Sloan and Foote, 1974)

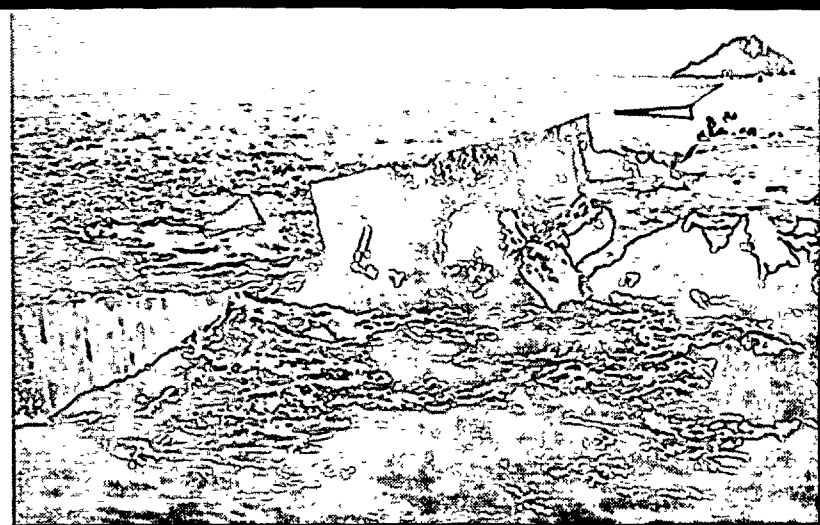


Figure 32. Damaged original extension of Crescent City Outer Breakwater--- viewed from Station 36+70 with Round Rock in distance(upper right) (From Magoon, Sloan and Foote, 1974)

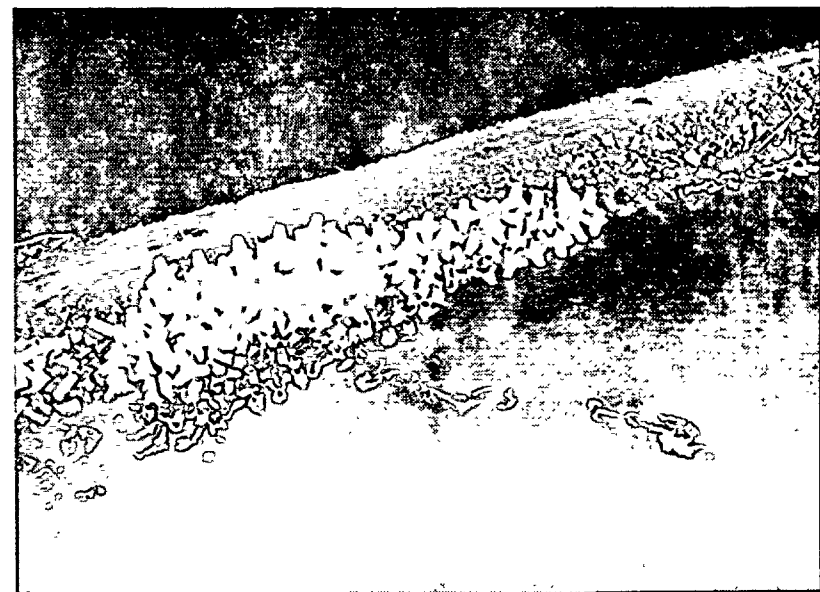


Figure 33. Tetrapod repairs to realigned extension of Crescent City Outer Breakwater (From Magoon, Sloan and Foote, 1974)

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"During the winter of 1956-1957, the stone section of the breakwater extension from Station 36/70 to 39/10 suffered a loss of armor stone and as sufficient stone could not be obtained from local quarries, one hundred and forty 25-ton tetrapods identical to the tetrapods placed in this section were placed in two layers from approximately Station 37/10 to 38/10 and were placed one layer from approximately Station 38/10 to 39/10 (see Figure [33]). These repairs were completed in June 1957.

"This section was extensively damaged during a severe storm in February, 1960, which resulted in the movement along the breakwater of 25-ton tetrapod units of distances of at least 100 feet (see Figure 7\*) and extensive breakage of tetrapods (see Figure 8\*). At the present time, approximately half of the tetrapods are damaged or broken, caused apparently either by movement or impact of large stone units or broken concrete fragments. The breakwater is also subject to severe overtopping (see Figure [35]). Two hundred and forty-six 40-ton dolosse were placed as repair between Stations 35/00 and 37/00 (area of maximum overtopping in Figure [34]) in late 1973 (see Figure 22\*)." .

#### Humboldt, California, Jetties

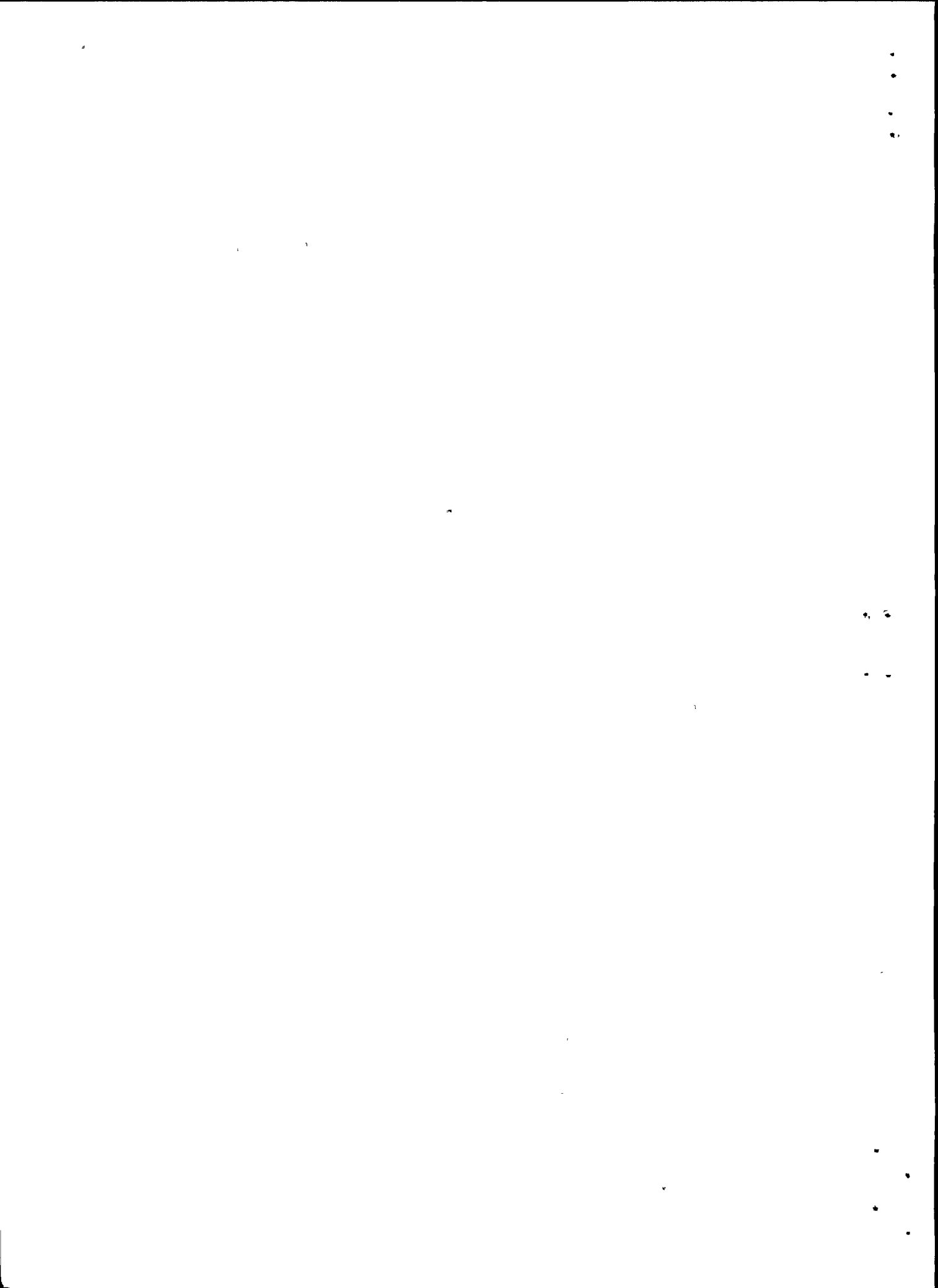
Magoon, Sloan and Foote (1974) report on damage by storm waves to the jetties at the entrance to Humboldt Bay, California. They state:

"The Humboldt jetties, shown on Figure [36] have probably experienced the most severe wave attack and necessitated the greatest amount of maintenance of any structure studied. The structures were initiated in 1889, and, although the exact quantities of new construction and maintenance stone cannot be determined exactly, the quantity of stone placed for repair has been greater than the quantity placed in the initial construction (more than 1,000,000 tons). Documented storm attack on the structure (see Figure [37]) has indicated that waves completely cover the seaward portions of the structure, with an average deck elevation of 26 feet MLLW. Several types of failure have been repeatedly experienced at the Humboldt jetties. Damage by uplift pressures is shown in Figures 13\* and 14.\* In an effort to prevent slope failure by overtopping, portions of the back sides of the structure were covered with concrete. As shown in Figures 15\* and 16,\* initial cracking of this concrete eventually breaks up completely.

"During the 1960s, a concrete monolith was constructed at the seaward head of the North Jetty. A ringing levee was placed around the head, with the intention of pouring mass concrete into the area surrounding the existing head. However, wave action through the ring levee resulted in washing away of the concrete as it was poured. Soon after construction, the ring levee was washed away resulting in

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\* In original paper



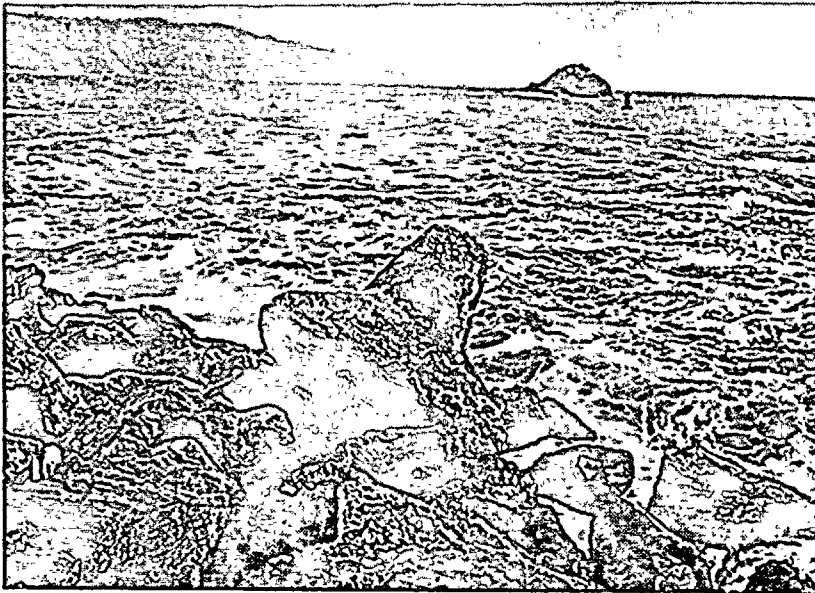


Figure 34. Storm wave displacement of originally submerged tetrapod from repaired area shown on Fig. 6.\*  
(From Magoon, Sloan and Foote, 1974)

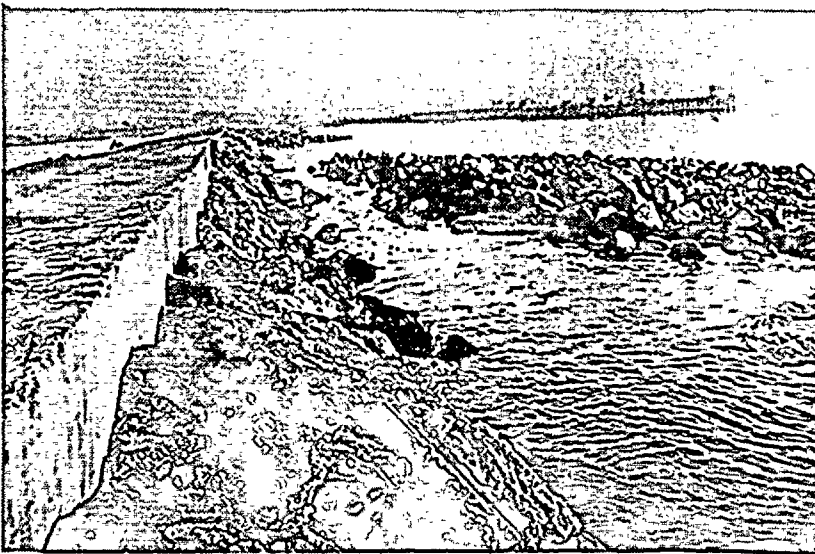
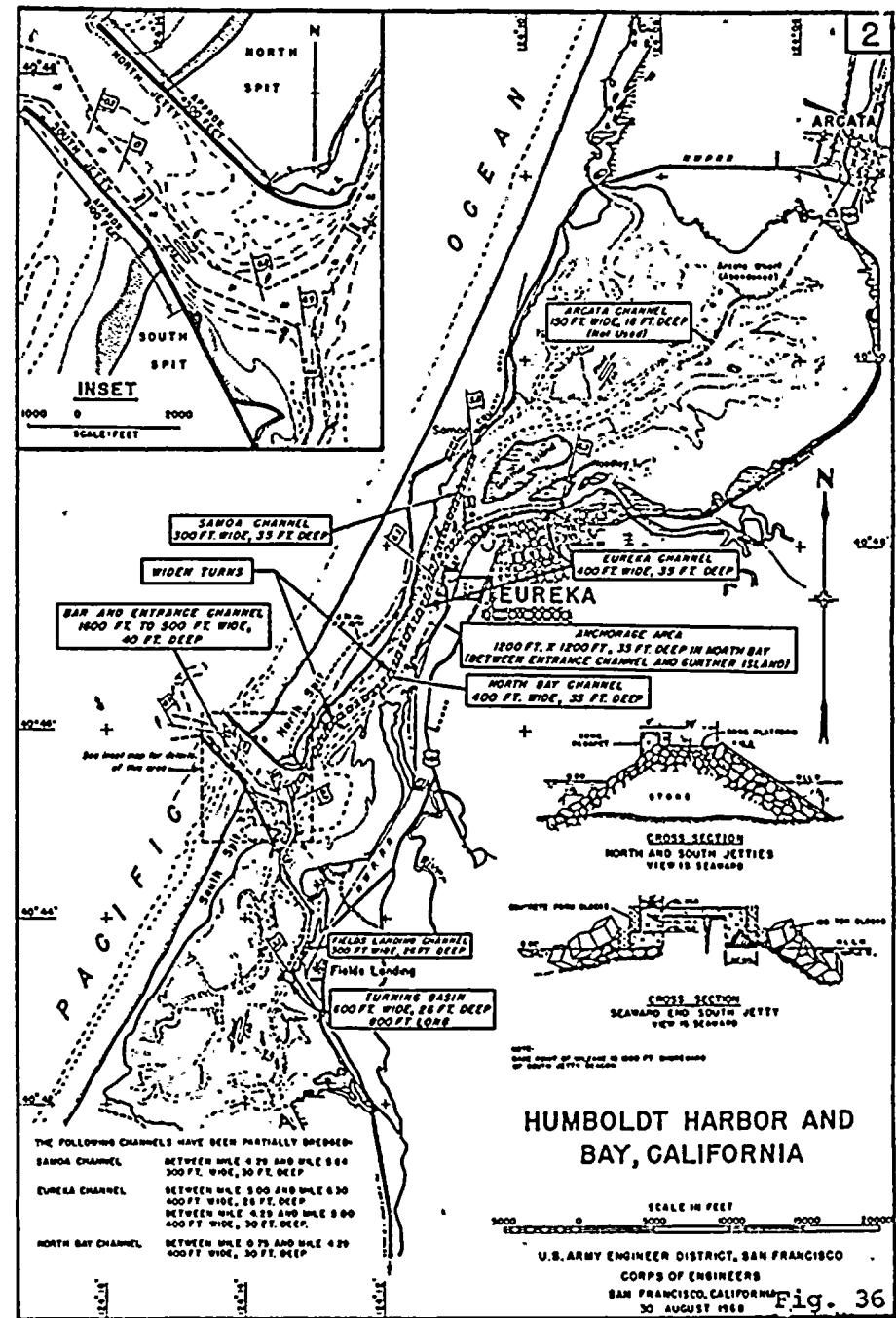


Figure 35. Wave trench resulting from the overtopping shown on Fig. 9--\* Crescent City Outer Breakwater.  
\*Original figure (From Magoon, Sloan and Foote, 1974)



(From Magoon, Sloan and Foote, 1974)

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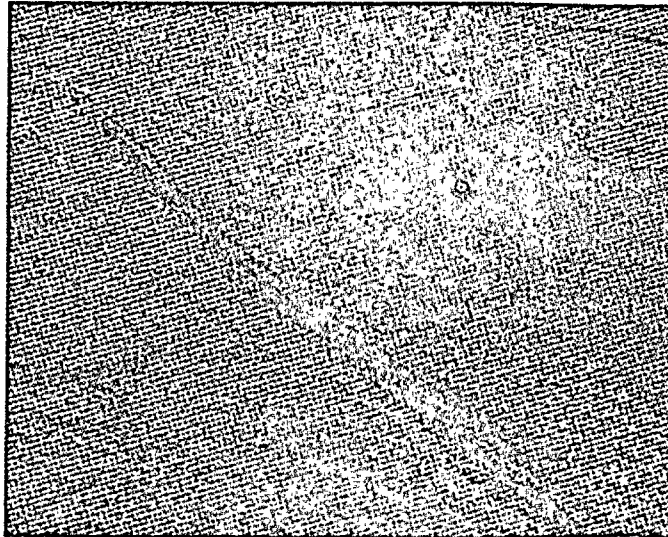


Figure 37. Jettied entrance to Humboldt Harbor and Bay during storm activity of February 1960. (From Magoon, Sloan and Foote, 1974)

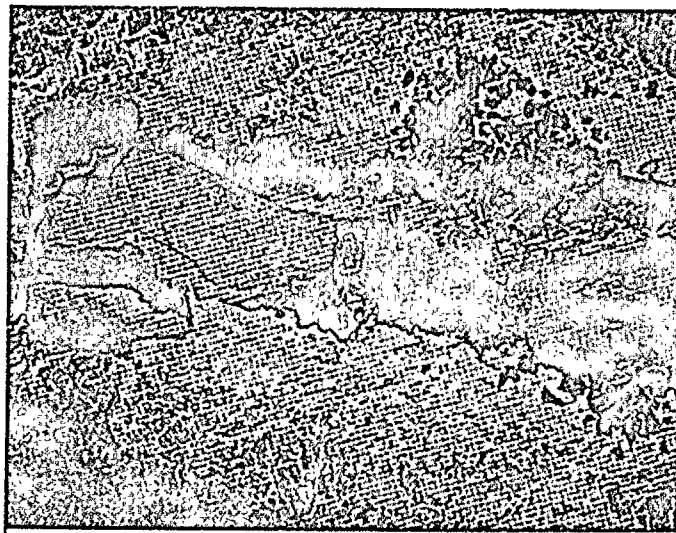
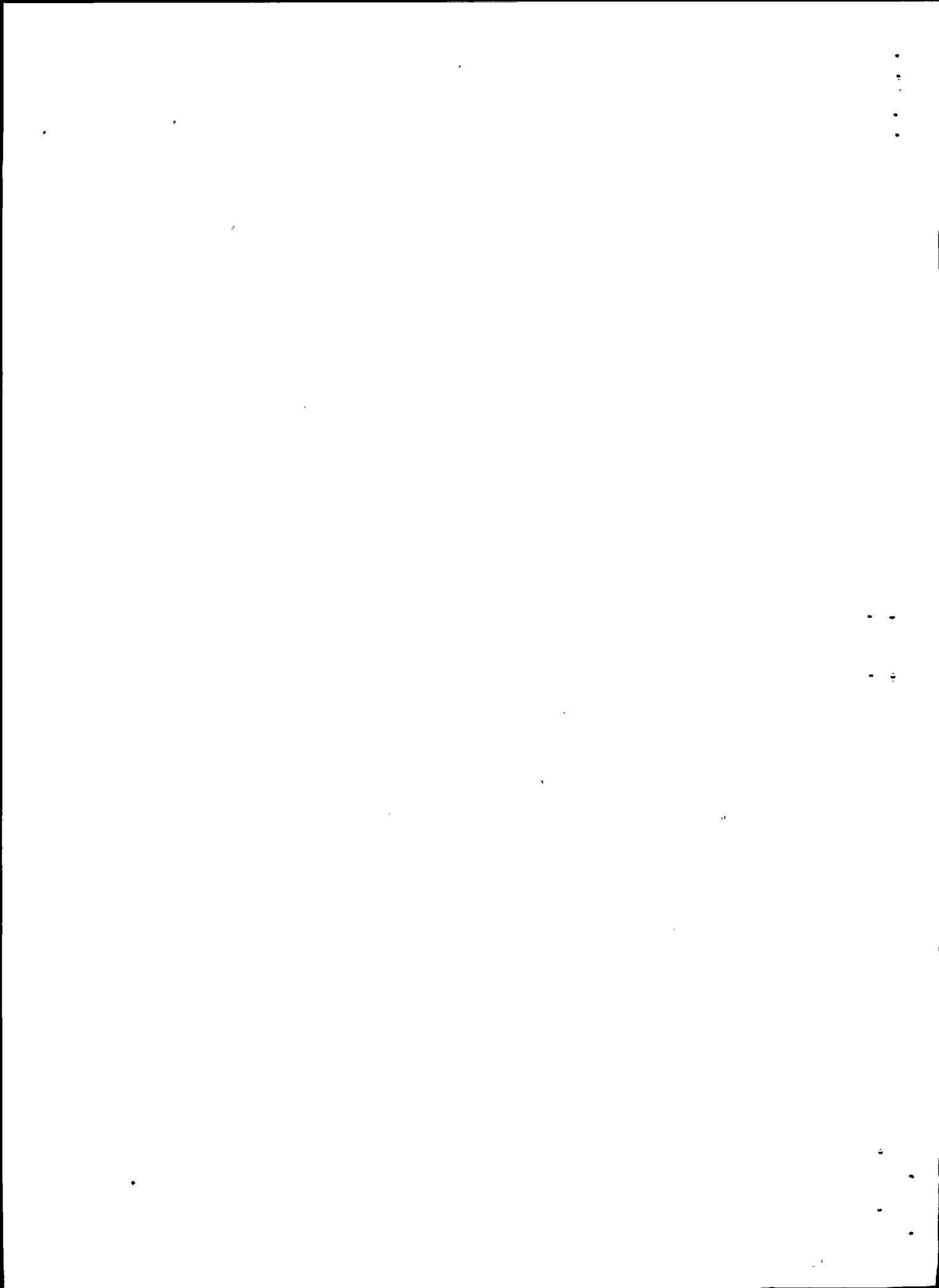


Figure 38. Monolithic breakup--Humboldt Harbor and Bay North Jetty. (From Magoon, Sloan and Foote, 1974)



an overhanging concrete mass shown in Figure 17.\* This subsequently collapsed, resulting in large broken pieces as shown in Figure [38]. In efforts to stabilize the heads of these structures, a great number of concrete blocks (see Figure 19\*) with maximum weights of 100 tons have been placed along the sides of the jetties. Essentially all of these blocks have been dislodged. Many of the blocks and supporting stones have been displaced landward and moved landward (see Figure 20\*).

"As described by Magoon and Shimizu, both the North and South Jetty heads have been repaired with 42 and 43-ton concrete dolosse armor units (see Figure 21\*). Most of these units are reinforced; however, as shown in Figure 22,\* unreinforced and fiber reinforced units have been placed at both the Humboldt jetties and at the Crescent City outer breakwater (see Figure 22\*). Based on the available inspections which do not include those units placed under water, about 12% of the unreinforced units are broken and none of the fiber reinforced units are broken."

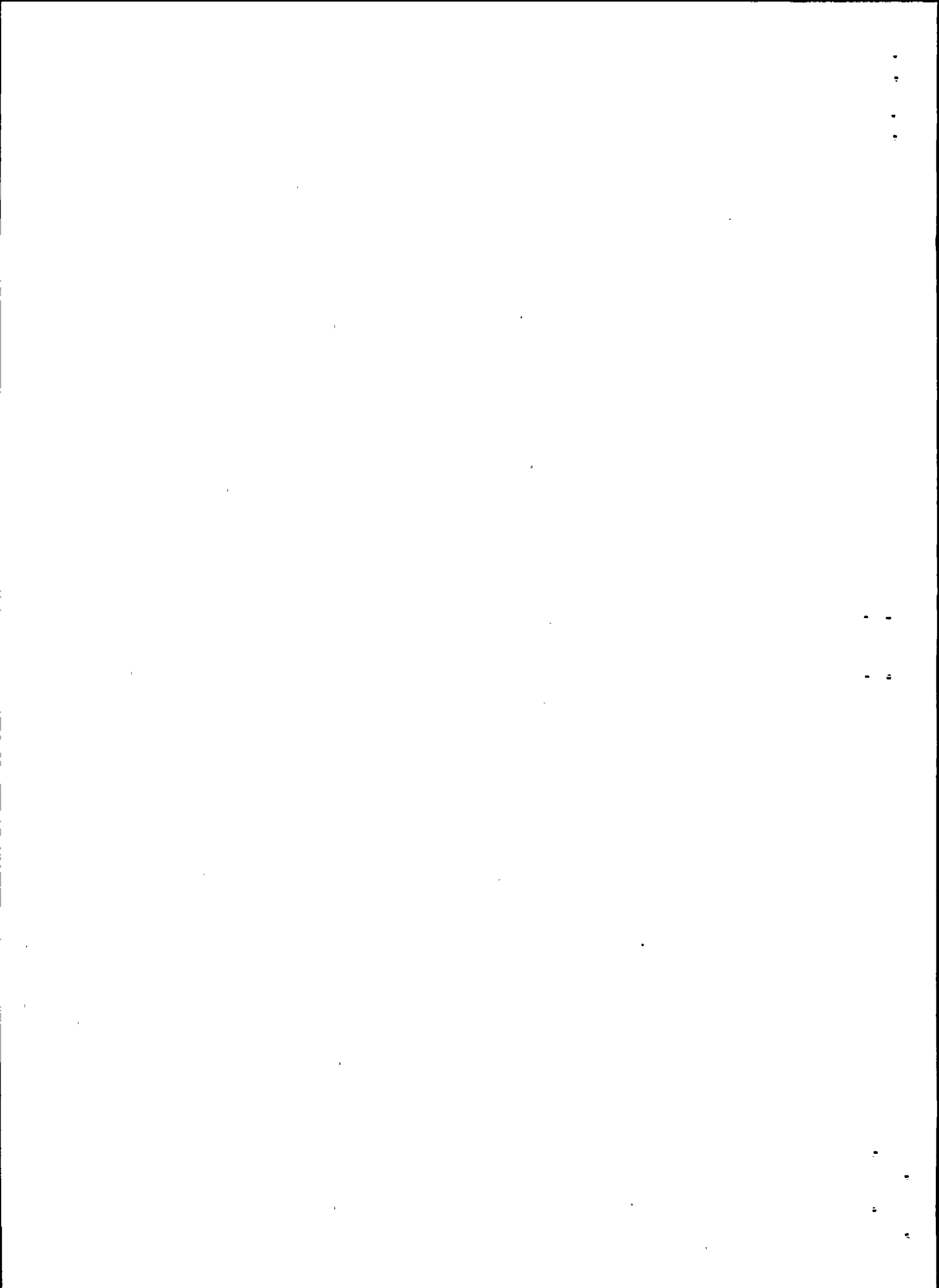
Kahului Harbor, Maui, Hawaii

Palmer (1960, p. 45) reports:

"*Kahului Harbor*.--Kahului Harbor is located on the northern coast of the island of Maui. A small cove behind the reef, within which 19th century whalers anchored, was developed into a deepwater port by dredging and by the construction of the east and west rubble-mound breakwaters that were 2,850 ft and 2,396 ft in length, respectively. The breakwaters were completed in 1931. Major repair of the structures at, and near, the heads was accomplished in 1957. A record of damages and cost of repairs for the past 12 yr is shown in Table [3].

TABLE [3].--KAHULUI BREAKWATER DAMAGES AND COST OF REPAIRS

<u>Date of Storm</u>	<u>Damaged Area</u>	<u>Cost</u>
January 1947	Seaward Ends of Both Breakwaters, Center and Near Root of East Breakwater	\$ 500,000
January 1952	Seaward Ends of Both Breakwaters	80,000
March 1954	Seaward Ends of Both Breakwaters	1,300,000
November 1958	Breach in Stone Portion East Breakwater and End of East Breakwater	Not Yet Repaired



"The breakwaters were repaired using 33-ton tetrapods in the armor layer. Two layers of components were placed on the slopes around the heads of the breakwaters. The seaward slope and the end slope (along the breakwater center line) were set at 1-on-3 with a transition to a 1-on-2 landward slope."

A model study was made in March 1958 of a breakwater at another harbor. This study revealed a weakness in the design of the ends of the Kahului breakwater. Palmer states:

"The dire prediction of the model study proved all too accurate. The severe storm of November 22, 1958, caused substantial damage to the east breakwater. The upper aerial photograph, in Fig. [39] shows waves attacking the Kahului breakwaters during this storm. The picture was taken about 12 hr after the peak of the storm, when it had abated to some extent. Note the 70-ft breach in the old stone portion of the east breakwater. The breach later widened to about 150 ft. Also, note that the wave completely obscures the end of the west breakwater (to the left). It was estimated that at the peak of the storm the breaker heights were 25 ft. About thirty 33-ton tetrapods were rolled away from the inboard quadrant of the end of the west breakwater where the slope was transitioned from 1-on-3 to 1-on-2 (see lower photograph in Fig. [39]). Some of them rolled as far as 100 ft on a more or less level grade. Of the units on the seaward slope of the breakwater, three were broken."

Sullivan (1979) presented a case history of the Kahului Breakwater.

He states:

"The first breakwater improvements constructed by the Corps consisted of a 400-foot-long extension to the east breakwater completed in 1913. In 1919, the Corps constructed the west breakwater to a length of 1,950 feet. In 1931 the east and west breakwaters were extended to their present lengths of 2,766 feet and 2,315 feet, respectively. The original east breakwater and the Corps improvements were rubblemound structures built of dense basaltic rock. A typical cross-section of the outer ends of both breakwaters is shown in Figure [40]. The breakwater crest elevation was +13 feet and the crest width was 15 feet. The base below -15 feet was quarry run material with a 25-pound minimum and a side slope of 1V on 1H. The core above -15 feet was constructed of 2-ton minimum stone, with 50-percent of the stone being 4-ton minimum. The armor layer above -15 feet on both sides was composed of a single layer of keyed and fitted 8-ton minimum stone. The side slope above -15 feet was 1V on 2H at the breakwater heads on 1V on 1.5H along the trunks.

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FIG. 39. STORM DAMAGE OF KAHULUI BREAKWATER  
(From Palmer, 1960)

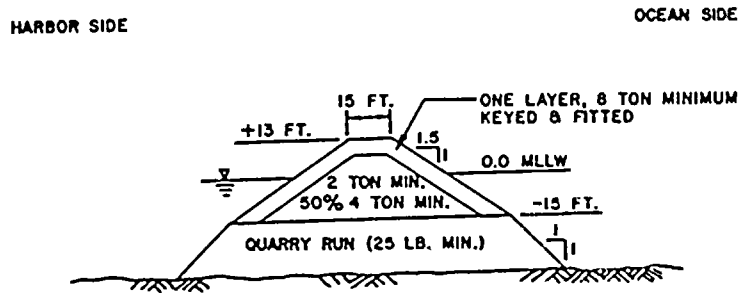
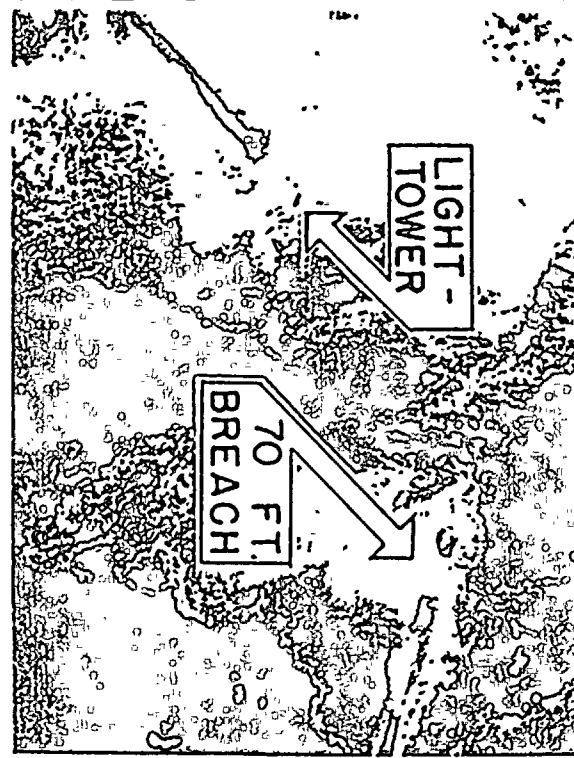
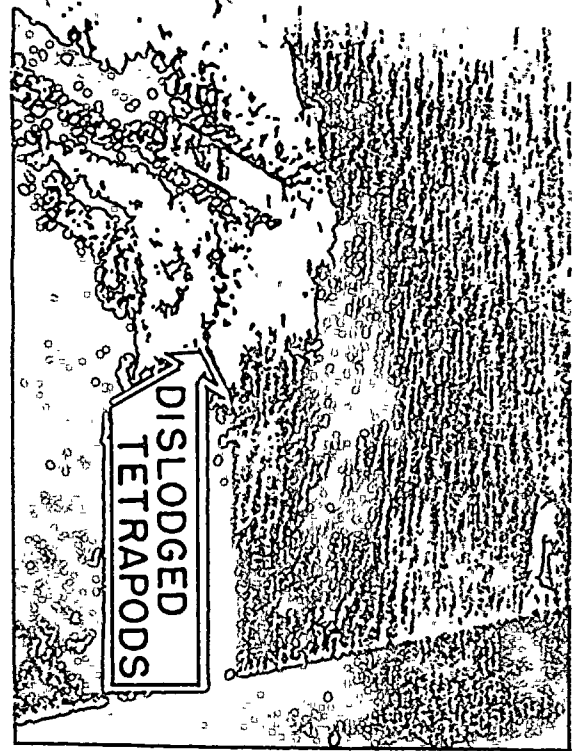


FIGURE 40. TYPICAL ORIGINAL RUBBLEMOUND BREAKWATER CROSS SECTION.  
(From Sullivan, 1979)

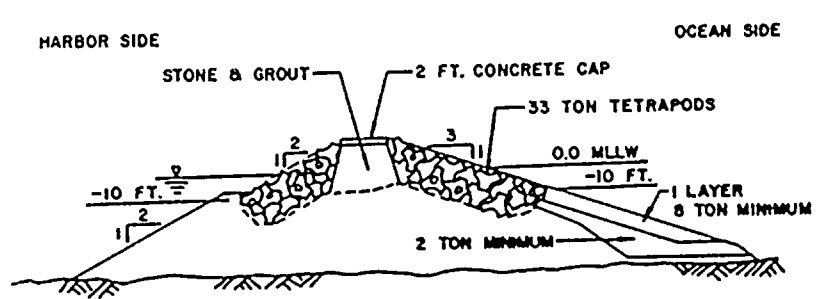
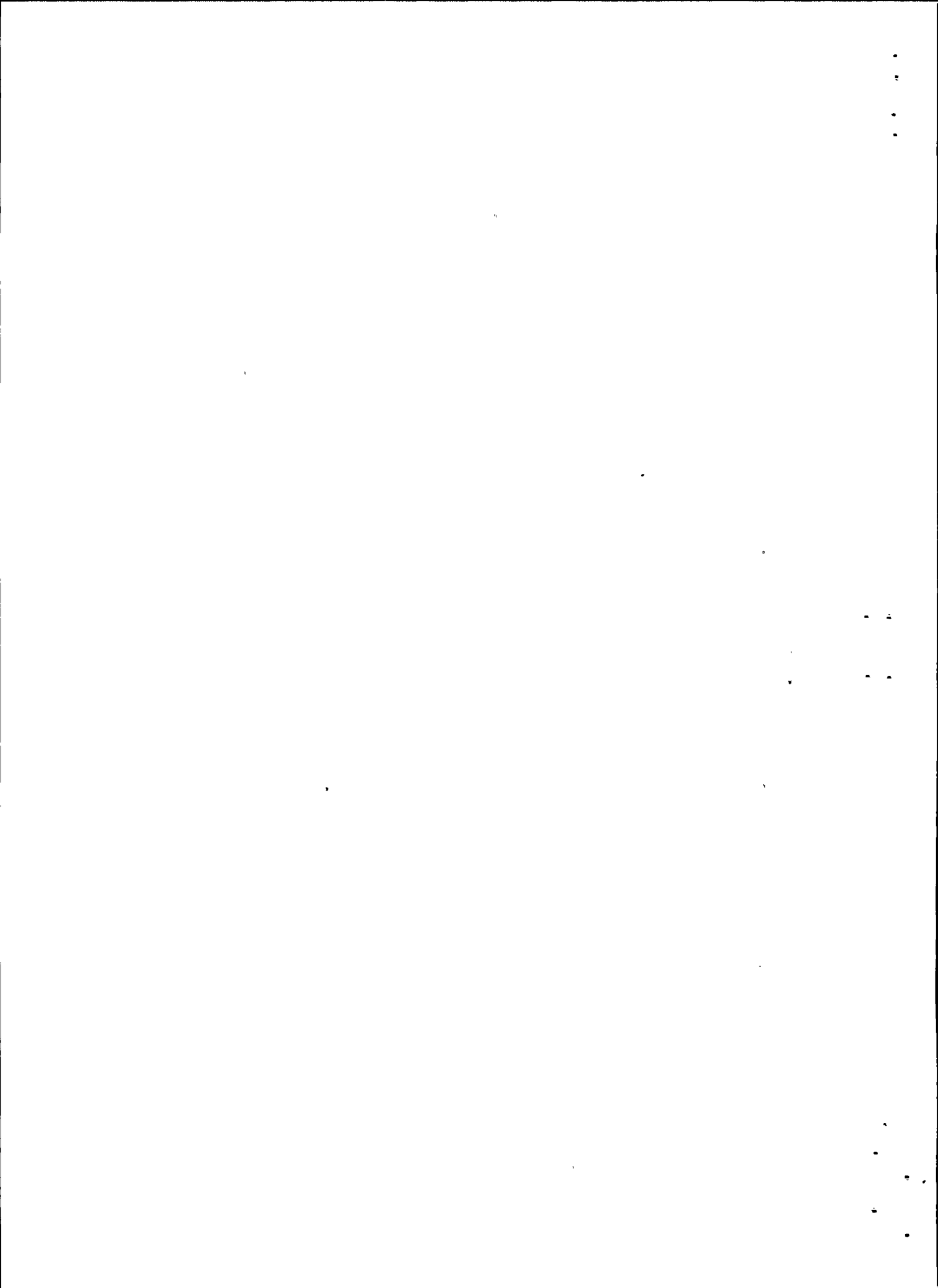


FIGURE 41. TYPICAL TETRAPOD BREAKWATER HEAD CROSS SECTION.  
(From Sullivan, 1979)





"The breakwaters required major repair quite regularly. Prior to 1954, the routine maintenance and repair practice was to restore damaged sections of breakwaters approximately to their original condition. Maintenance costs between 1931 and 1954 exceeded \$1 million.

"After the breakwaters were severely damaged by storm waves in 1954, it was decided that future repair work should be based on current design criteria rather than restoration to the approximate original condition."

\* \* \*

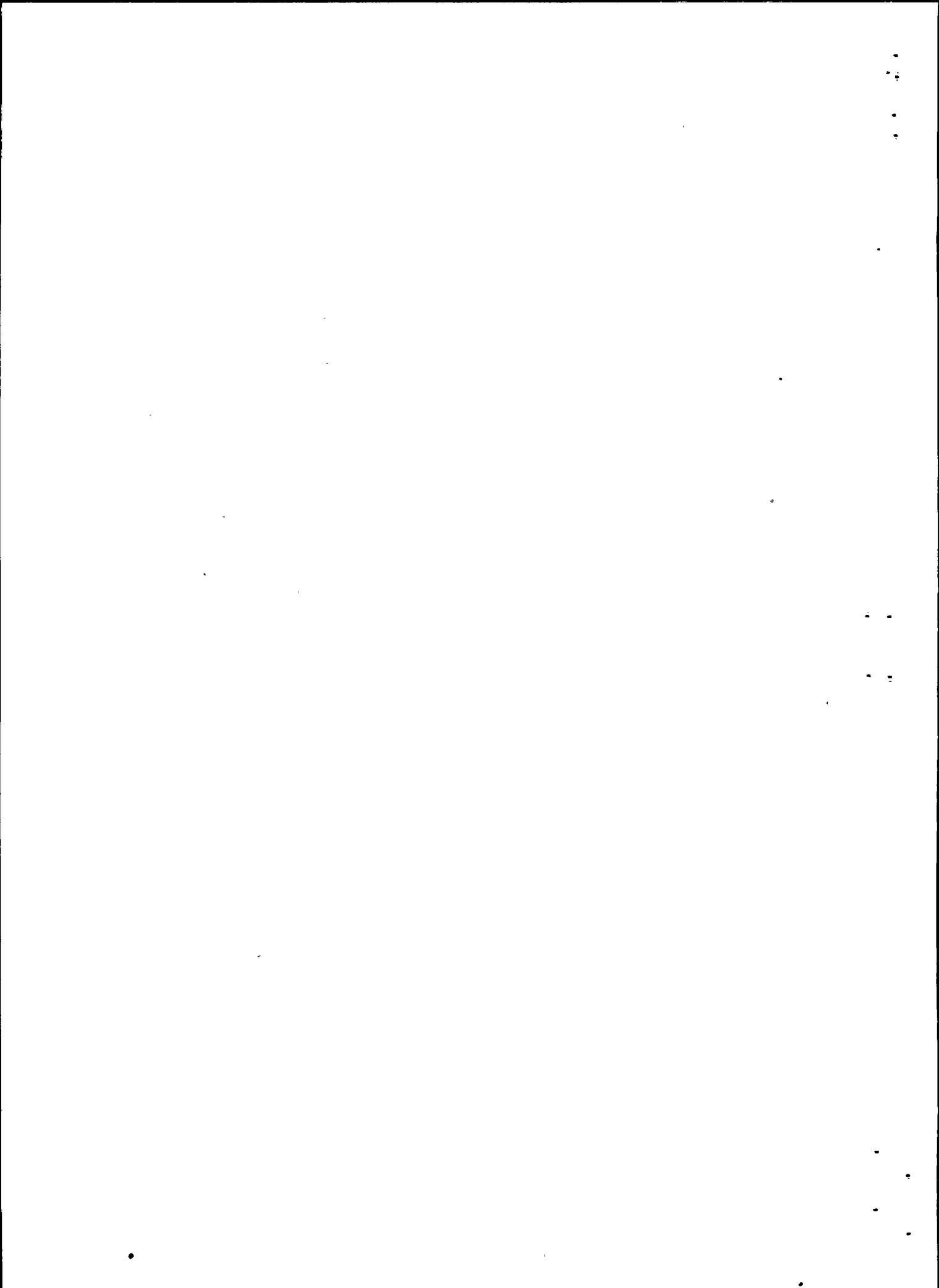
"The area of repair is shown on Figure [41] and a typical repair section is shown on Figure [42]."

\* \* \*

"To prevent encroachment of the structures on the entrance channel the breakwaters were shortened slightly during the 1956 repair and a 1V on 2H slope was used on the inboard quadrant of the armor wrap around on the heads of the breakwaters. The steep slope proved to be a major design deficiency. No model studies were conducted for the 1956 repairs. Subsequent model tests of tetrapods for improvements to Nawiliwili Harbor on the island of Kauai disclosed the deficiency due to the 1 on 2 slope, and in November 1958, about 2 years after completion of the repairs, the ends of both breakwaters were severely damaged by storm waves estimated to be only 23 feet high at the structures. The storm waves displaced about 7 tetrapods from the head of the east breakwater and swept away all of the head of the west breakwater. The waves also breached the east breakwater at the junction of the tetrapods and the original armor stone as shown on Figure [43].

"Stop gap repairs consisting of construction of a heavy monolithic concrete core and the heaviest available armor stone (12-ton minimum) placed on the seaward slope were done in 1959, and design analysis for a major rehabilitation was initiated. The stop gap repairs of the break in the east breakwater were made on a setback alignment to provide a flatter seaward slope.

"Based on the model tests the tetrapods on the top portion of the inboard quadrant of the east breakwater were reinforced with a new armor layer consisting of one layer of 35-ton tribars overlain by one layer of 50-ton tribars, and the east breakwater trunk was armored with 2-layers of 35-ton tribars from the crest to the toe. Repair of the inboard quadrant of the west breakwater where the tetrapods had completely failed was accomplished using 2-layers of 50-ton tribars to a depth of -20 feet and 35-ton tribars to the toe of the slope. All the tribars were random placed over the existing slope which varied from about 1V on 3H to 1V on 6H above MLLW, and from 1V on 1.5H to 1V on 3H below the water line. The



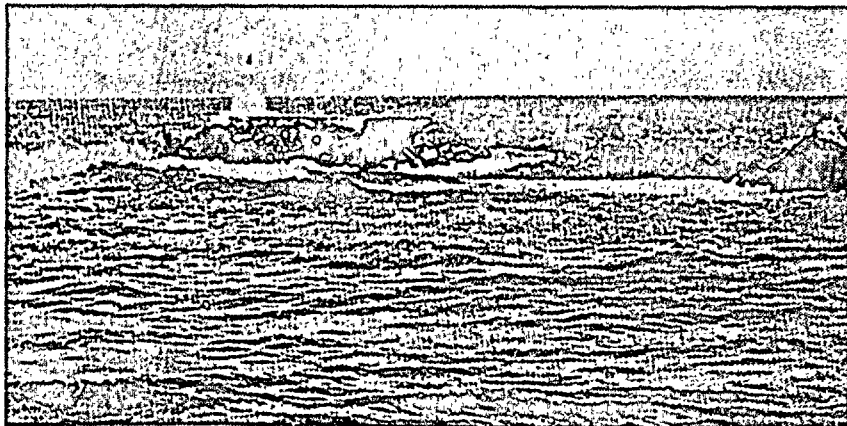
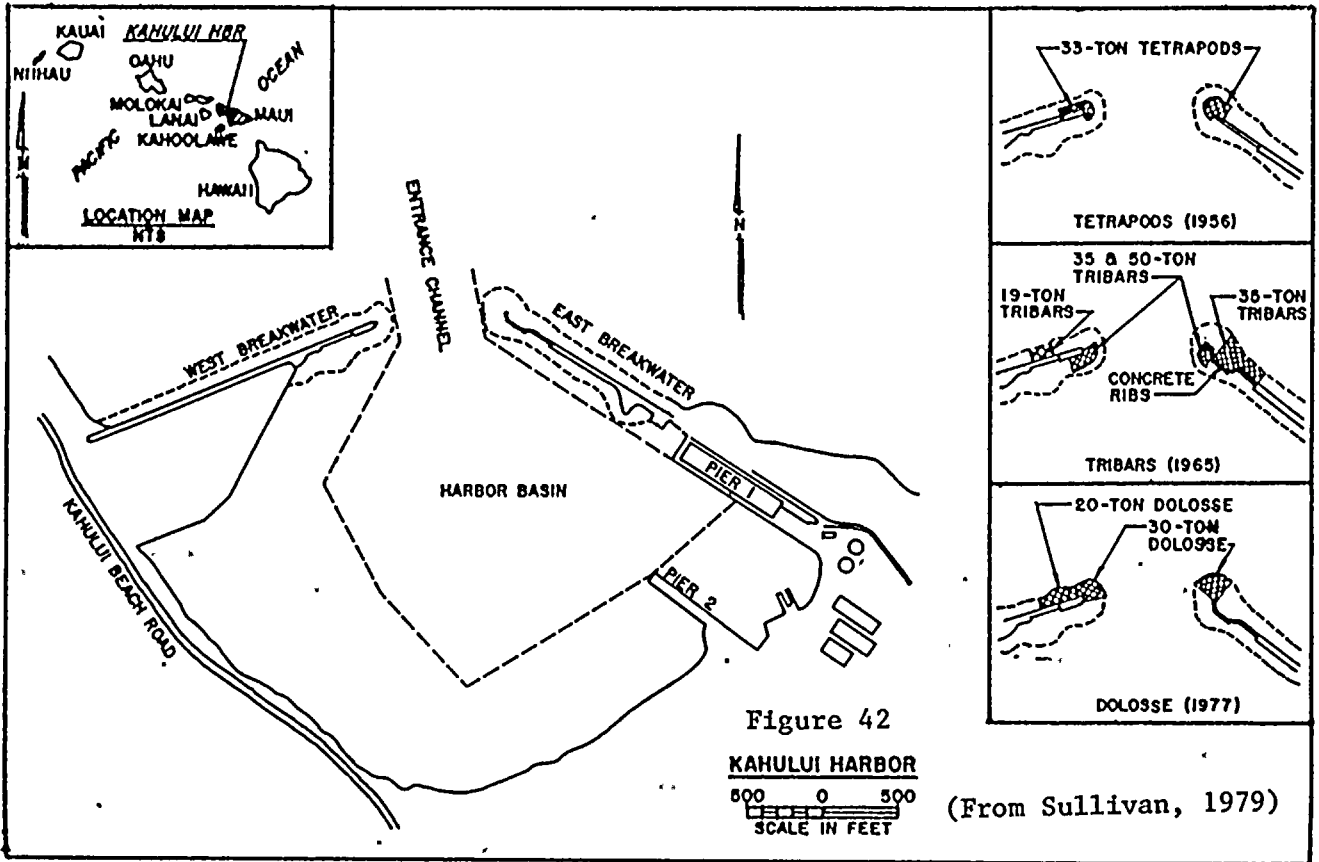


Figure 43. Damage to east breakwater, November 1958  
(From Sullivan, 1979)

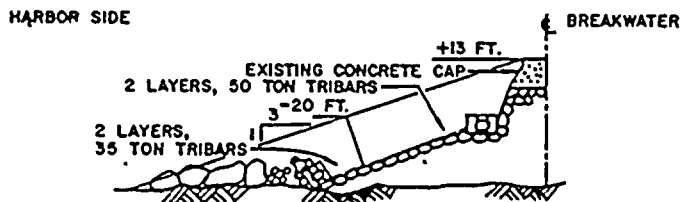


Figure 44a. Typical tribar breakwater head section  
(From Sullivan, 1979)

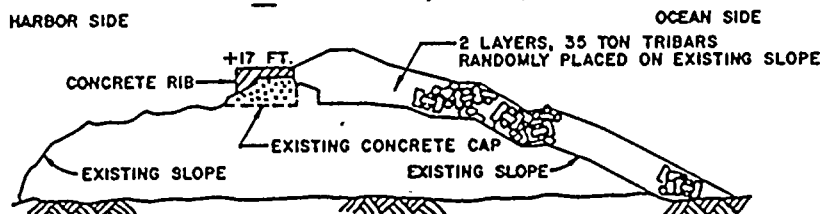
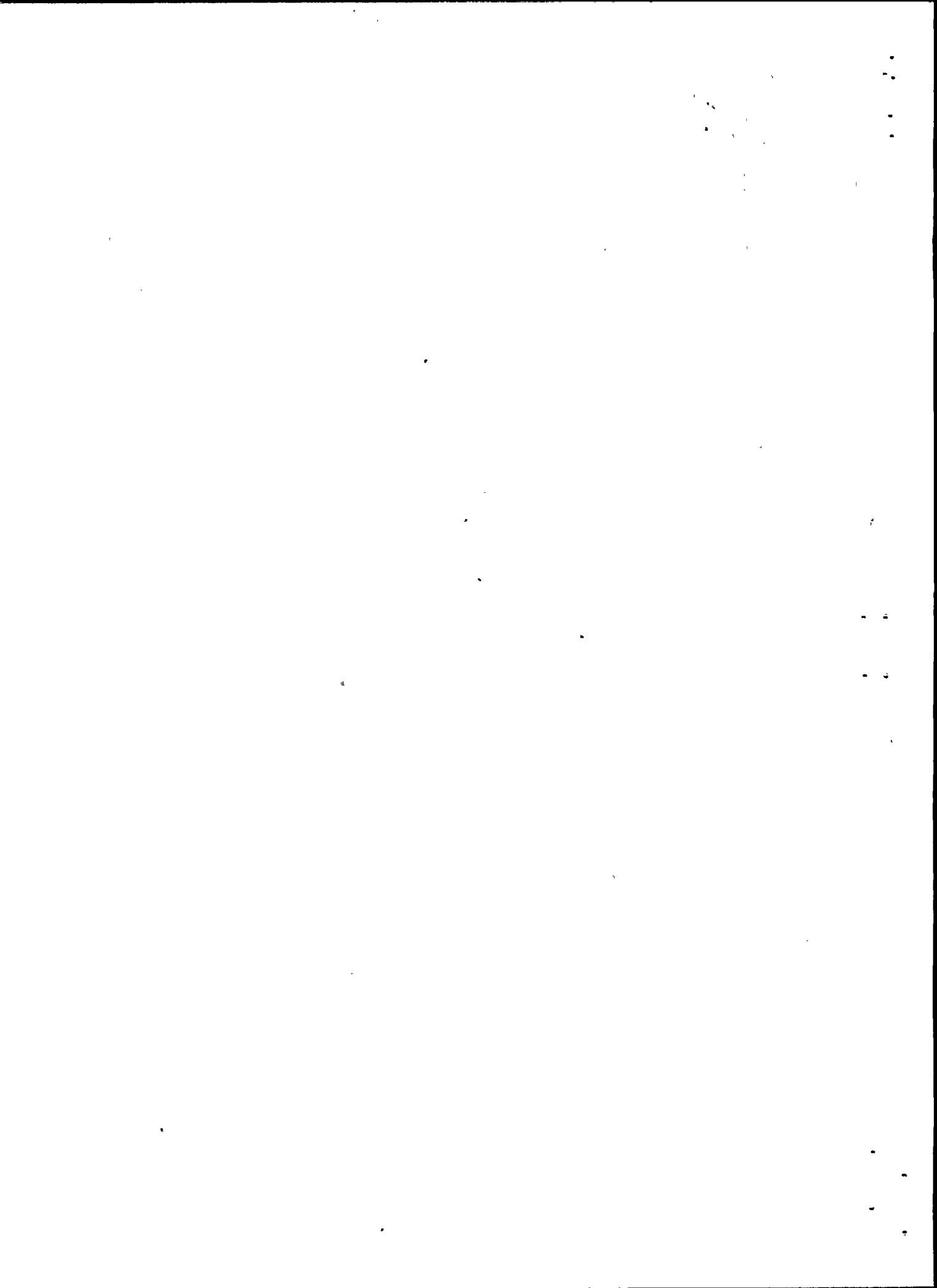


Figure 44b. Typical tribar breakwater trunk section  
(From Sullivan, 1979)



relatively steep slopes of portions of the breakwater heads necessitated the use of 50-ton tribar units, as the model tests had shown that 35-ton units were not satisfactory on the breakwater heads with slopes steeper than 1V on 6H. Reinforced concrete ribs were constructed on the crest of the existing structures to prevent the tribars from being rolled over the crest. ... Typical tribar cross-sections are shown on Figures [44a,b].

\* \* \*

"A breakwater surveillance program was initiated in 1966 to monitor the effectiveness of the precast armor units and correlate movement of the armor units with storm waves. A wave gage was installed approximately 1,860 feet seaward of the harbor entrance and selected armor units were tagged and their movement recorded by surveys and aerial photographs.

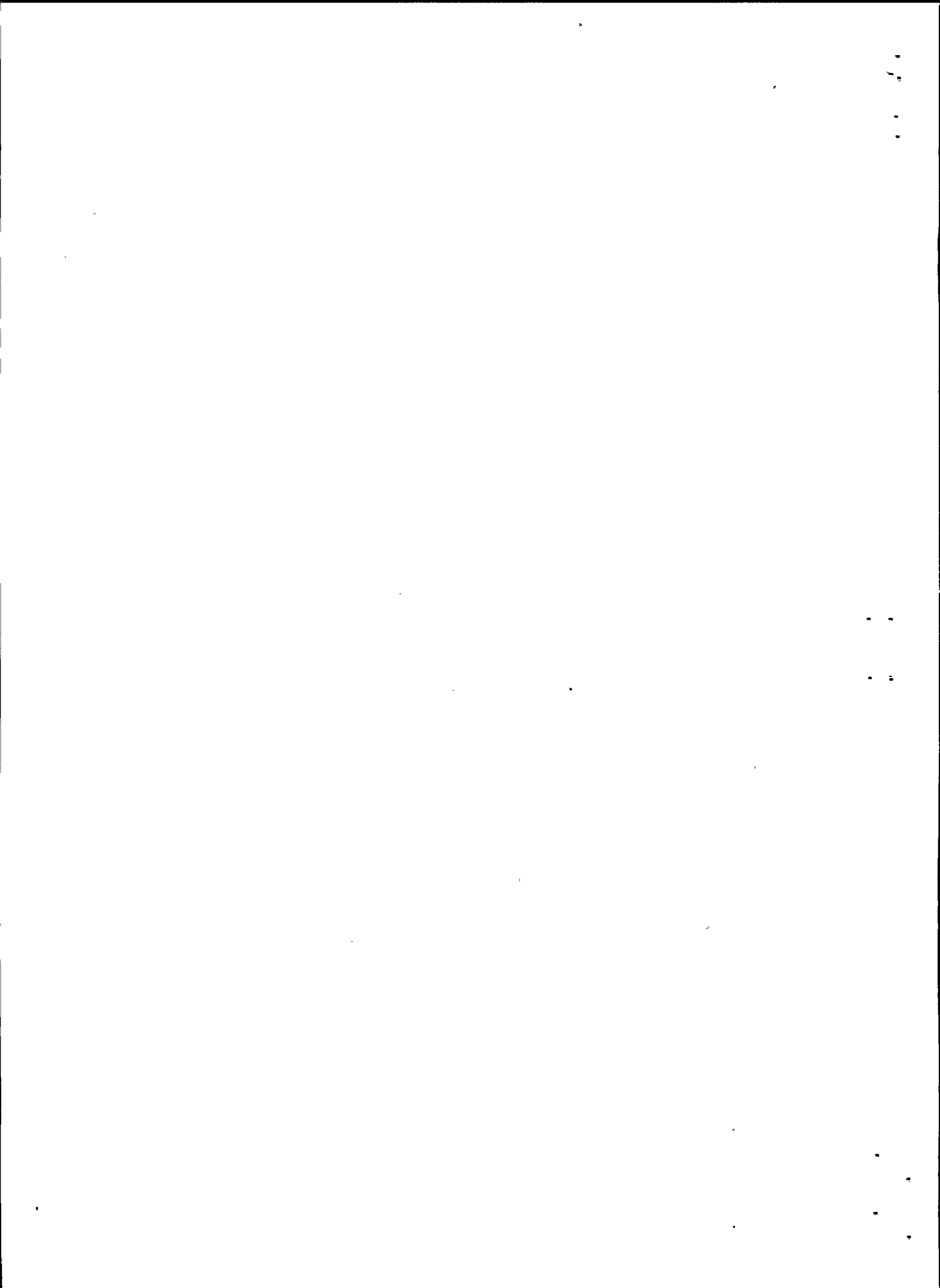
"In December 1967 storm waves from the north caused severe damage to the west breakwater trunk, dislodging the 8-ton armor stone and undermining the core material. Concrete grout which had previously been applied to the crest contributed to the failure by causing excessive back pressure which resulted in more rapid erosion of the core and did not permit the capstones to settle until large voids had developed underneath and large cap sections then failed. The trunk damage was repaired in 1969 using 19-ton tribars. A severe northern storm in November 1969 again damaged the west breakwater trunk, requiring the replacement of approximately 25 19-ton tribars which had been moved shoreward as much as 300 feet. The damage occurred in an area of one-layer tribar transition to the original 8-ton stone, and presumably resulted from insufficient interlocking. At this time the tribars and concrete crest ribs were extended landward 80 feet and the landward end of the tribar armor was buttressed by 25 35-ton tribar units."

\* \* \*

"Inspection of the breakwaters in 1973 showed considerable damage and settlement of the 33-ton tetrapod armor on the seaward quadrant of both breakwater heads. In addition, the 8-ton armor stone along the east and west breakwater trunks was failing and required repair. Dolosse concrete armor units were selected for the repair work due to their smaller size requirements for the design conditions than tetrapods or tribars, which results in cost savings and greater ease of handling."

Additional repair work was done in 1977. Sullivan states:

"Shortly after repair work was completed in 1977, the breakwaters were battered by large storm waves. The Kahului harbor master stated that they were the largest waves that he had witnessed in 10 years. The breakwaters sustained very little damage, and the



dolos appeared to hold their own against the storm wave attack. However, it is still too early to tell when and where the next repair will be required."

Some additional conclusions of Sullivan are:

"Grouting or capping the breakwater crests was found to cause excessive internal pressures resulting in rapid erosion of core material when armor stone is displaced. In addition, grouting prevents crest stones from settling and sealing the core until large amount of core material has been lost and large voids have developed underneath. The cast in place concrete ribs have functioned very effectively to prevent movement of the armor units across the breakwater crest, without increasing internal pressures.

"Considerable damage has been experienced at transition sections between different concrete armor units and on concrete armor units and stone. Care in interlocking and/or buttressing the transition sections is required for the units to achieve their maximum stability."

Nawiliwili Harbor, Kauai, Hawaii

Palmer (1960, p. 51) states:

"Although the Kahului and Hilo breakwaters were damaged by the storm of January, 1947, no damage was sustained by the Nawiliwili breakwater because the waves were from due north, a direction that is not a critical exposure for Nawiliwili Bay. However, waves generated by the storm of March, 1954 caused failure of the end of the structure and damage to the adjoining seaward slope. The storm of September, 1957, and Hurricane Nina further damaged the breakwater. In all, approximately 105 ft of the seaward end was knocked down and about 600 ft of the adjacent seaward slope severely damaged."

Gansbaai, South Africa

The following information on the breakwater at Gansbaai, South Africa, is from a report by Edge and Magoon (1979).

"Located 100 kilometers southeast of Cape Town, the breakwater is directly exposed to the prevailing westerly and south-westerly swells which approach over a shallow reef area. After passing the reef the waves focus into the harbor area. The old breakwater, a vertical wall caisson, was extended with a rubble structure. The rubble mound consists of:





1 layer	17.1-ton dolos
1 underlayer	12.4-ton dolos
1 underlayer and rubble core	4.5-ton dolos

The crest of the dolos is approximately 6.0m above low water datum. The breakwater was constructed on a relatively smooth, mild slope, rock bed with an average depth of 8.0m at the site.

"The first storm damage occurred in July 1970, with a maximum significant wave of 6.1m and a period of 11 to 17 seconds. The storm caused a hole in the dolos at the connection with the caisson. Numerous units were fractured. The same type of damage was repeated in July and December 1977 (See Figure [45]). The following comments by Retief (1978)\* gives his observations of the failure.

'It appears that the interaction of the focussed wave energy and the vertical caisson breakwater have caused a local concentrated attack on the rubble extension at the section indicated in the photographs.

'Model tests of the Gansbaai breakwaters have not simulated the prototype damage to the same extent and we can only conclude that the prototype failure was predominantly due to the fracture of the 17.1-ton units (0.3 stem ratio) and subsequent collapse of the profile.

'An interesting observation is that relatively little damage has been sustained on the remainder of the seawater slope over a period of eight years.

'Field experience at this site as well as model studies have shown that repair of a fractured dolos slope is relatively unsuccessful, unless, either the fractured units are first removed, or the repair layer overlaps the stable layers on either side of the damaged portion.

'An analysis of broken units has not produced any noteworthy fracture pattern. Model tests have, however, shown that these units do rock considerably under storm conditions and that unravelling at the toe can occur on the smooth bed rock.'

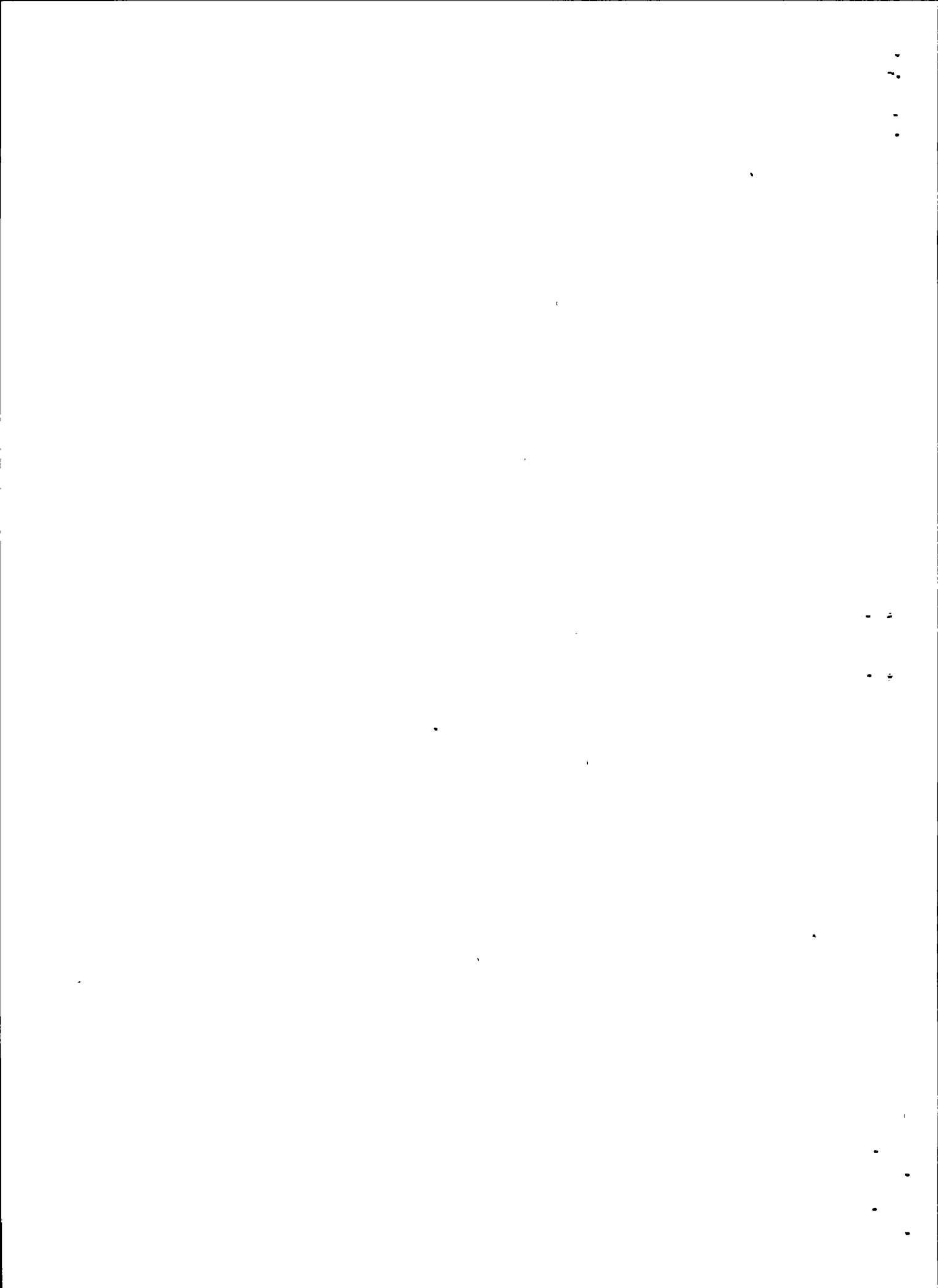




Figure 45. Photo of damage at Gansbaai breakwater after storm of December 1977 (From Edge and Magoon, 1979)



1 layer	17.1-ton dolos
1 underlayer	12.4-ton dolos
1 underlayer and rubble core	4.5-ton dolos

The crest of the dolos is approximately 6.0m above low water datum. The breakwater was constructed on a relatively smooth, mild slope, rock bed with an average depth of 8.0m at the site.

"The first storm damage occurred in July 1970, with a maximum significant wave of 6.1m and a period of 11 to 17 seconds. The storm caused a hole in the dolos at the connection with the caisson. Numerous units were fractured. The same type of damage was repeated in July and December 1977 (See Figure [45]). The following comments by Retief (1978)\* gives his observations of the failure.

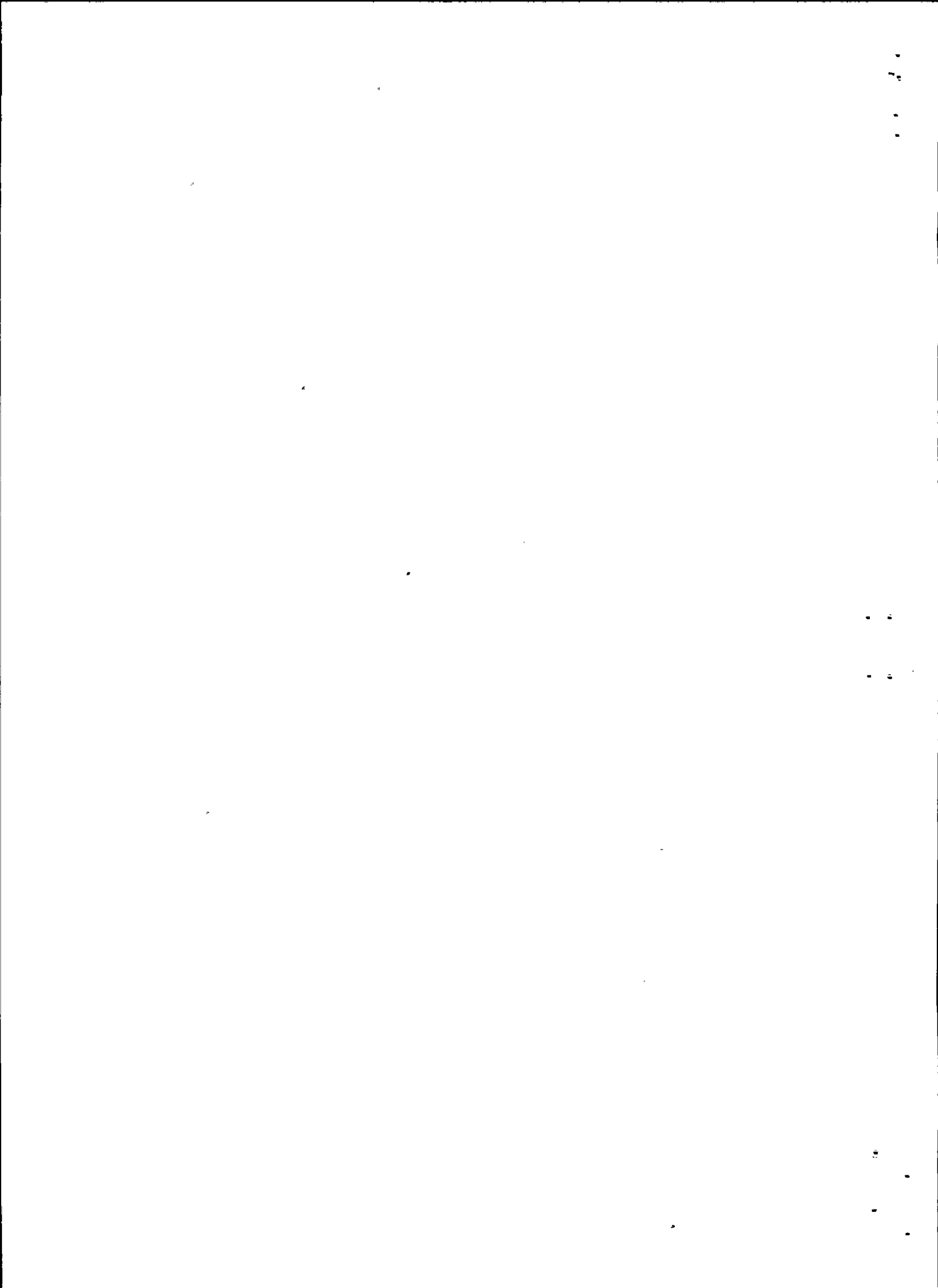
'It appears that the interaction of the focussed wave energy and the vertical caisson breakwater have caused a local concentrated attack on the rubble extension at the section indicated in the photographs.

'Model tests of the Gansbaai breakwaters have not simulated the prototype damage to the same extent and we can only conclude that the prototype failure was predominantly due to the fracture of the 17.1-ton units (0.3 stem ratio) and subsequent collapse of the profile.

'An interesting observation is that relatively little damage has been sustained on the remainder of the sea-water slope over a period of eight years.

'Field experience at this site as well as model studies have shown that repair of a fractured dolos slope is relatively unsuccessful, unless, either the fractured units are first removed, or the repair layer overlaps the stable layers on either side of the damaged portion.

'An analysis of broken units has not produced any noteworthy fracture pattern. Model tests have, however, shown that these units do rock considerably under storm conditions and that unravelling at the toe can occur on the smooth bed rock.'



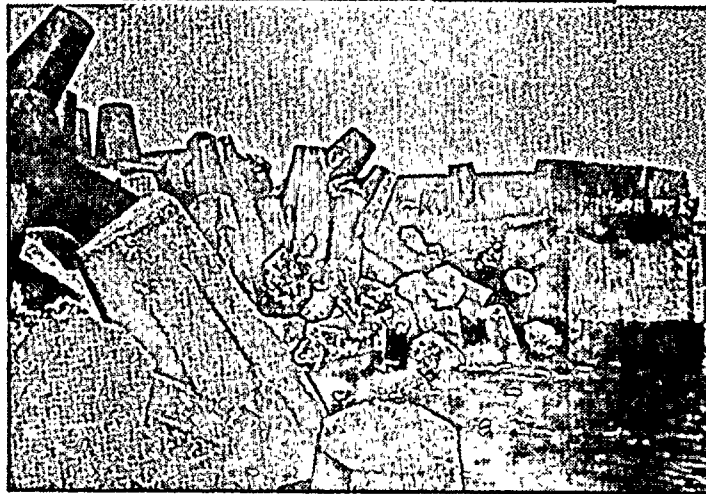
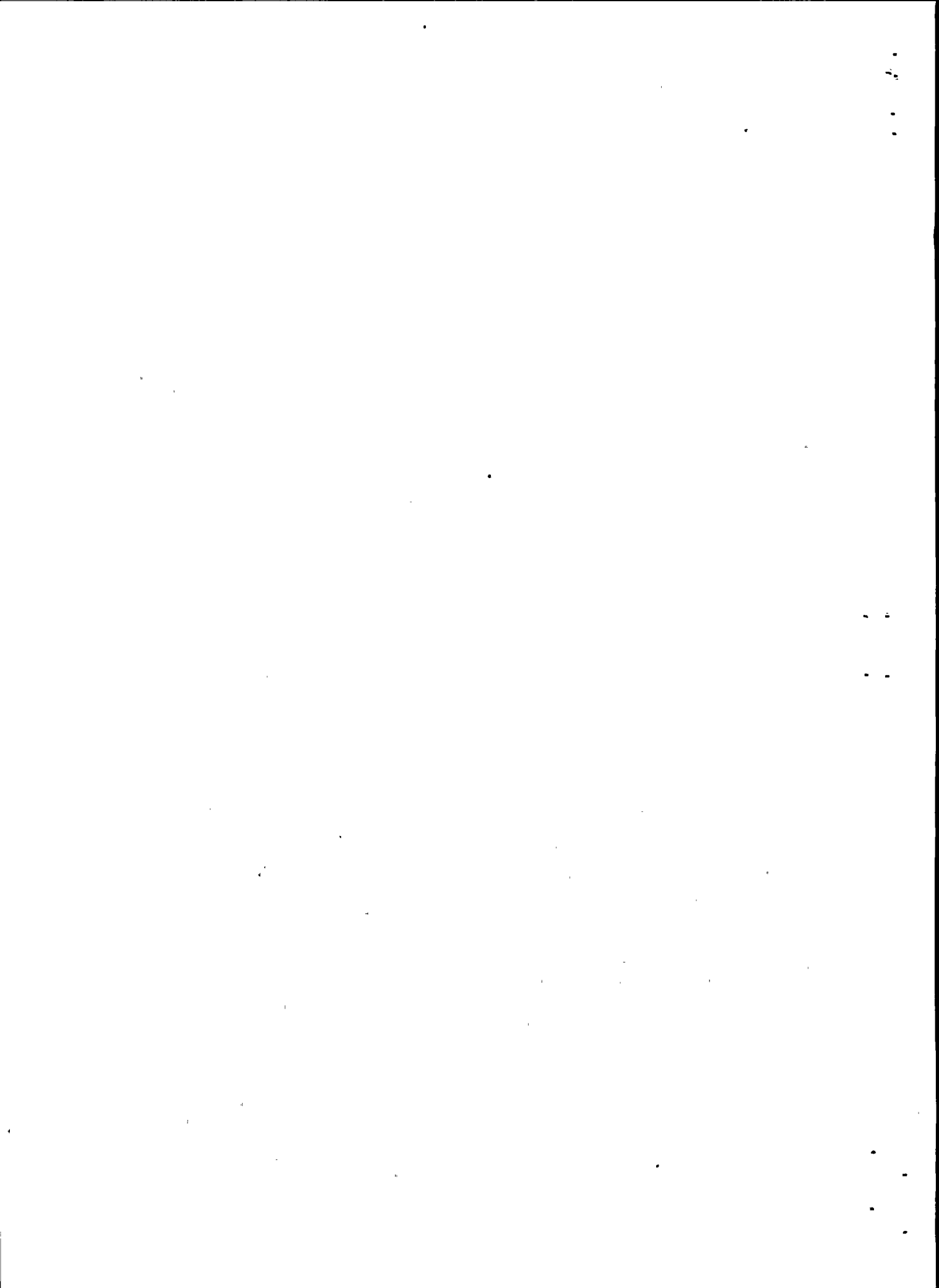


Figure 45. Photo of damage at Gansbaai breakwater after storm of December 1977 (From Edge and Magoon, 1979)





San Pedro Bay, California

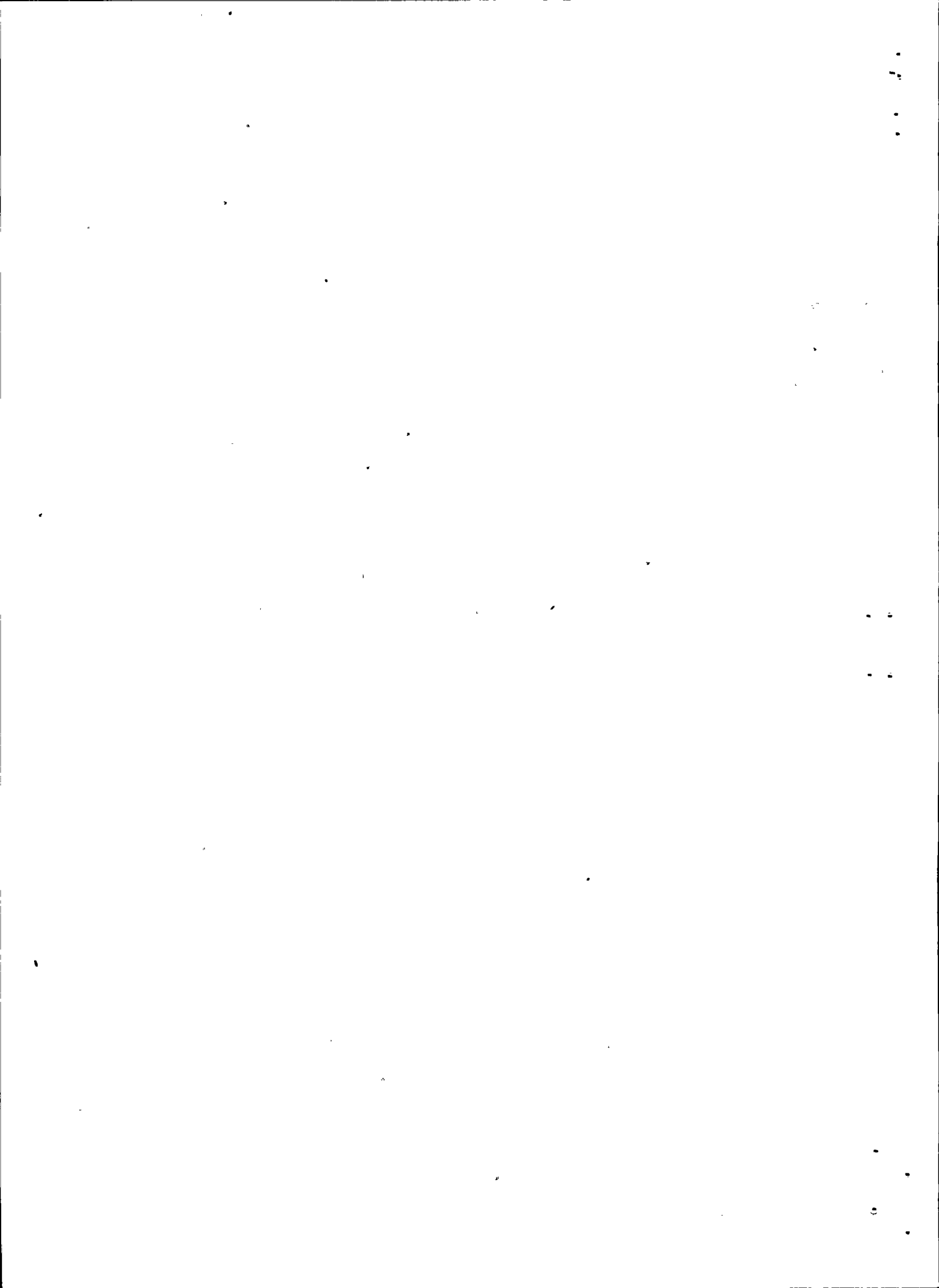
Hudson in his report "Reliability of Rubble-mound Breakwater Stability Models" (1975) compares prototype performance with the results of model studies for a number of breakwaters. Two of these examples were of breakwaters that had been damaged by storm waves. In regard to the San Pedro Bay, California, breakwater (see also, U. S. Army Waterways Experiment Station, 1953), Hudson states:

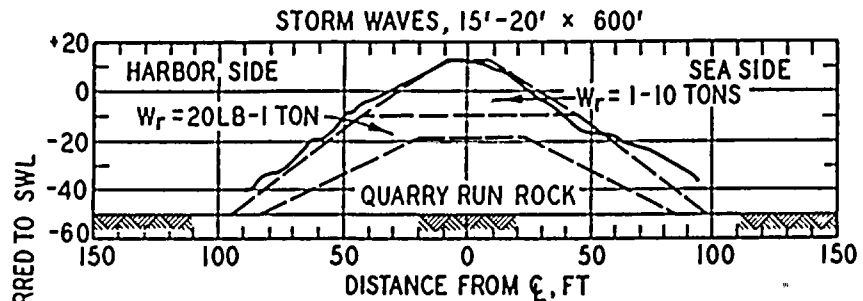
"The damage to a portion of the San Pedro Bay breakwater caused by storm waves in 1939 can be compared with the results of tests conducted at WES in 1947, since the San Pedro breakwater was very similar to those breakwater sections used in the scale-effect tests described in paragraphs 12 and 13 (Figure 2\*). Also, the wave heights that attacked the prototype structure were approximately the same as those used in the model tests. The major differences were that: (a) The model structures were subjected to waves of nearly constant heights of 15 and 21 ft for a time sufficient to obtain maximum damage (stabilized condition) for each test wave used, whereas the prototype damage was caused by storm waves of variable height and period, the heights of which were increscent to about 20 ft. Thus, it is not known if the prototype structure was stabilized or not. (b) The significant wave period of the prototype waves was increscent to about 15 sec, whereas the model wave period was constant at 8.5 sec.

"Comparisons of prototype and model damage are shown in Figures [46] and [47]. The prototype section with minimum damage is compared with the model section tested using 15-ft waves, and the prototype section with maximum damage is compared with the model section tested using 21-ft waves. It is seen that the disposition of the armor units and the final shape of the damaged prototype structure are similar to that obtained in the model tests. However, the amount of breakwater material displaced was greater in the model than in the prototype. With the data available, it was impossible to make the comparisons, model to prototype, for identical wave conditions. However, the similarity of damages obtained is believed sufficient to show that model test results, if the model is properly designed, constructed, and operated and if the prototype structure is constructed correctly, can be relied upon to provide the basis for the design and construction of safe and efficient rubble-mound structures."

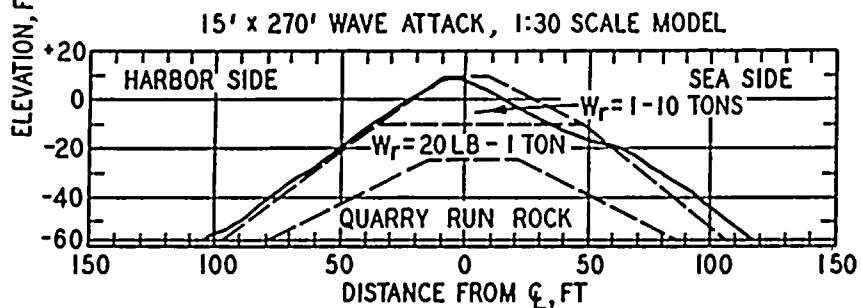
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\* In original report.



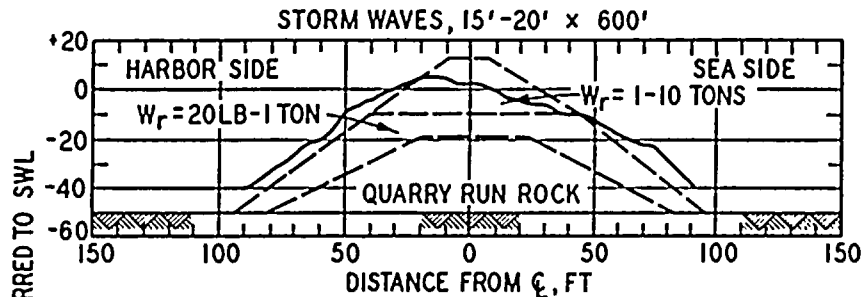


a. MINIMUM STORM DAMAGE TO BREAKWATER  
SAN PEDRO BAY, CALIF.

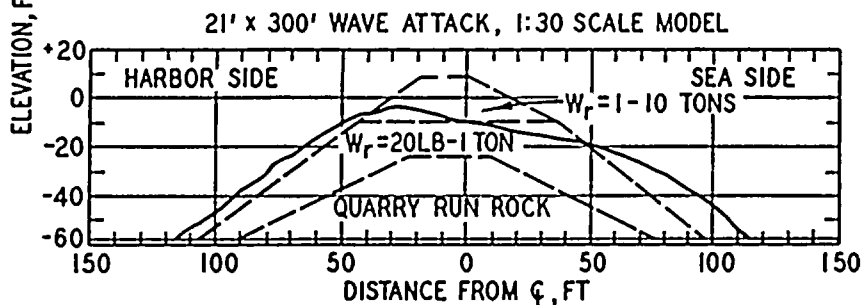


b. DAMAGE TO STABILIZED MODEL BREAKWATER

Figure 46. Comparison of damage to prototype  
and model breakwaters: 15-ft waves  
(From Hudson, 1975)

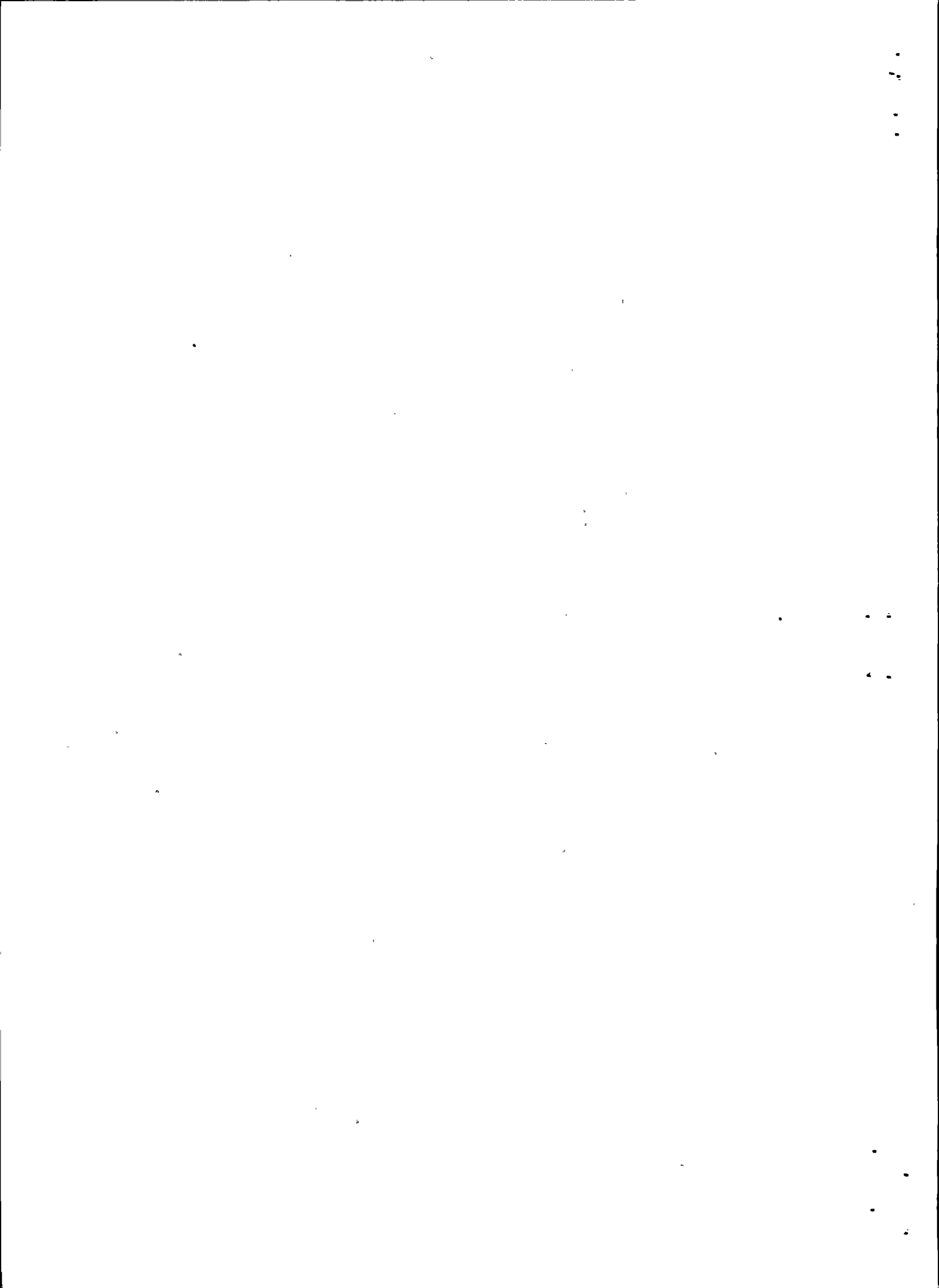


a. MAXIMUM STORM DAMAGE TO BREAKWATER  
SAN PEDRO BAY, CALIF.



b. DAMAGE TO STABILIZED MODEL BREAKWATER

Figure 47. Comparison of damage to prototype  
and model breakwaters: 21-ft waves  
(From Hudson, 1975)



### Silver Bay, Minnesota

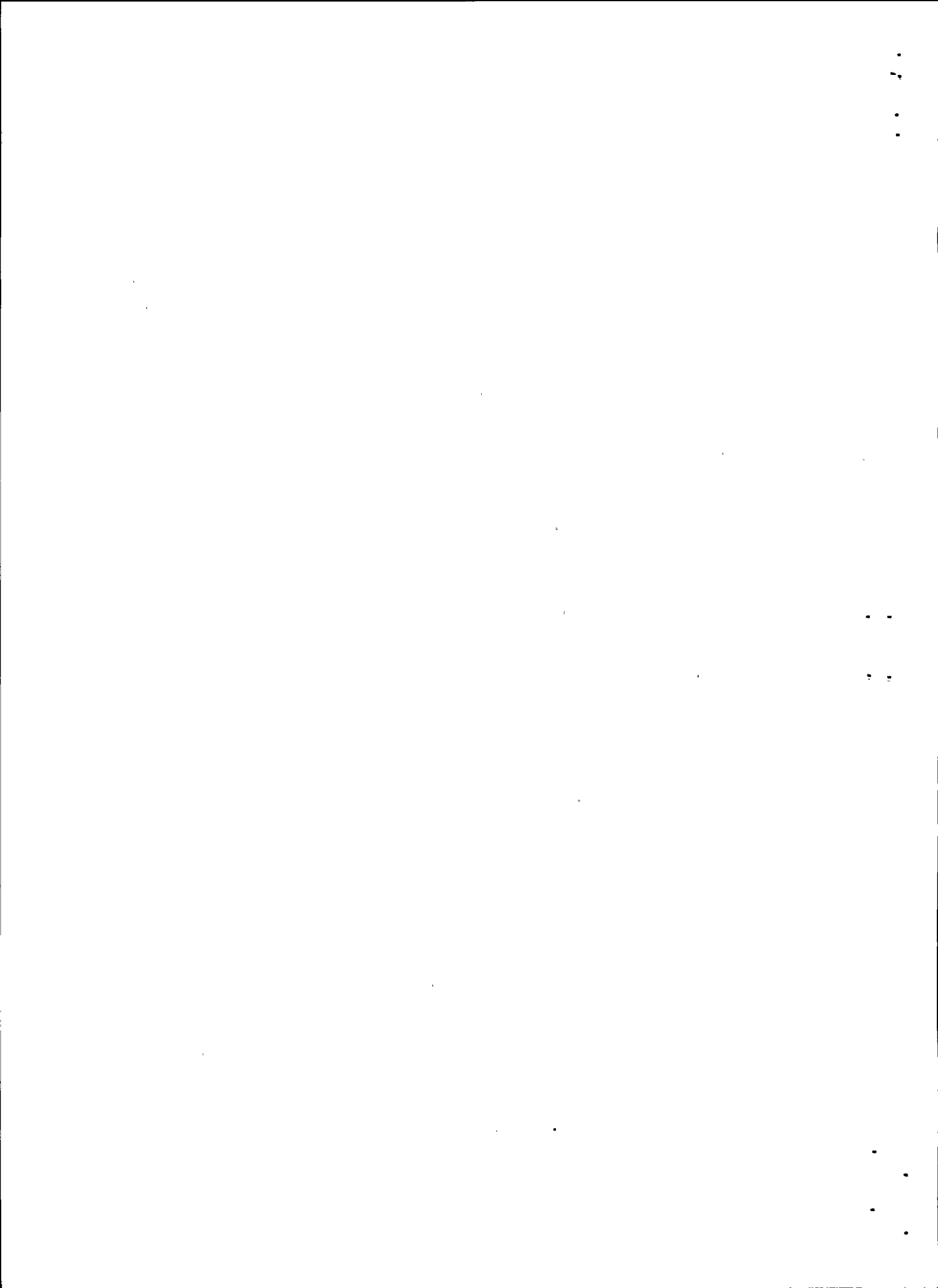
Tests were made using a hydraulic model of the rubble-mound breakwater located at Lake Superior at Silver Bay, Minnesota (Hudson, 1975). The breakwater was damaged by two storms. The first of these was on 10 May 1953 which occurred while the breakwater was still under construction, before the armor units had been placed. This will not be considered herein. The second storm occurred after completion of the breakwater, on 20 November 1953. The armor units were stones weighing from 10 to 14 tons, with a specific weight of 175 lbs/ft<sup>3</sup>. The design wave height was 14 feet. A typical cross section of the breakwater is shown in a photograph of the model test result in Figure 48a. A copy of a photo of the actual damage is shown in Figure 48b.

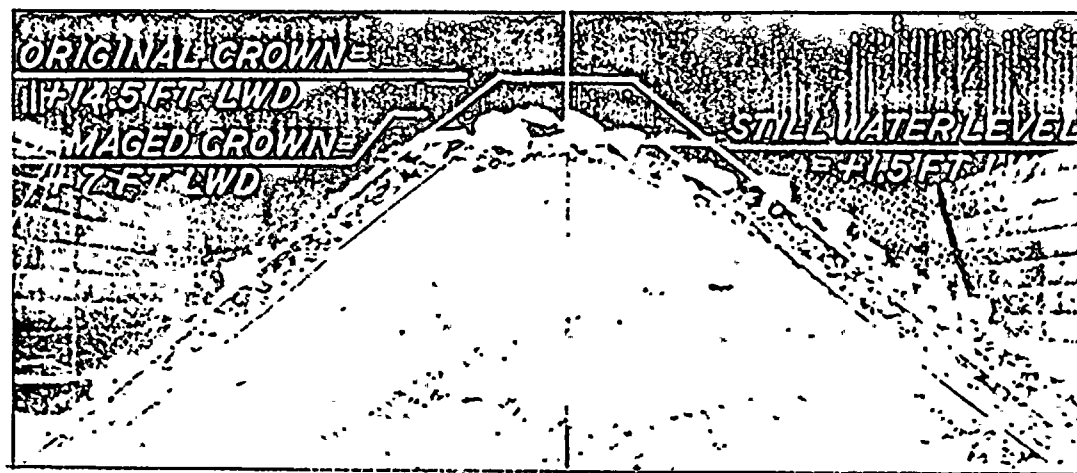
### Sandy Bay, Cape Ann, Massachusetts

The portion of the rubble-mound detached breakwater in Sandy Bay at Cape Ann, Massachusetts, that was constructed was completed in 1916. The original plan of a long dog-legged breakwater never was finished, with much of the substructure built up to about mean low water, MLW, with no superstructure built on it. The substructure of the southern arm (3600 feet) and about 2500 feet of the substructure of the western arm were built, but only a little over 900 feet of the superstructure was completed. In the report "Features of Various Offshore Structures" (Peraino, Chase, Plodowski and Amy, 1975), some information on the present state of this breakwater is given. However, this information is only about the superstructure.

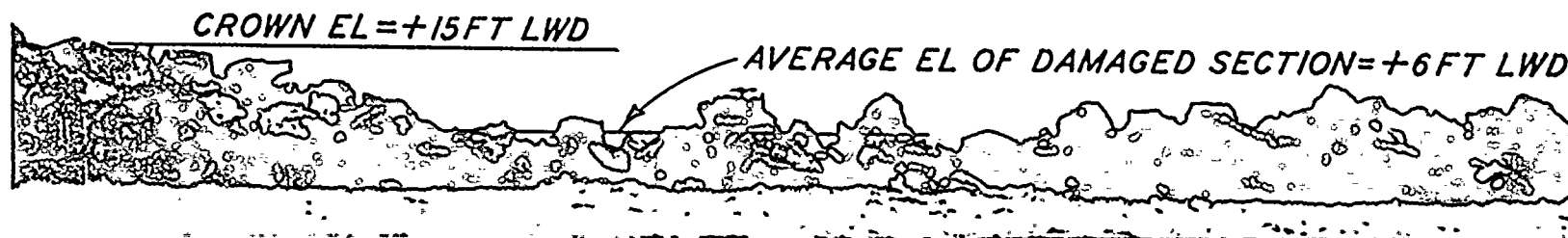
### Wick Bay, Scotland

The following quotation from Thos. Stevenson regarding damage that occurred in 1872 to the breakwater in Wick Bay, Scotland, is from Gaillard (1904):



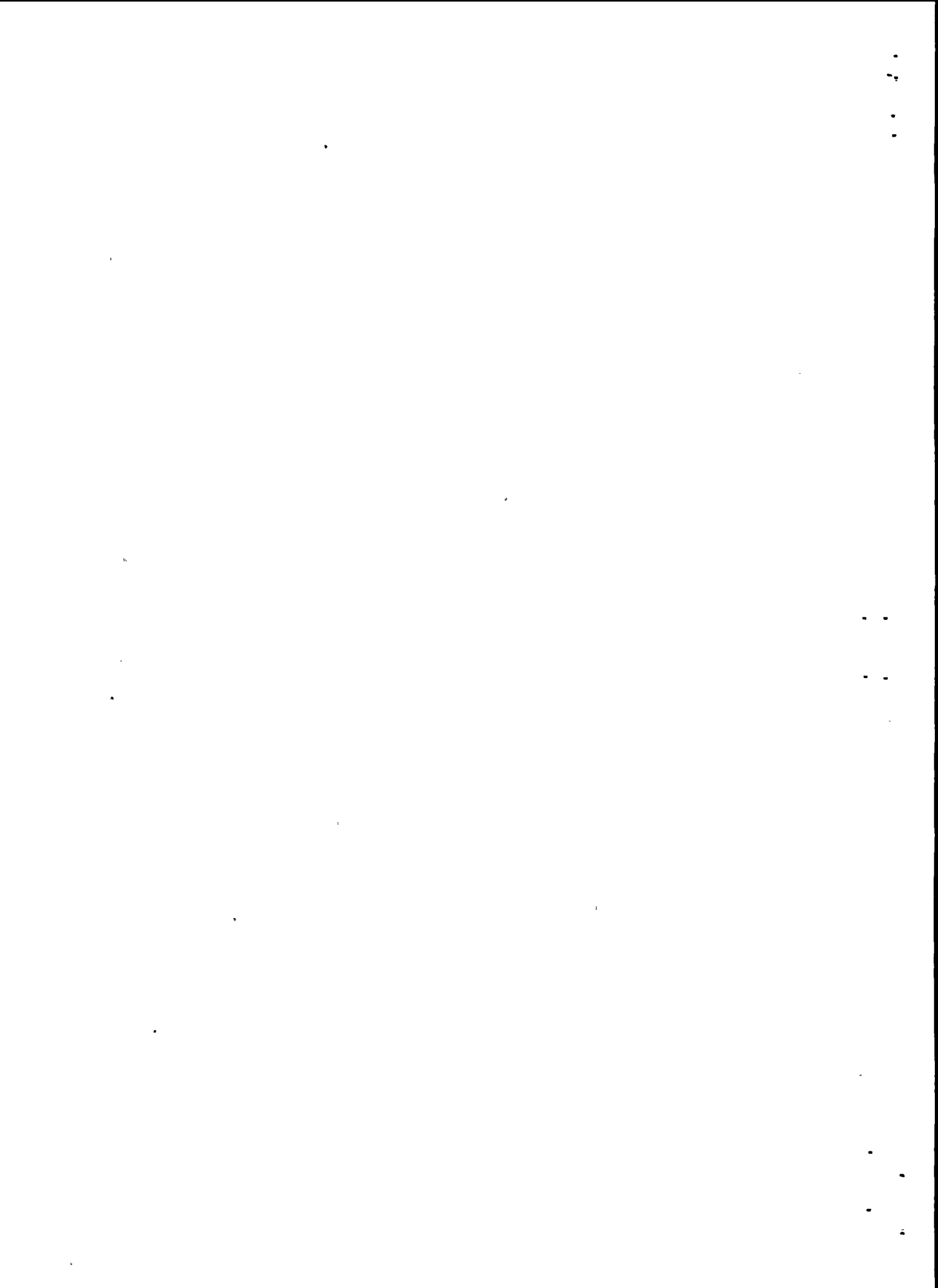


d. 1:30-SCALE MODEL WAVE ATTACK  
DURATION = 16 HR, H = 14 FT



b. PROTOTYPE WAVE ATTACK, 20 NOV 1953  
DURATION = 11 HR  
WAVES INCRESCENT TO 15 FT IN HEIGHT

Figure 48. Comparison of wave attack in prototype and corresponding model  
on breakwater at Silver Bay Harbor, Minnesota  
(From Hudson, 1975)



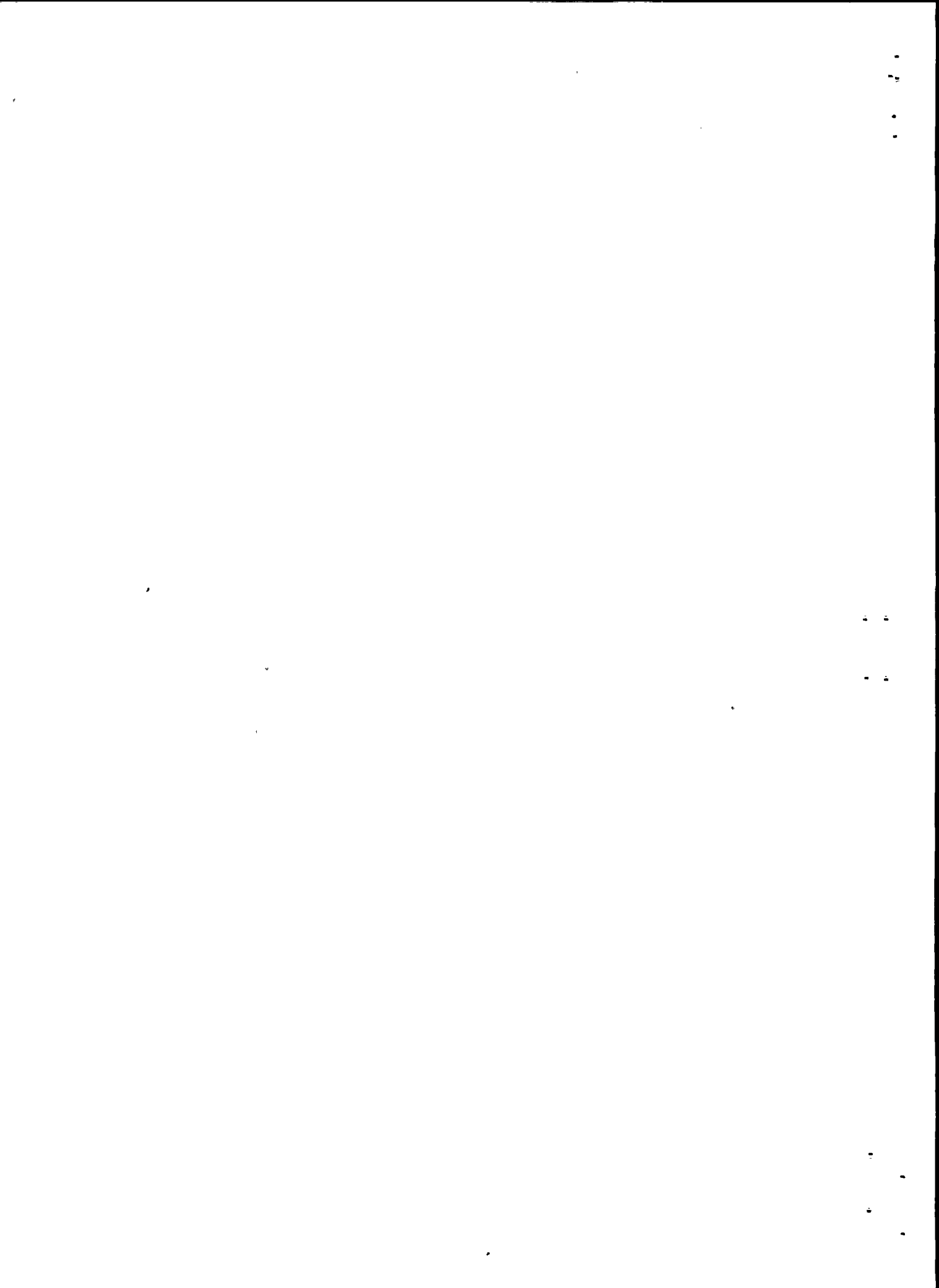


"The end of the work, as has been explained, was protected by a mass of cement rubble work. It was composed of three courses of large blocks of 80 to 100 tons, which were deposited as a foundation in a trench made in the rubble. Above this foundation there were three courses of large stones carefully set in cement, and the whole was surmounted by a large monolith of cement rubble, measuring about 26 by 45 feet by 11 feet in thickness, weighing upward of 800 tons. The block was built *in situ*. As a further precaution, iron rods 3.5 inches in diameter were fixed in the uppermost of the foundation courses of cement rubble. These rods were carried through the courses of stonework by holes cut in the stone, and were finally embedded in the monolithic mass, which formed the upper portion of the pier. (See Pl. IV.)

"Incredible as it may seem, this huge mass succumbed to the force of the waves, and Mr. DeDonald, the resident engineer, actually saw it from the adjacent cliff being gradually "slewed" round by successive strokes until it was finally removed and deposited inside the pier. It was not for some days after that any examination could be made of this singular phenomenon, but the result of the examination only gave rise to increased amazement at the feat which the waves had actually achieved. It was found on examination by diving that the 800-ton monolith forming the upper portion of the pier, which the resident engineer had seen in the act of being washed away, had carried with it the whole of the lower courses, which were attached to it by the iron bolts, and that this enormous mass, weighing not less than 1,350 tons, had been removed *en masse* and was resting entire on the rubble at the side of the pier, having sustained no damage but a slight fracture at the edges. A further examination also disclosed the fact that the lower or foundation course of 80-ton blocks, which were laid on the rubble, retained its position unmoved. The second course of cement blocks, on which the 1,350-ton mass rested, had been swept off after being relieved of the superincumbent weight, and some of the blocks were found entire near the head of the breakwater. The removal of this protection left the end of the work open, and the storm, which continued to rage for some days after the destruction of the cement rubble defense, carried away about 150 feet of the masonry (one-seventh of the whole), which had been built solid and set in cement. The same remarkable feature of former damage was strikingly apparent in the last damage,—*the foundations, even to the outer extremity of the work, remaining uninjured.*"

#### Port Sines, Portugal

The rubble mound breakwater at Port Sines, Portugal, is the largest of its kind in such an exposed environment. The breakwater is situated in



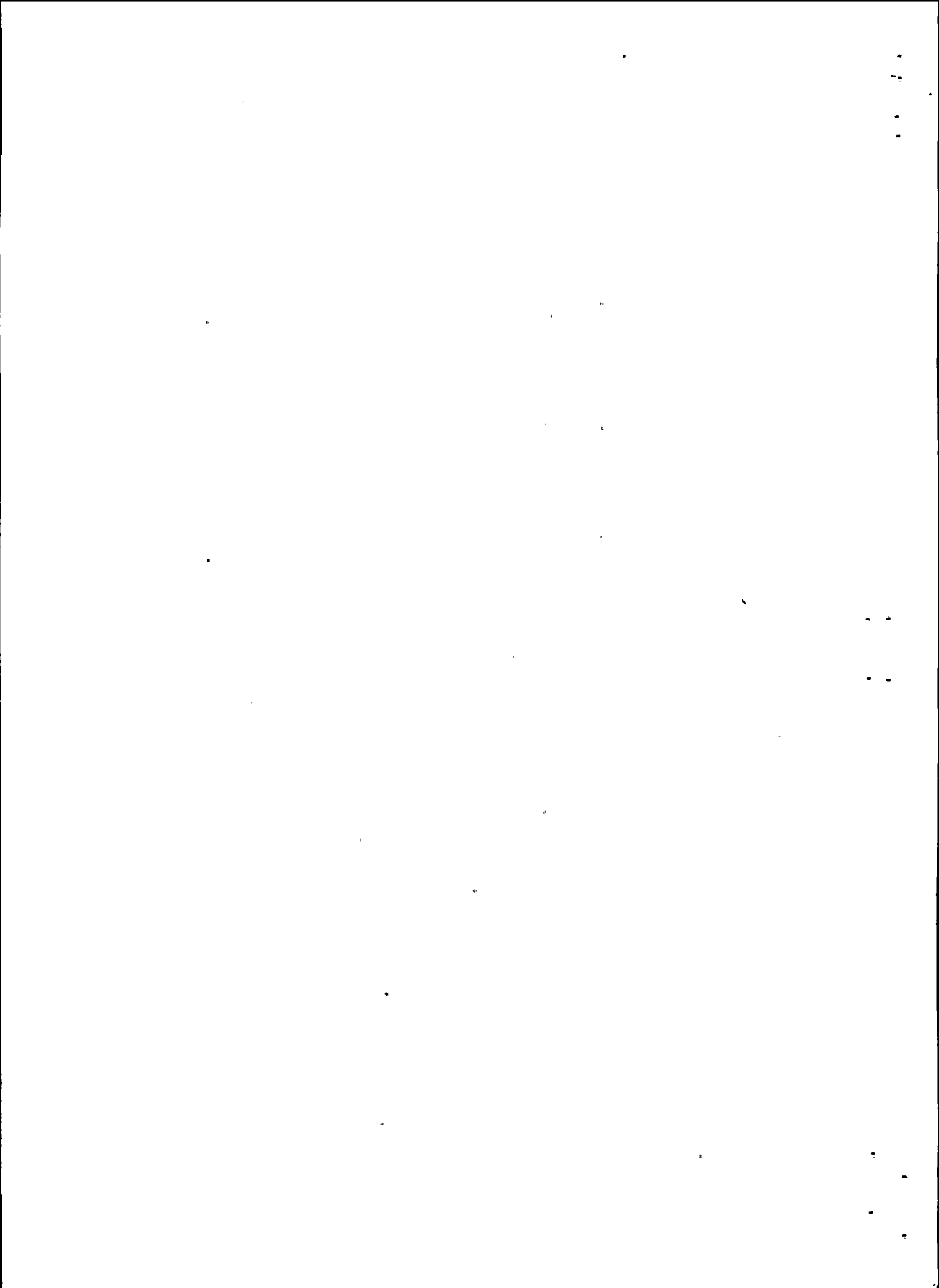
rather deep water (about 50 meters at the seaward terminus) and is exposed to the full force of North Atlantic storms. The 100-year return period significant wave height has been estimated to be 11 meters (Port Sines Investigating Panel, 1981). Construction of the breakwater was nearly completed when severe damage was caused by the storm waves of 26 February 1978, 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 metric 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.

A typical cross section of the breakwater is shown in Figure 49, with an example of the type of profile degradation being shown in Figure 50 (Port Sines Investigating Panel, 1981). A very large number of the cast concrete (not reinforced) dolos units were broken. The Port Sines Investigating Panel (1981, p. 53) give the following information:

"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 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

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\*These refer to the original report.



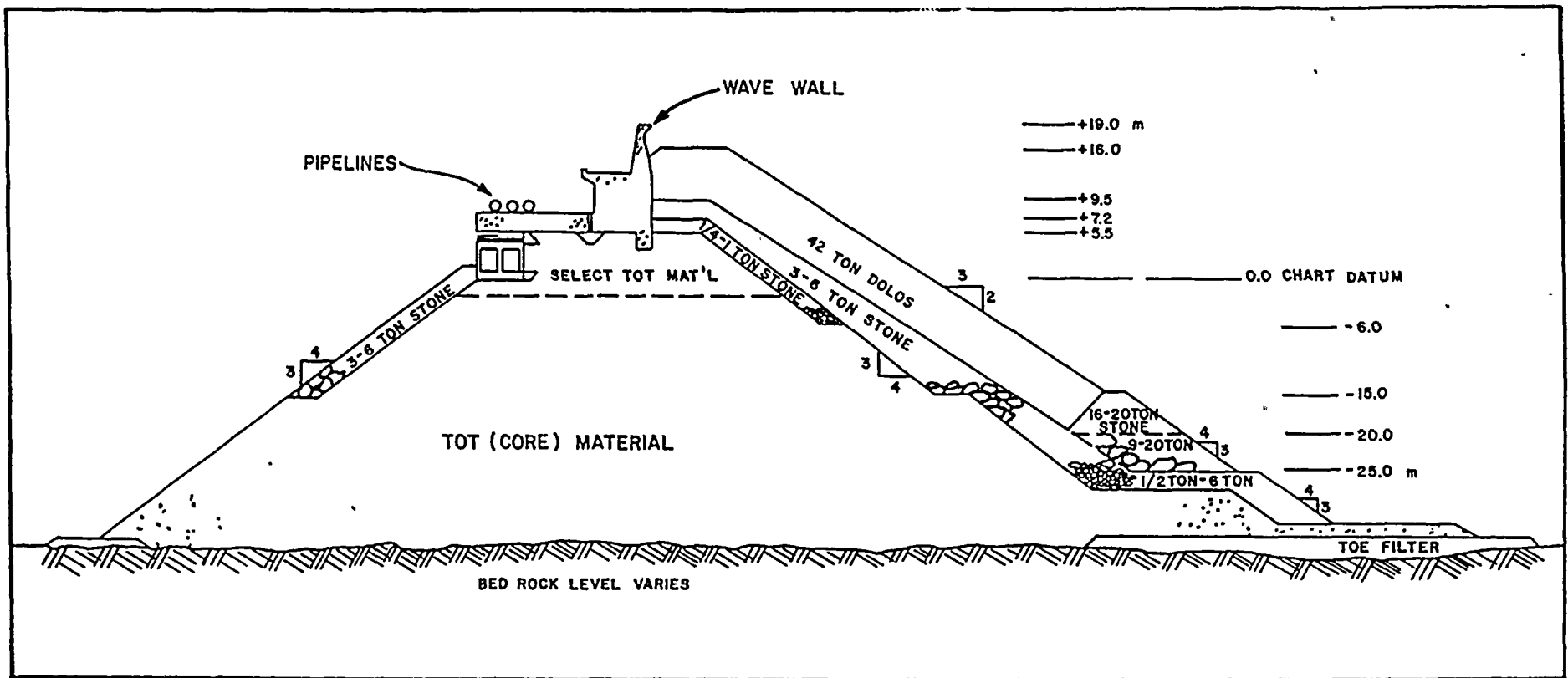
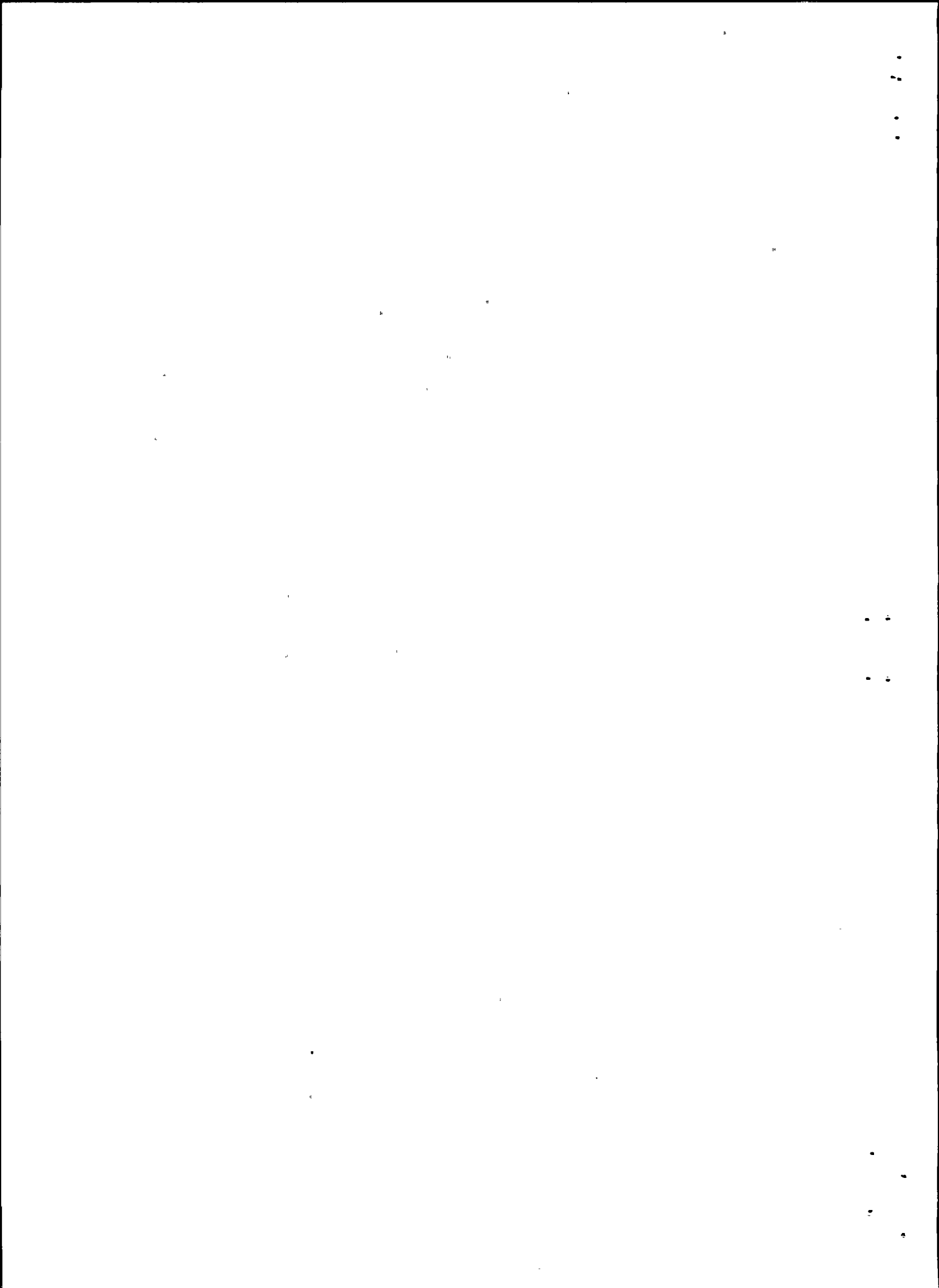


FIGURE 49. FINAL DESIGN CROSS-SECTION OF BREAKWATER  
 (From Port Sines Investigating Panel, 1981)



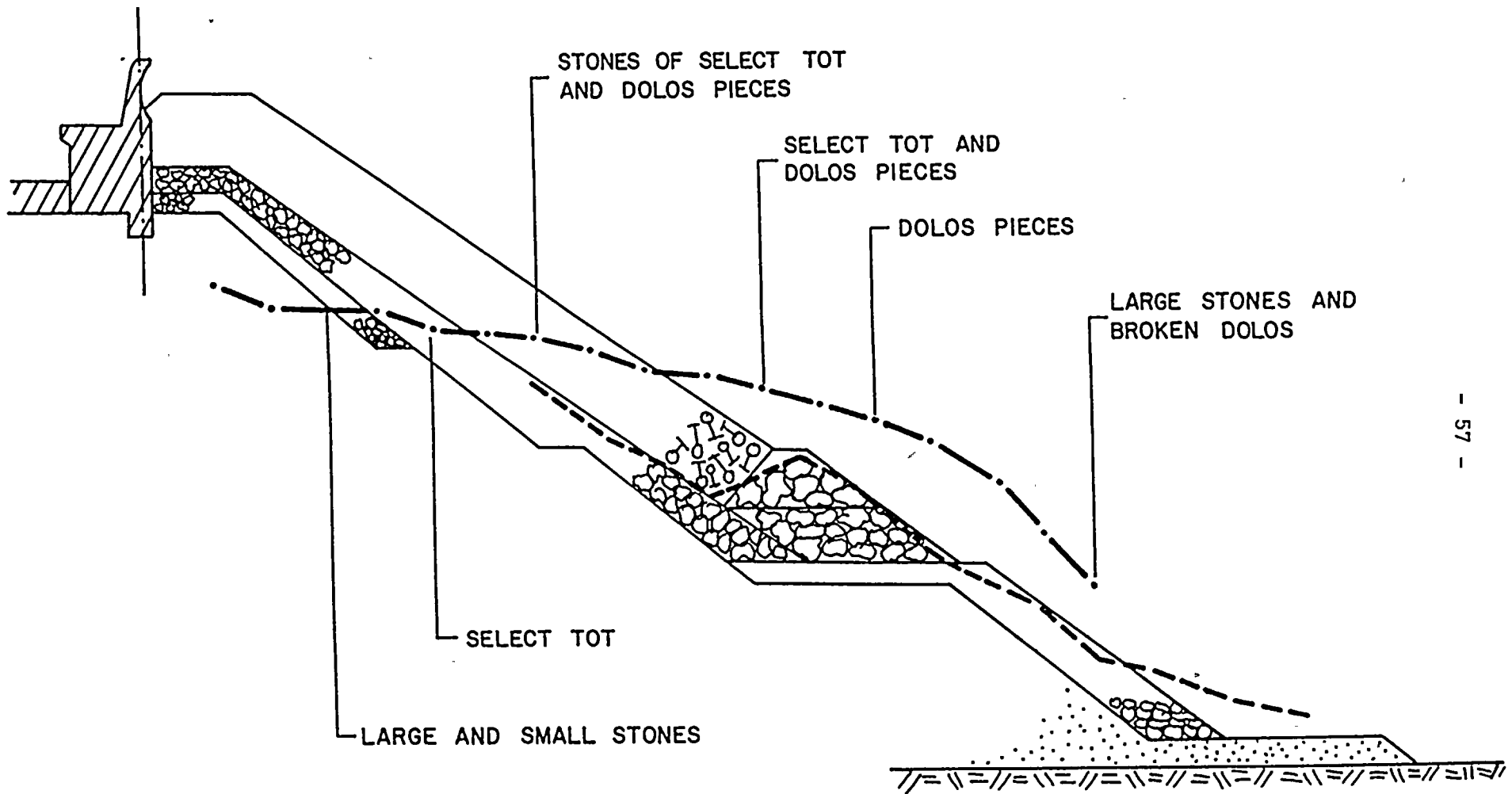
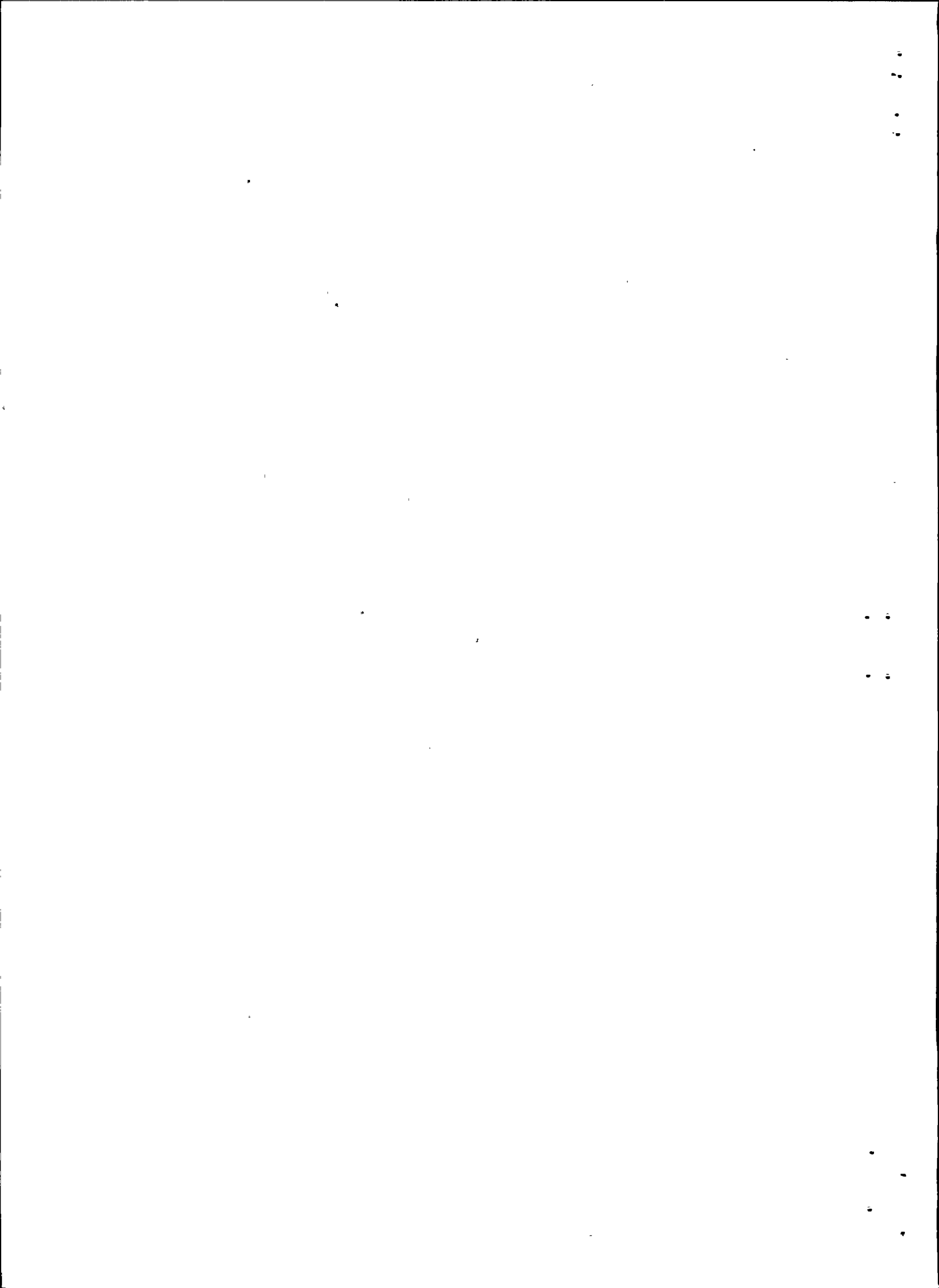


FIGURE 50. A TYPICAL CROSS-SECTION AS OBSERVED BY DIVERS  
 (From Port Sines Investigating Panel, 1981)





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 movement of the broken dolos parts."

Much useful information on the Port Sines breakwater has been presented by Zwamborn (1979). In part, he states:

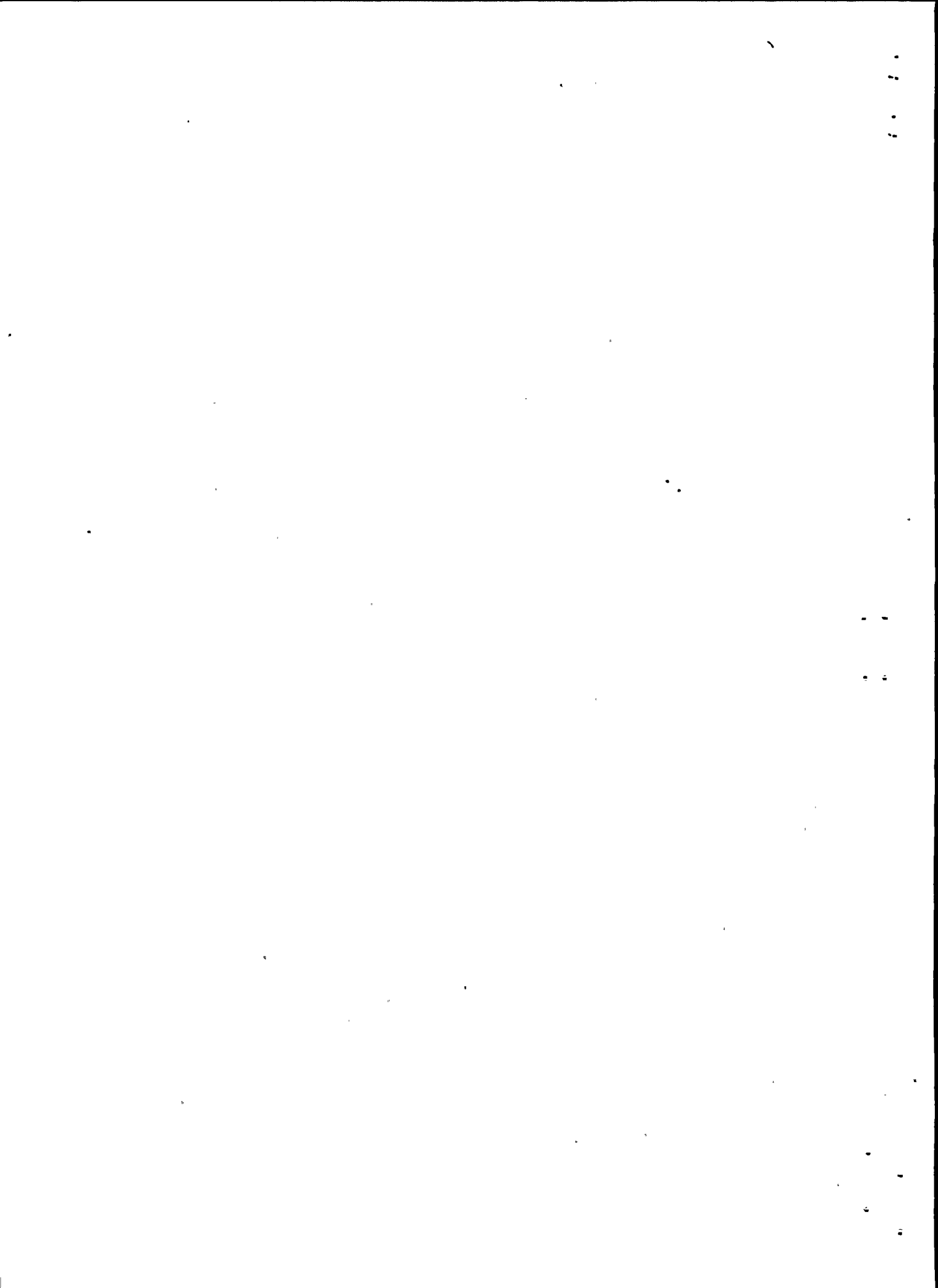
"After the storm, a survey was made of the damage, both above water (levels on wave wall, lower roadway and caisson tops, Fig. [51b]) and underwater (profiles, mainly of the damage areas, Fig. [51a]). The wave wall was seen to have collapsed completely over a distance of about 150m near Berth no. 2 and over a distance of 300m near Berth no. 1. In these sections no dolos armour was visible; in fact, near Berth no. 2, the dolos armour had disappeared over a total length of 250m (Fig. [53a]). In addition, there were two smaller areas between Berths nos. 2 and 3 where the dolosse had disappeared although there was no damage to the superstructure (Fig. [51a]).

"The underwater profiles in the damage areas are remarkably similar. Mean profiles of the four failure areas, indicated in Fig. [51a], are shown in Fig. [52]. They all show an approximate slope of 1 in 5 to a depth of about -10m CD and 3 in 4, the original slope, below this level. Diver reports and underwater photography prove that virtually all dolosse in these areas were broken and the pieces mixed with underlayer and filter stone.

"The sections of the breakwater armour which did not fail show above water level, areas which suffered very little damage alternated by areas which were seriously damaged (Fig. [53b]). The section between chainages 1400 and 1500 (sections 22 to 24) shows a considerable loss of dolosse above water while the underwater profile indicates a buildup of material right down the slope (Fig. [52]).

"Although visually the breakwater head shows very little damage, inspection of the underwater parts indicates a large amount of dolos breakage on and just below the water line (no detailed diver survey was available for this area)."

The formal conclusions of the Official Portuguese Investigating Team is of interest. Some of their conclusions, translated into English, are given in the report of the Port Sines Investigating Panel (1981, p. 72) as follows:



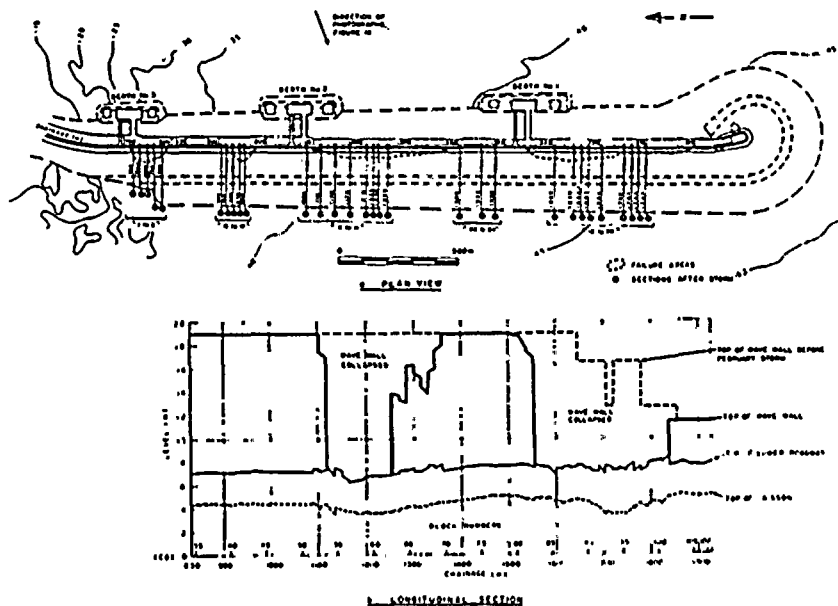


Figure 51 Plan and longitudinal section of main breakwater after February 1978 storm (From Zwamborn, 1979)

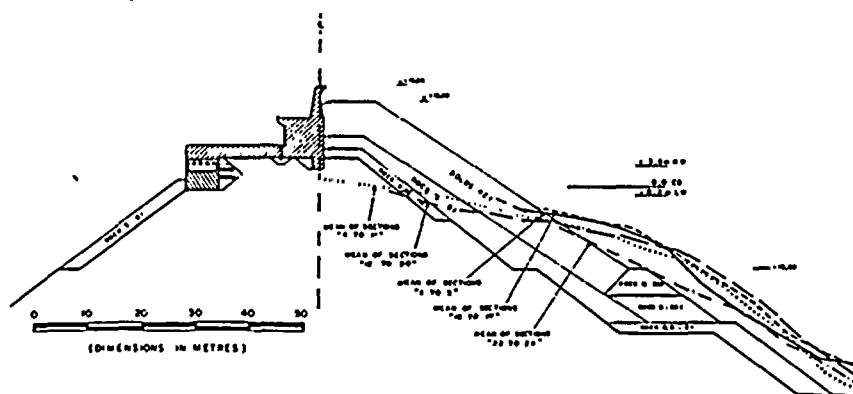
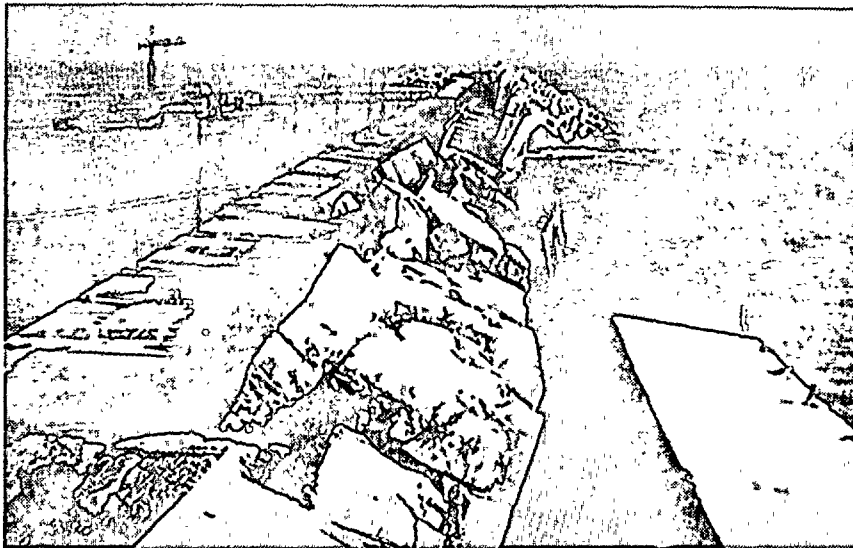
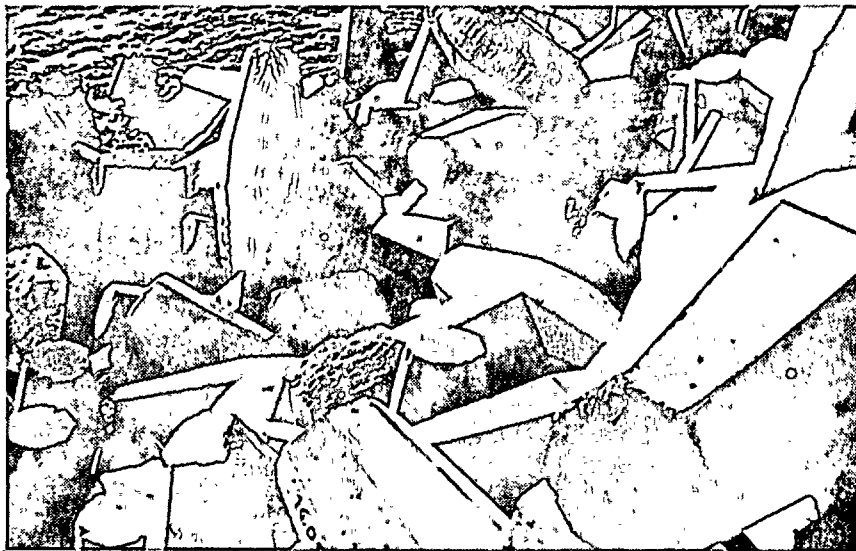


Figure 52 Mean profiles after February 1978 storm (From Zwamborn, 1979)





a) Main damage area between Berth No's 2 and 1, 12/6/78



b) Damage to Dolos Armour between Berth No's 2 and 3 12/6/78



" 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."

The writer visited the breakwater at Port Sines, Portugal, on 11 August 1981. During this visit he learned that even more severe storm waves struck the breakwater during February 1979. Quite a bit of additional damage occurred. It was not possible to see all of the damage, as much repair work had been done by August 1981, using cast concrete modified cubes (weighing more than 90 metric tones, each). Of considerable importance is the fact that the writer was able to walk to the end of the breakwater. This end section, several hundred feet in length, has had no repair work done on it, but is still several feet above the low water level.

Detailed physical model studies have been made and others are presently being made as a major portion of the engineering planning of the rehabilitation of this breakwater.

#### Columbia River South Jetty, Oregon

The jetties at the entrance to the Columbia River have been in existence for many years, and are fully exposed to the great storms of the North Pacific Ocean. According to a recent information booklet (U. S. Army Engineer Division, North Pacific, 1979):

"The first structural improvement in the interest of navigation was the initiation of the south jetty in 1885. The Federal navigation project at the mouth of Columbia River was first authorized by the River and Harbor Act of 3 March 1905 and was later modified by the Act of 3 September 1954."



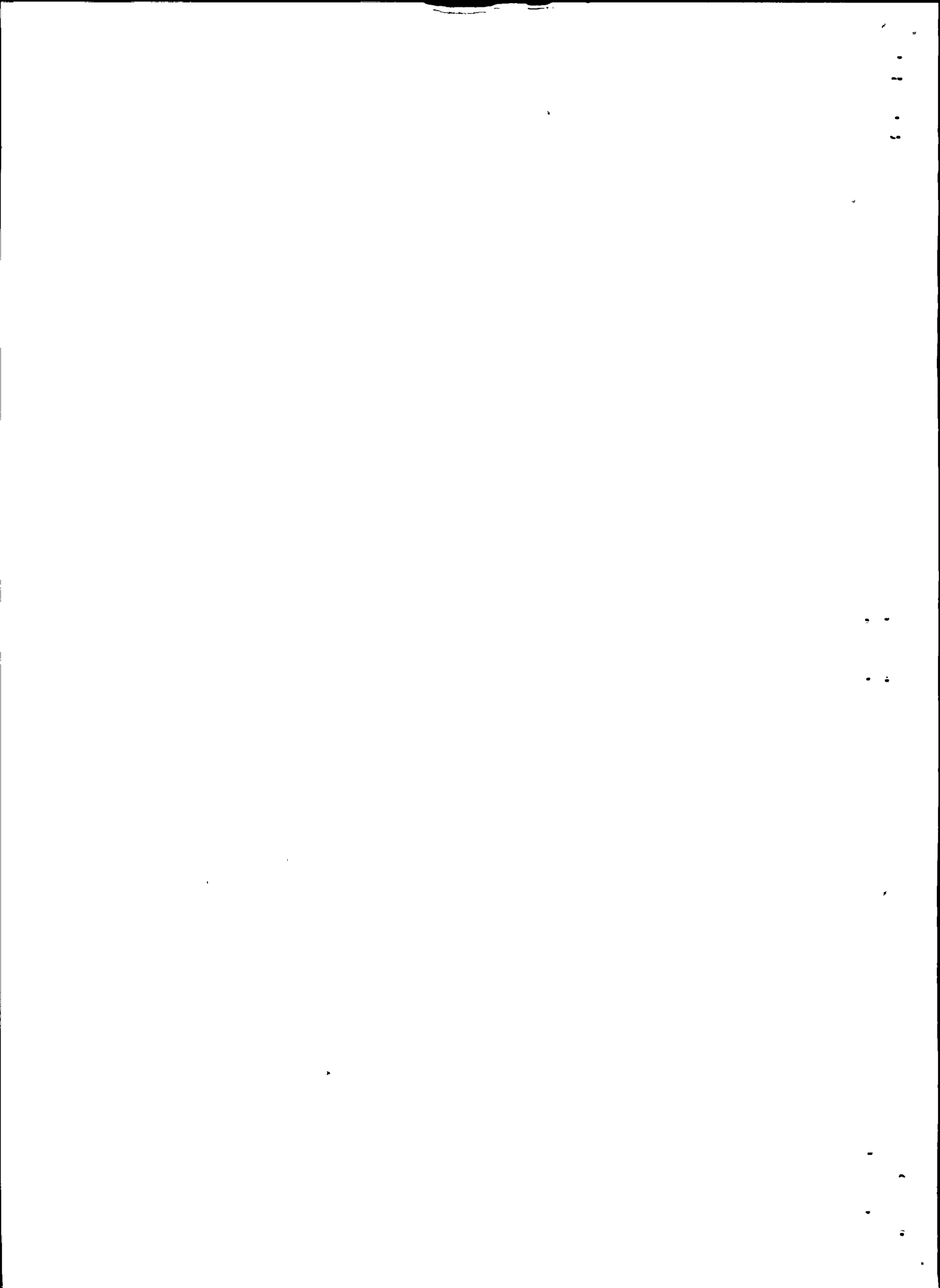


"The south jetty, as indicated earlier, was initiated in 1885 and the first stage was completed in 1896. Reconstruction and extension of the jetty began in 1903 and was completed in 1913. In 1941, a concrete terminal block, 170 feet long, was constructed. In 1961 to 1963 about 11,500 feet of the structure was repaired, extending to a point approximately 1,700 feet short of the seaward end of the terminal block. The repaired portion of the jetty is in good shape but about 1 mile of the authorized length was not repaired and is below the low water level."

Some details of the jetty are shown in Figure 54, which is from the publication cited above. Additional information is available in a paper by Hickson and Rodolf (1950). They state:

"The extension of the south jetty was started in 1903 and completed in 1913. The jetty was constructed of rubble stone, and, because the exposure was so great and the incessant wave action prevented repair operations by floating plant, it was necessary to delay maintenance until the amount of work required would justify the cost of the necessary trestle and plant. No maintenance was done on the jetty until the fall of 1931, and by that time the sea had flattened the enrockment to about low-water level and spread out the stone so that the width of the outer 2-3/4 miles was about 200 ft. at low-water level. Under three contracts, 2,200,000 tons of stone were placed in the superstructure, which was carried out to within approximately 3,300 ft. of the outer end of the jetty as completed in 1913. This was believed to be the limit of the superstructure required. The reconstruction of the south jetty was completed in 1936, but the action of the waves across the end face of the new work started piecemeal disintegration of the outer end. During a normal winter season the superstructure would ravel back 300 ft. or more. The outer end was impregnated with 12,737 tons of hot mixture of asphaltic mastic (85 percent sand and 15 percent asphalt) in an attempt to prevent raveling. While computations and later observations indicate that the asphaltic mix completely filled the voids to about low-water level (26 ft.), it did not prevent breakdown of the end, and raveling still continued. A solid concrete terminal was then constructed above low-water level and has proven effective. The concrete terminal is about 3,900 ft. shoreward from the end of the original jetty as completed in 1913.

"The south jetty as now constructed is a massive structure (see Figs. [55], [56], and [57]). The top width varies from 45 ft. to 70 ft., with an elevation of 26 ft. above mean lower low water. The sea slope is approximately 1 on 1-1/2 consisting of stone weighing up to about 25 tons, with 45 percent of entire enrockment having an average weight of 10 tons to the piece. The base width of the outer portion is approximately 350 ft. and the total height



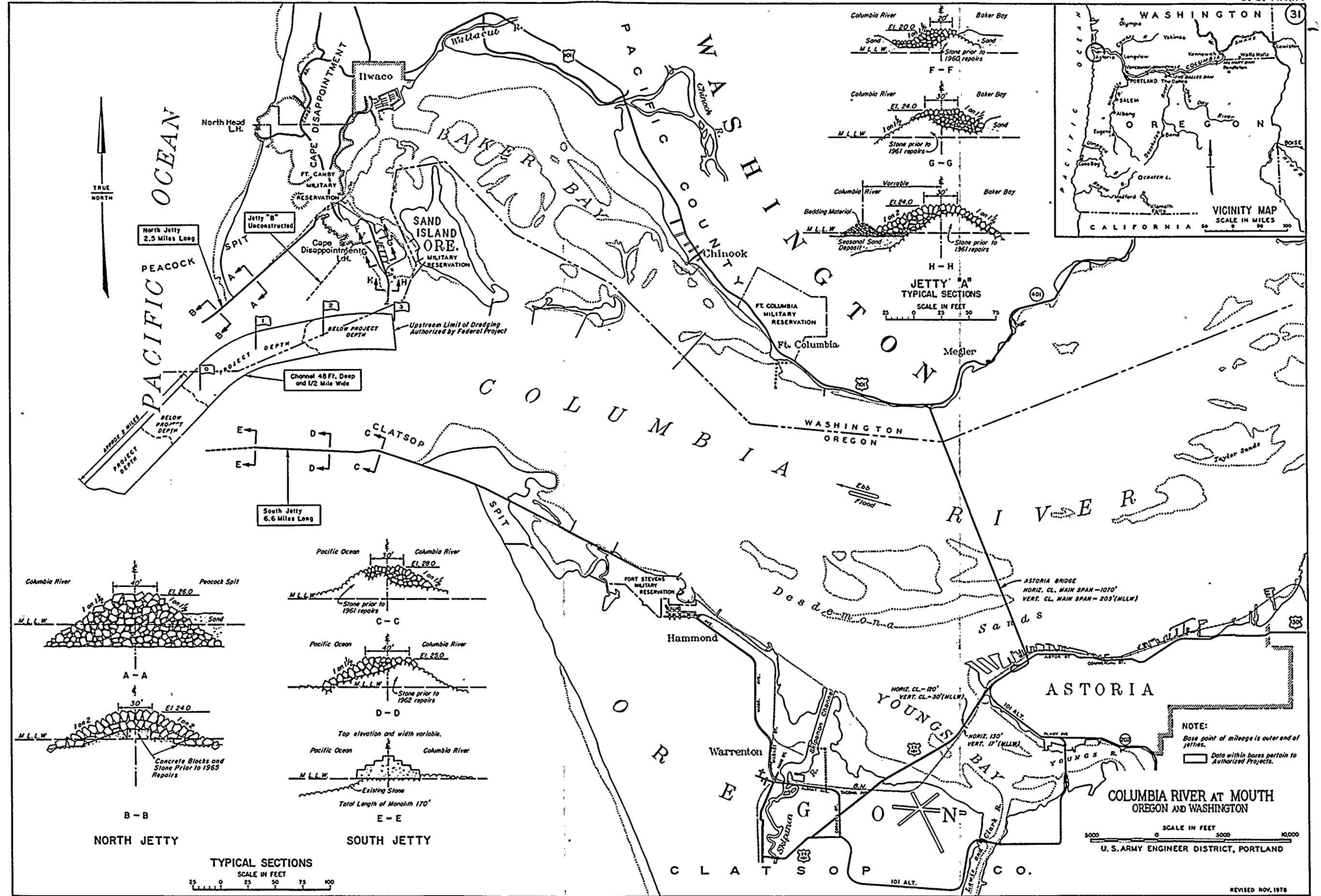
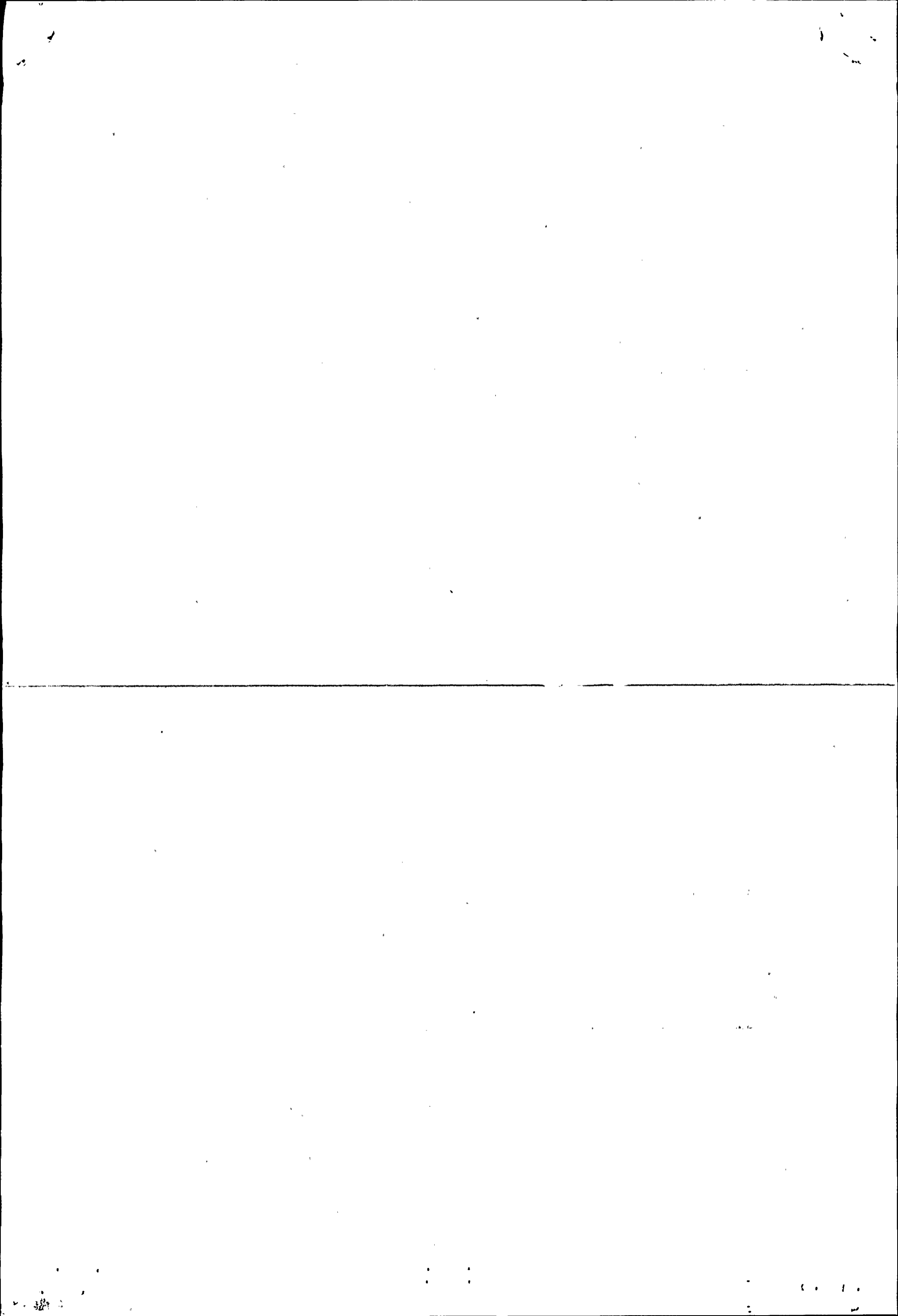


Figure 54 (From U.S. Army Engineer Division, North Pacific, 1979)



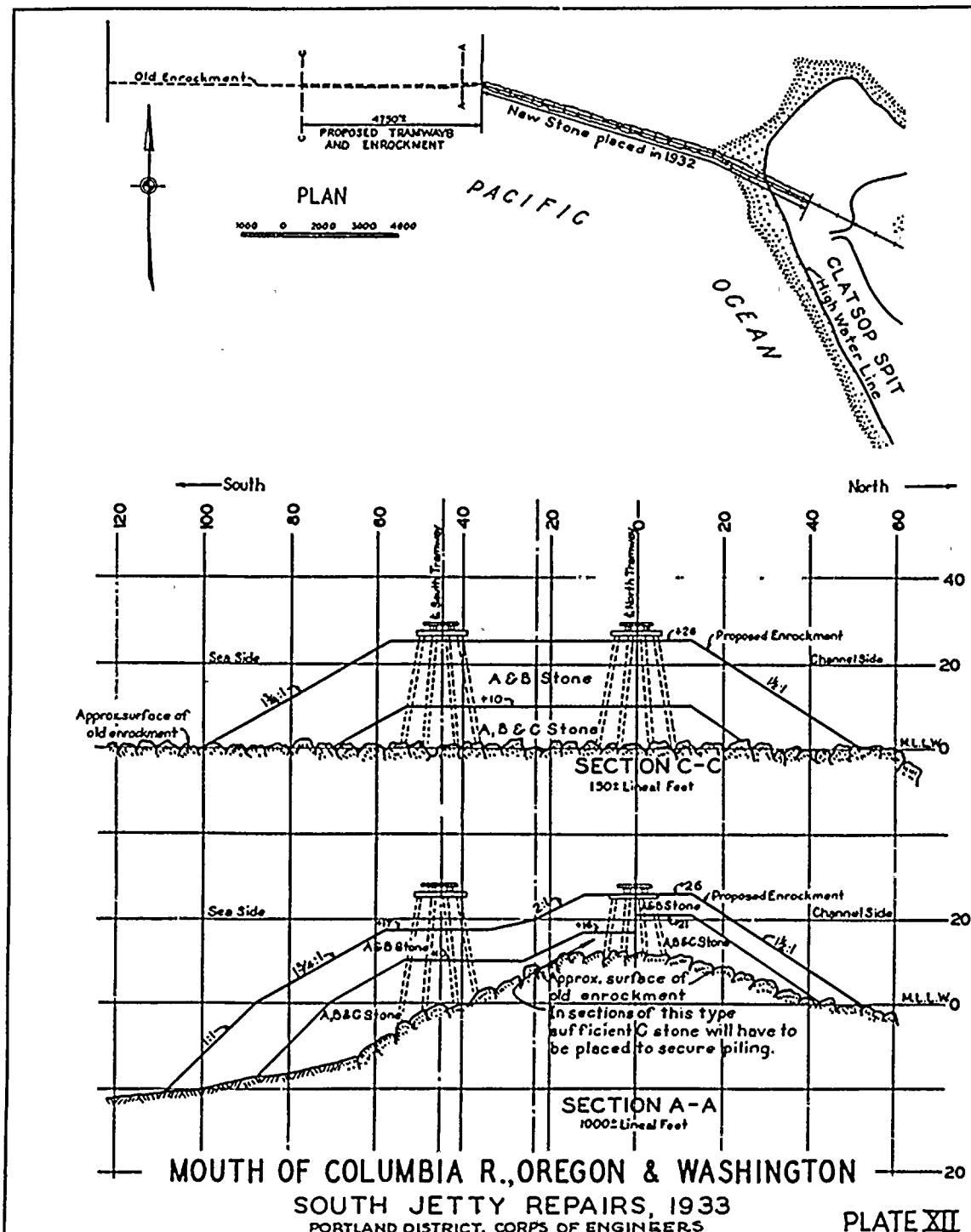
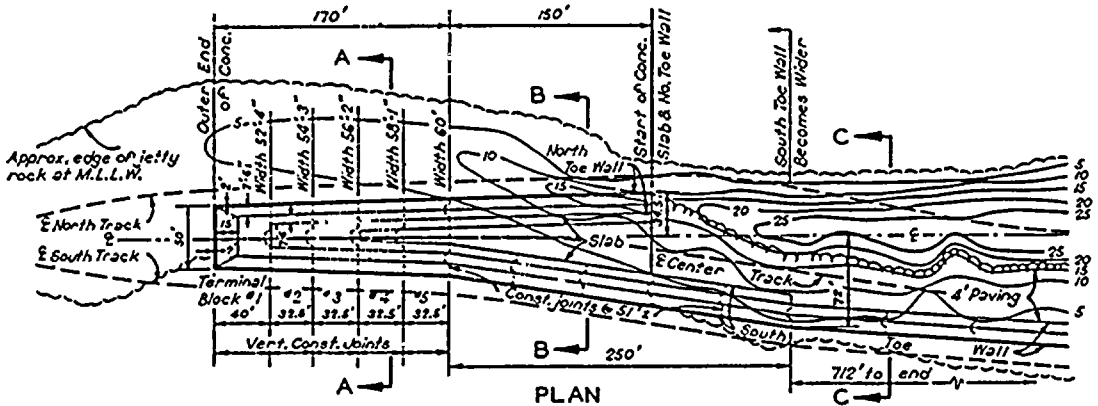
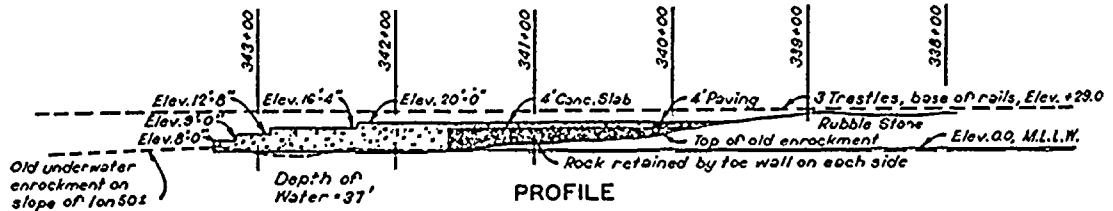


Figure 55

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MOUTH OF COLUMBIA RIVER,  
 OREGON AND WASHINGTON  
 SOUTH JETTY TERMINAL

Scale in Feet

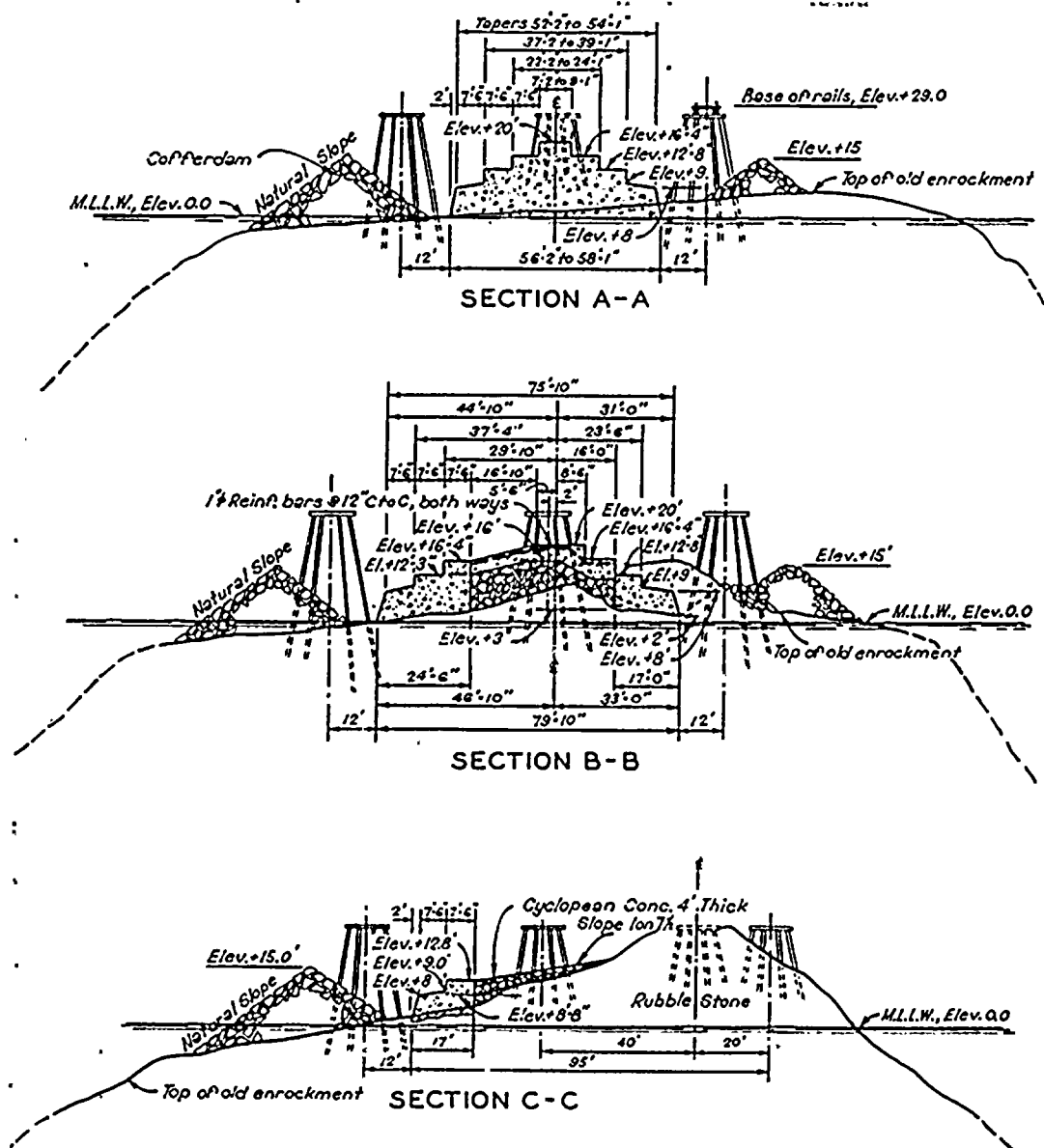


PORTLAND DISTRICT, CORPS OF ENGINEERS

Figure 56

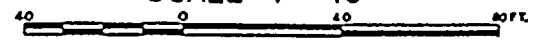






MOUTH OF COLUMBIA RIVER,  
OREGON AND WASHINGTON  
SOUTH JETTY TERMINAL

SCALE 1" = 40'



PORTLAND DISTRICT, CORPS OF ENGINEERS

Figure 57



from bottom ranges up to 76 ft. The outer 3,900 ft. have been beaten down to 10 or more feet below low water and rebuilding is not believed to be necessary. This outer section, in depth of 70 ft., serves as a protective apron."

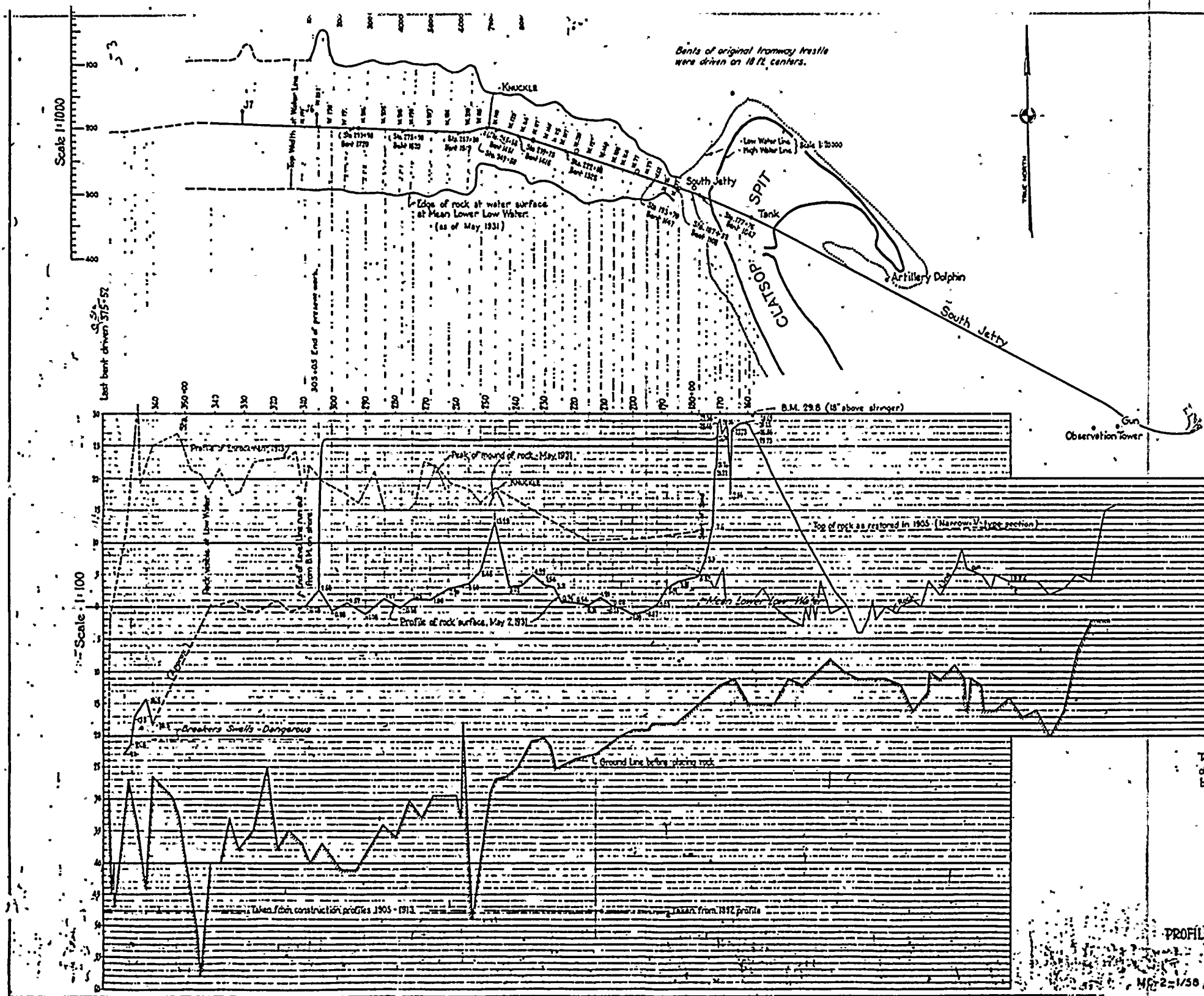
There appears to be a discrepancy here, in regard to the outer end of the jetty; a copy of a survey furnished to the writer by the U. S. Army Engineer District, Portland, is given in Figure 58. In this figure it can be seen that the survey of 2-3 May 1931 showed the top of the south jetty to have deteriorated to about mean lower low water (MLLW) from station 195+00 to 225+00 and from about 280+00 to 310+00, with it being at a higher elevation in between. From station 310+00 to 345+00 the top of the degraded jetty was estimated to be at about MLLW, with the notation on the chart stating "rock visible at low water." Note the horizontal scale, with the distance between stations 310 and 345 being 3500 feet. Another drawing was obtained by the writer from the U.S. Army Engineer District, Portland, showing the results of a survey made in April 1940. The survey between stations 324+00 and a little past 338+50 showed an elevation of about 25 feet above MLLW sloping down to about MLLW at 341+70. The surveyors estimated the top of the degraded jetty to be at an elevation of about MLLW seaward to about station 350+50, with a note on the drawing station "elev. MLLW to +2."

Thus, quite a few thousand feet of the jetty has remained in a degraded state for many decades with the elevation of the top of the jetty being at about MLLW.

#### Japan

Several extensive reports have been written in Japan on the damage to breakwaters located along the coast of Japan (Kitajima, et al., 1977;

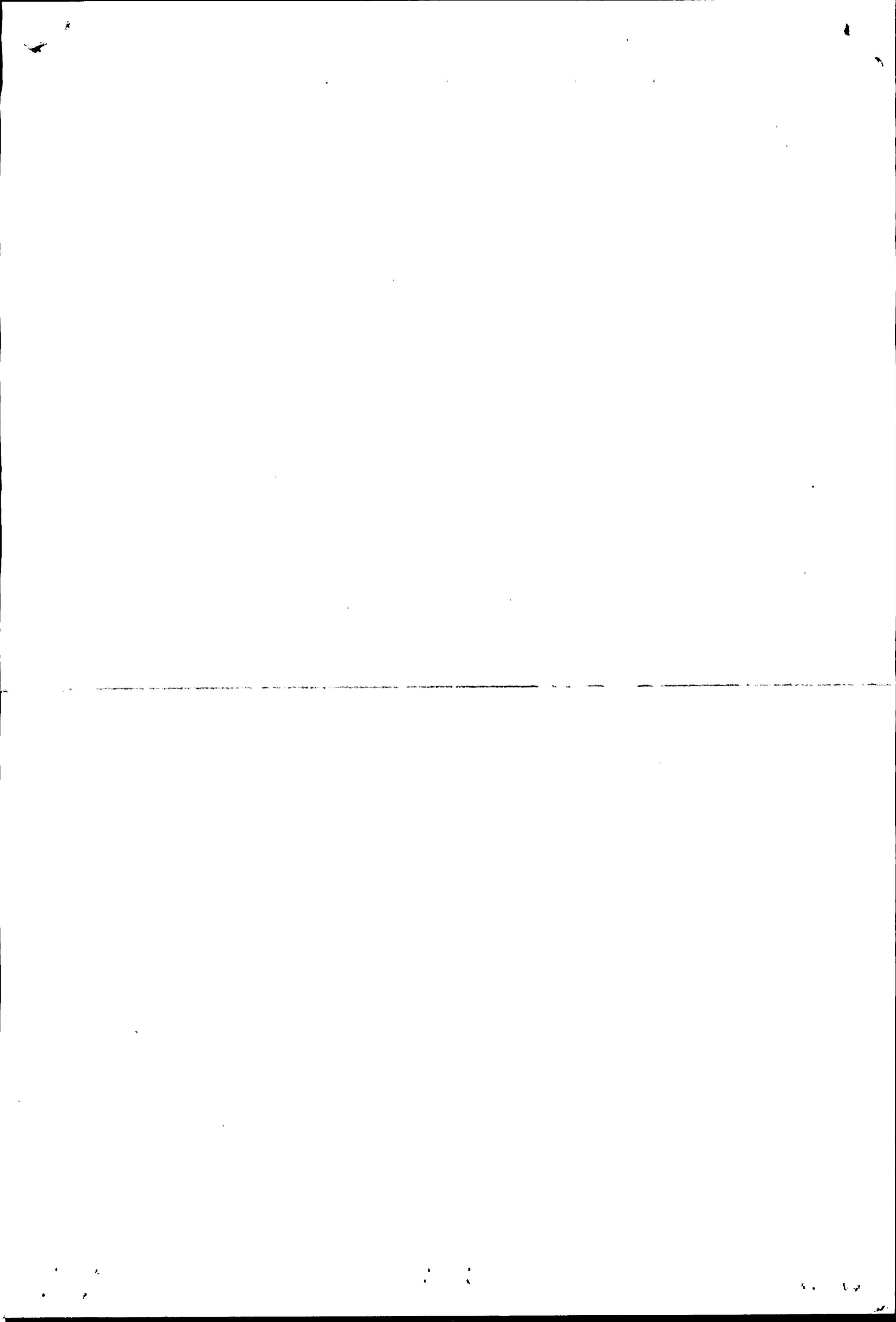




To accompany report dated May 9, 1931  
 of Special Board Columbia River appointed  
 by S.O. 15, Office Chief of Engineers, March 24, 1931  
 (COPY)

**MOUTH OF COLUMBIA RIVER**  
**SOUTH JETTY**  
**PROFILE AND TOP WIDTH FROM CLATSOP SPIT WESTWARD**  
 MAY 2-3, 1931  
 U.S. ENGINEER OFFICE, PORTLAND OREGON DISTRICT  
 (EXTENDED TO INCLUDE OLD JETTY EAST OF CLATSOP SPIT)  
 MC-2-1/50, A-7-46/34

FIGURE 58



Takeyama and Nakayama, 1977; Teruaki and Iguchi, 1977). These reports have been translated, and copies have been obtained by the writer. Of the order of a hundred breakwaters in Japan have been damaged, and details are given in these reports on the original plan, cross-sections and other details of the structures. Details are also given of the damage sustained under the attack of storm waves, and the sea conditions that existed at the times of the damage. Details of repair work done or proposed are also given. The writer went through these reports, but found that all of the breakwaters covered were mostly of the composite type. That is, they consisted of an underwater rubble mound base, with large concrete caissons or similar structures mounted on top of the base, with these structural elements extending up through the water surface to some distance above the high tide level. None of these structures could be studied in regard to the immediate problem of this report.

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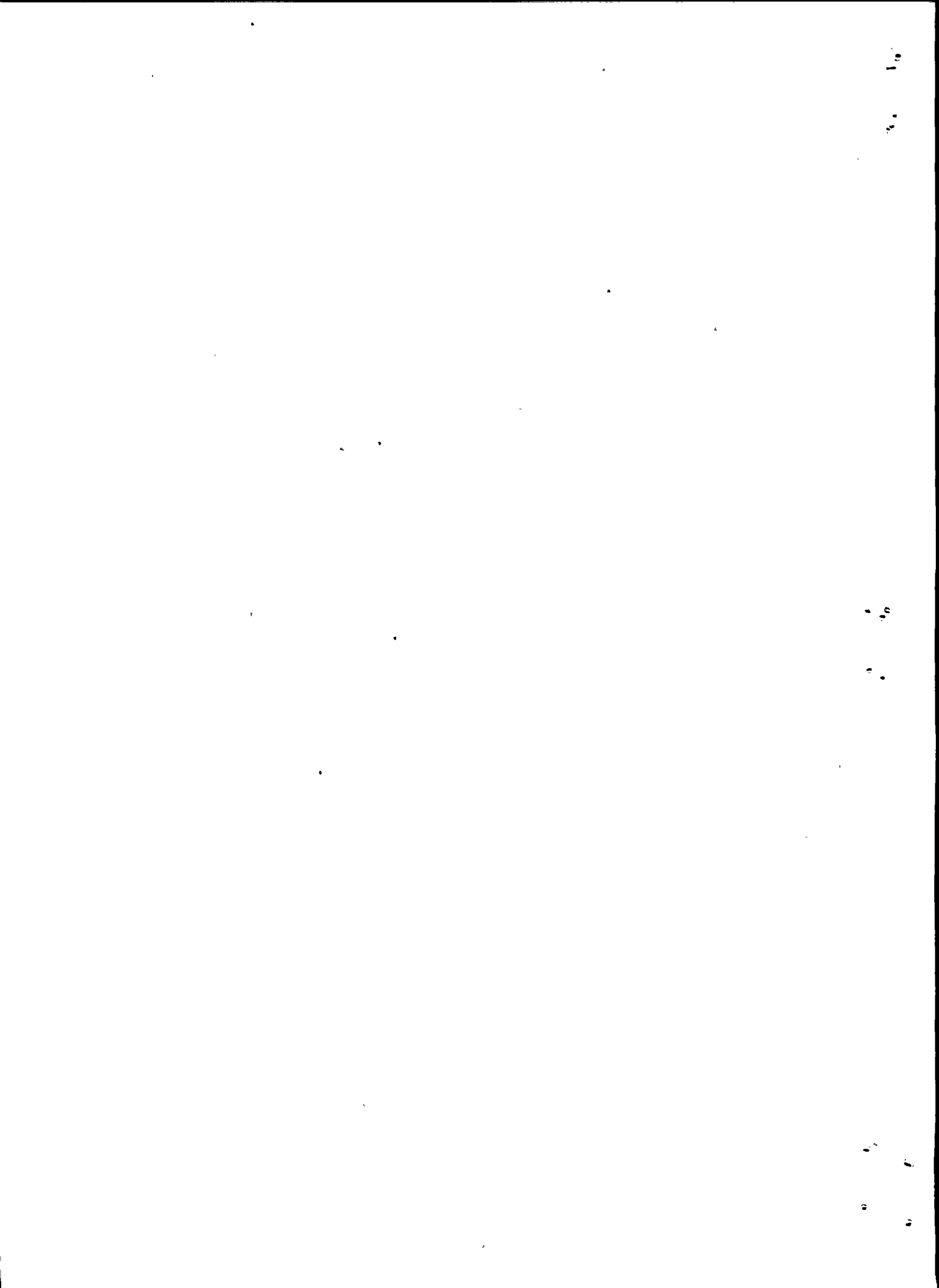
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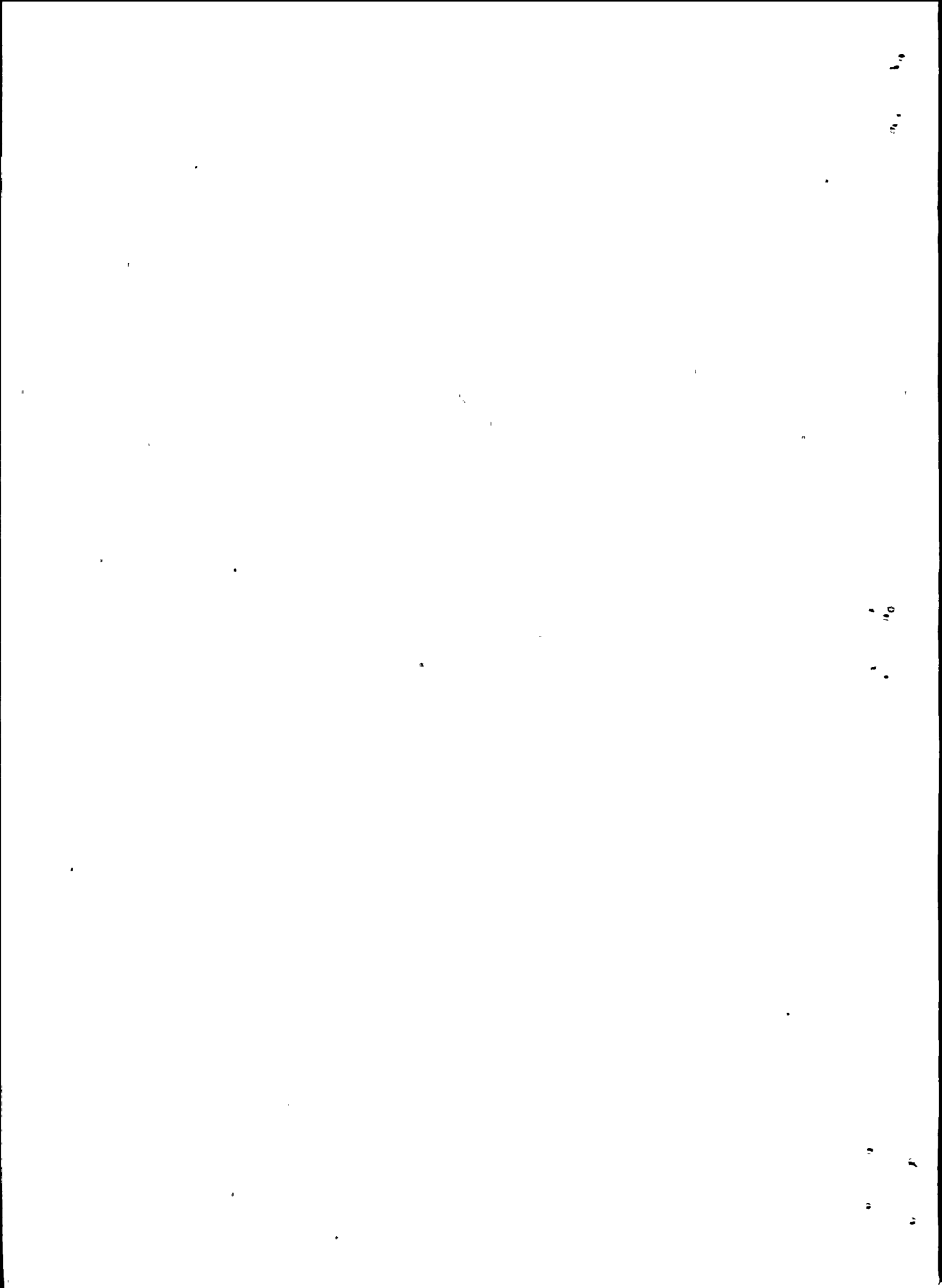
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