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SYNTHESIS OF BREAKWATER DESIGN AND DESIGN REVIEW PROCEDURES

BY HSIANG WANG

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Summary

This document disseminates the existing design information of breakwaters which is being proposed on the essential protective structures for power plants in offshore environments. The content of this document is generic in nature and covers all pertinent factors that need to be considered to insure adequate structural safety.

Major breakwaters within and outside this country have been reviewed first. Included in this review are three types: construction, site environment, and unique design features. Of particular emphasis is the documentation of case histories of structural failures, their causes, modes and extent of damage.

The current design practice, including the selections of environmental and structural factors, and the analysis and testing procedures for environmental and structural interactions, is summarized and evaluated.

An extensive literature research has been performed and references documented.

COO-AT(11-1)-2406

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by

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TABLE OF CONTENTS

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LIST OF FIGURES

LIST OF TABLES

1. INTRODUCTION

Prior to 1950, the design and construction of breakwaters was based largely on experience. They have suffered many setbacks and the confidence of engineers was quite shaky. Substantial effects have since then been given in research for better understanding and more rational design practice for breakwaters. To date, the amount of information available is considerable and is accumulating at an increasing rate. Unfortunately, this information, though voluminous remains largely unsynthesized.

Since breakwater design is a complicated problem involving numerous variables a rational design procedure has yet to emerge. Engineers who are commissioned to the breakwater design often face the cumbersome task of sorting out the right information applicable to their cases. For regulatory agencies who are responsible for reviewing the applications, the task of how to effectively apply the existing information is even more difficult because of the time constraint and, sometimes, the availability of this information.

The present document, prepared under the sponsorship of the U.S. Atomic Energy Commission, Contract No. AT(11-1)-2406, disseminates the existing design information of breakwaters which is one of the essential protective structures for power plants in offshore environments. The content of this document is generic in nature and covers all pertinent factors that need to be properly considered to insure adequate safety.

The work begins with a survey of literatures in which major breakwaters within and outside this country have been reviewed. Included in this review are three types; constructions, site environment, and unique design features.

Of particular importance is the documentation of case histories of structural failures, the reasons, modes and extent of damages.

The current design practice, including the selections of environmental and structural factors and the analysis and testing of environmental-structural interactions, is summerized and evaluated. The emphasis has been placed upon synthesizing practices in the United States, the European Countries and far East.

Based on the synthesized findings, guidelines for design evaluations are developed. Acceptable standards and their technical basis are given whenever possible. Since breakwater varies greatly in shapes and forms, considerable flexibility remains in design procedures and construction methods. The review guidelines provided in this document are by no means complete and shall be used with discretion.

2. SURVEY OF LITERATURES

2.1 Major Breakwaters in the United States

2.1.1 Breakwater in Hawaiian Islands

Breakwaters have been generally required to create most of the harbors in the Hawaiian Islands. The sites of major deep water ports are shown in Fig. 2.1. The coast of the islands are exposed to attack by waves generated in a vast expanse of ocean extending more than 2,000 miles in all directions.

A. Breakwater Design

Breakwaters built in the Hawaiian Islands prior to 1950, are generally of the typical section shown in Fig. 2.2. The controlling factors of design were the type and capacity of available equipment, native materials, labor costs, and sea conditions. The rubble-mound structures were featured by Keyed Stonework in the armor layer. Another feature of one of the older structures is the 25-ft. wide berm of the Hilo breakwater. It was surmized that the berm possibly contributed to the stability of the head of the structure.

B. Recent Breakwater Construction and Repair

The only new federal breakwater to be contructed since 1950 is at Kawaikae Harbor located on the northwestern coast of the island of Hawaii. The structure employs stone in the armor layer because an extensive coral reef provides protection against severe wave action.

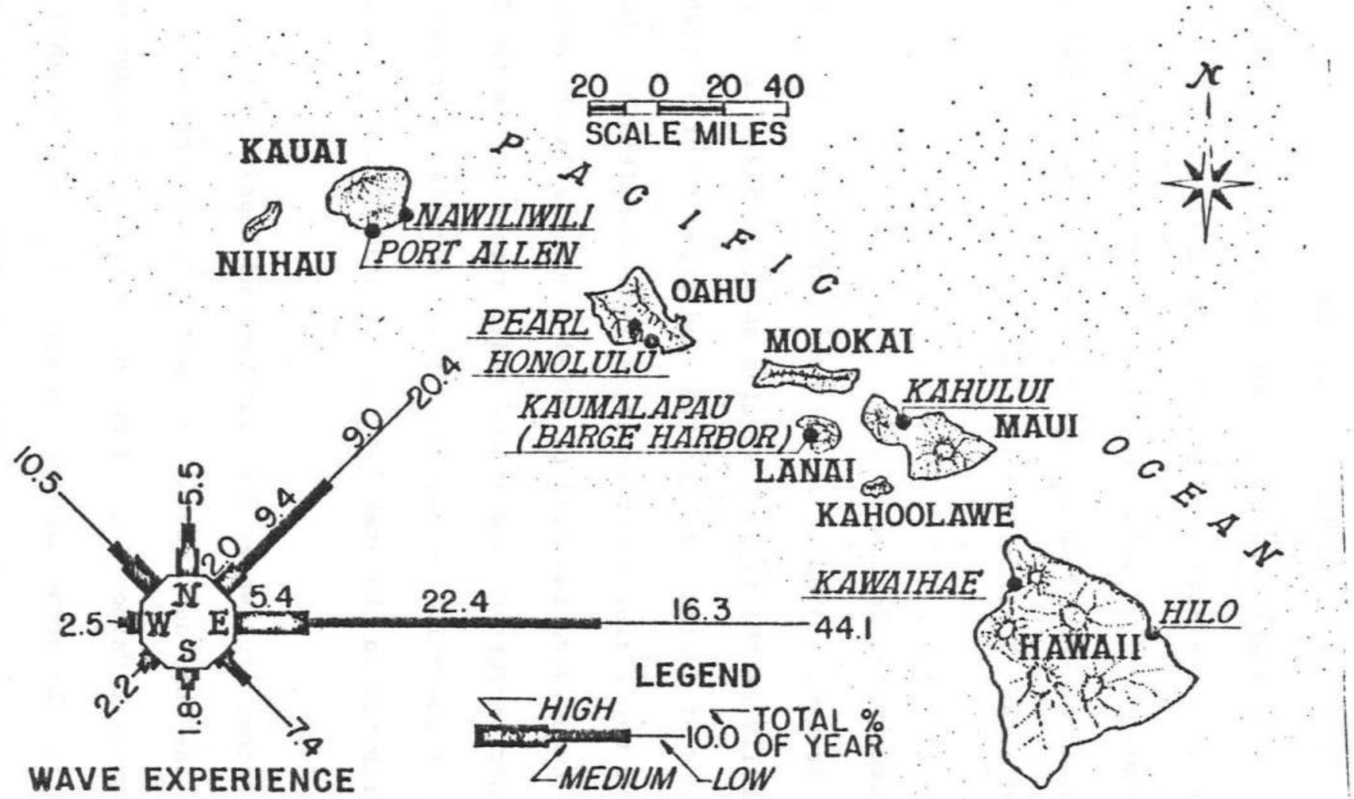


Fig. 2.1 Major Deepwater Harbors in Hawaii Islands

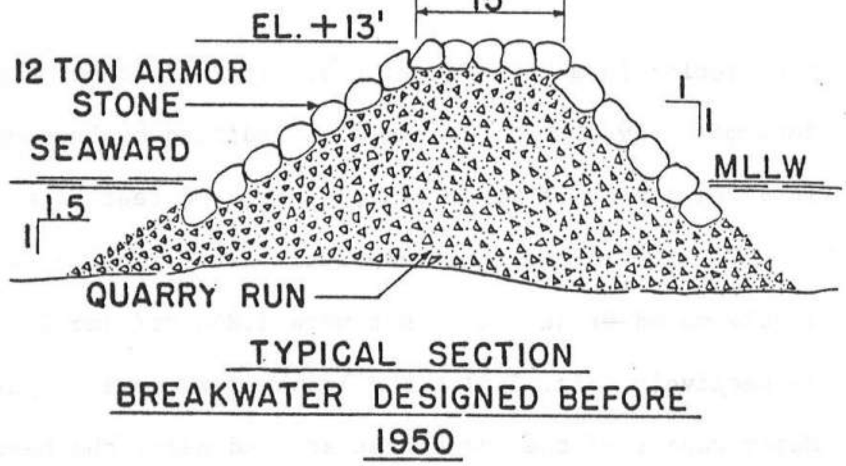


Fig. 2.2 Typical Section Breakwater Design Before 1950

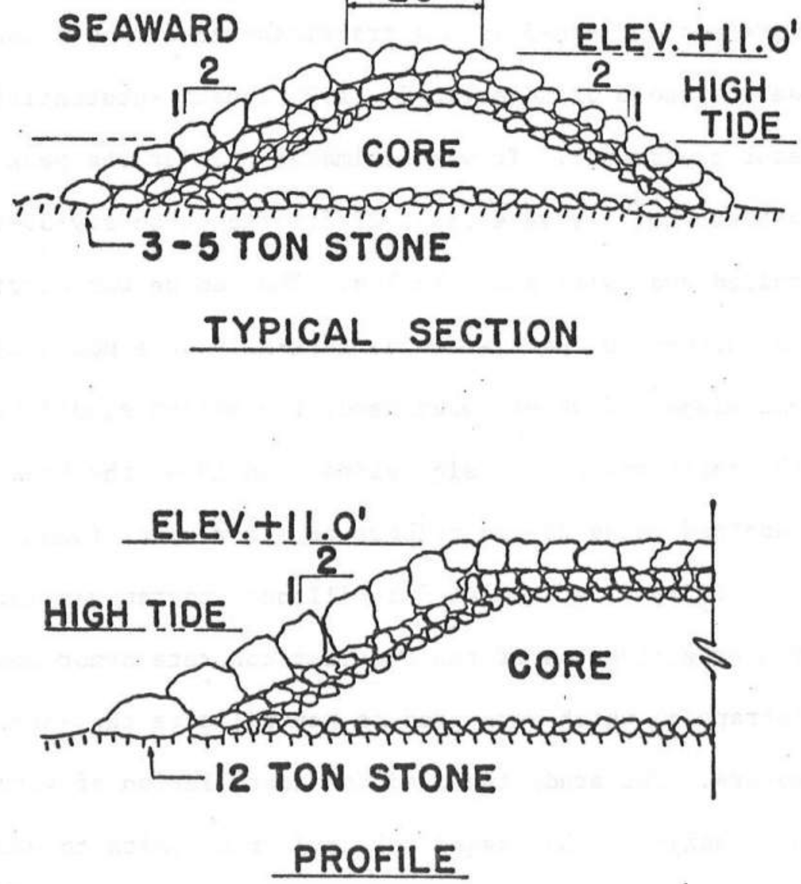


Fig. 2.3 Sections of Kawai Hae Breakwater

The section is shown in Fig. 2.3. The weight of armor stone was determined by Iribarren formula as modified by Hudson. The structure survived storms of waves ranging 15 to 18 feet high with no damage.

Kahului Harbor is a deepwater port flanked by east and west rubble-mound breakwaters that were 2,850 ft/ amd 2,396 ft. in length, respectively (Fig. 2.4). The breakwaters were completed in 1931. Major repair of the structures at, and near, the heads was accomplished in 1957 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. The severe storm of November 22, 1958 caused substantial damage to the east breakwater. It was estimated that at the peak of the storm the breaker heights were about 25 ft. About thirty 33-ton tetrapods were rolled away with a few broken. The damage was attributed to design deficiency in that tetrapods generally were not used at the seaward end slope. However, when used, the weight should be increased over the tetrapods on the side slope. In 1964, the breakwater was further repaired using 38-ton tribars as armor units (Ref. 79).

In 1966, a Kahului Surveillance Program was initiated to monitor the effectiveness of the pre-cast concrete armor units (tribars and tetrapods) which were used to rehabilitate the east and west breakwaters. The study included (a) installation of wave gage to obtain wave heights, (b) tagged selected armor units to observe their performances, (c) field survey of subsedence and change of shapes, (d) performing aerial surveys. In a recent Corps of Engineers publication

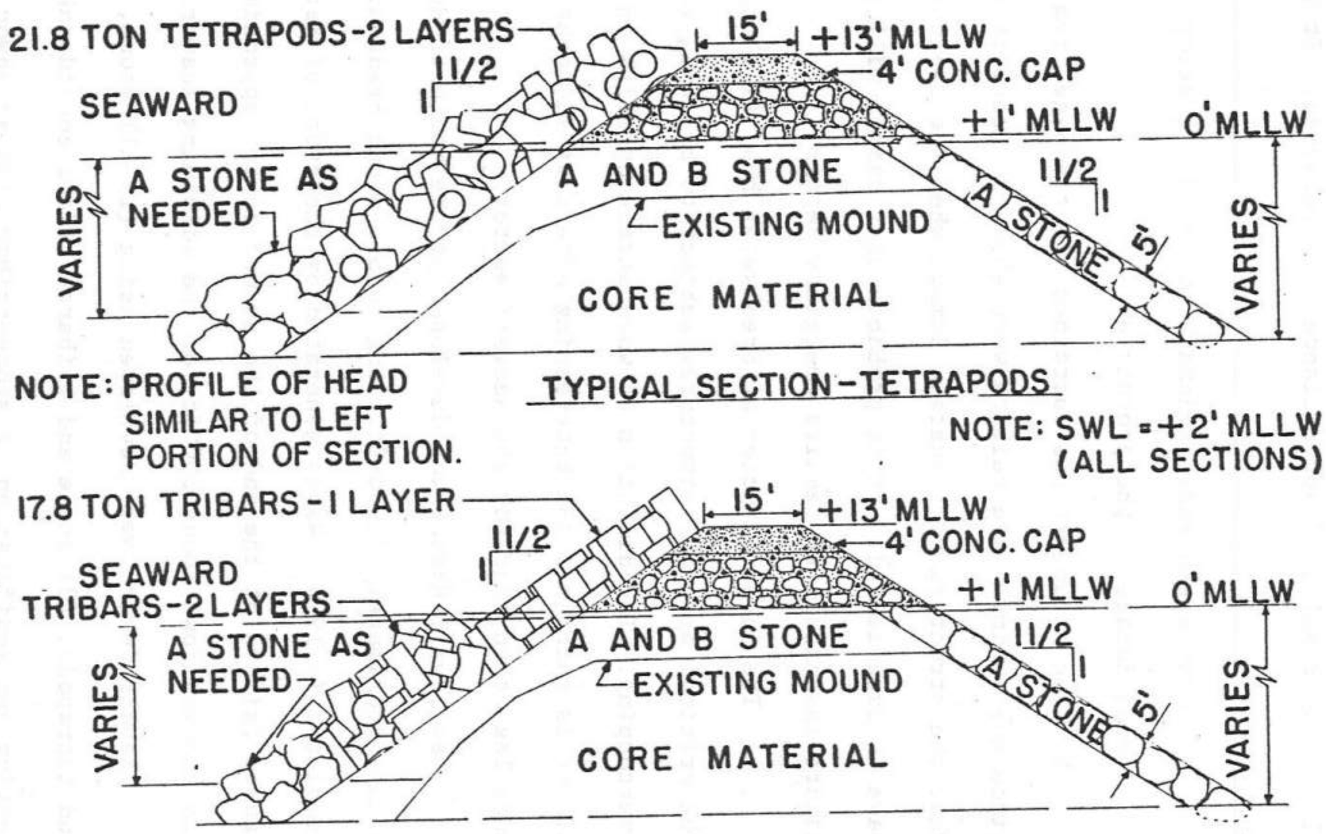


Fig. 2.4 Typical Sections of Kahului Breakwater

(1974) the results of surveillance were reported. It was concluded that:

a. Most areas showing damage in the Model study (Ref. 79) show damage in the prototype.

b. The structure has sustained minor damage from storm waves. Since the maximum wave heights were about 25 feet, it is suspected that the structure will sustain damage when the 34-foot design wave is experienced. It is probable that damage will be of a nature and extend to require emergency repair.

c. The inboard sector of breakwater head (110^{o} to 180^{o}) is the critical area for structures subject to breaking waves and overtopping. The direction of wave attack on units in the inboard sector is such that the buttressing effect of adjacent units is much less than that for the seaward sector.

Nawiliwili Harbor was developed at the mouth of Hulica River with approximately 2,000-ft. long rubble-mound breakwater that was completed in 1930. Waves generated by the storm of March, 1954 caused failure of the end of the structure for approximately 105 ft. and knocked down about 600 ft. of the adjoining seaward slope.

Repair design was developed using (1) all stone, (2) stone and tetrapods, (3) stone and tribars. After considerable laboratory testing and modification, a stone-tribar plan as shwon in Fig. 2.5 were adopted. According to laboratory tests, the structure withstood broken waves equivalent to 36 ft. The tribars used in the armor layer weighed 17.8-ton. The breakwater sustained very minor damage during Hurricane Dot, in 1959, with hindcasted deepwater wave height ranging from 27-35 feet. One reinforced concreted past was broken off by a tribar that had been swept from the top row of units over the crest.

Hilo Harbor, second largest seaport in the state, is on the Northeast Coast of the island of Hawaii in Hilo Bay. The existing harbor was constructed in 1930 and it consists of rubble mound breakwater 10,080 feet long and a control depth of 35 feet. Model tests and studies on Hilo Harbor were completed in 1966. The plan called for (a) modification of the existing breakwater by strengthening, raising, and extending the breakwater to a total length of 11,580 feet, (b) a west breakwater 3,000 feet long, (c) a 3,000-foot land dike. The existing breakwater sustained numerous damages in the head section and the structure trunk during storms. Emergency repair works were often required. It is evident the structure breakwater is inadequately designed.

C. Summary

Prior to 1950, breakwaters in Hawaiian islands were featured by keyed stone in the armor layer. The design of breakwater has undergone significant changes thereafter. A scientific approach has replaced the pure empiricism. Considerable design criteria have been established. An essential prerequisite to breakwater components were extensively used as armor units. Tribars were introduced.

Damages sustained were mainly due to breaking waves. Only local damages rather than total structure failure were experienced. In general, heads of breakwaters sustained more severe damage than trunks.

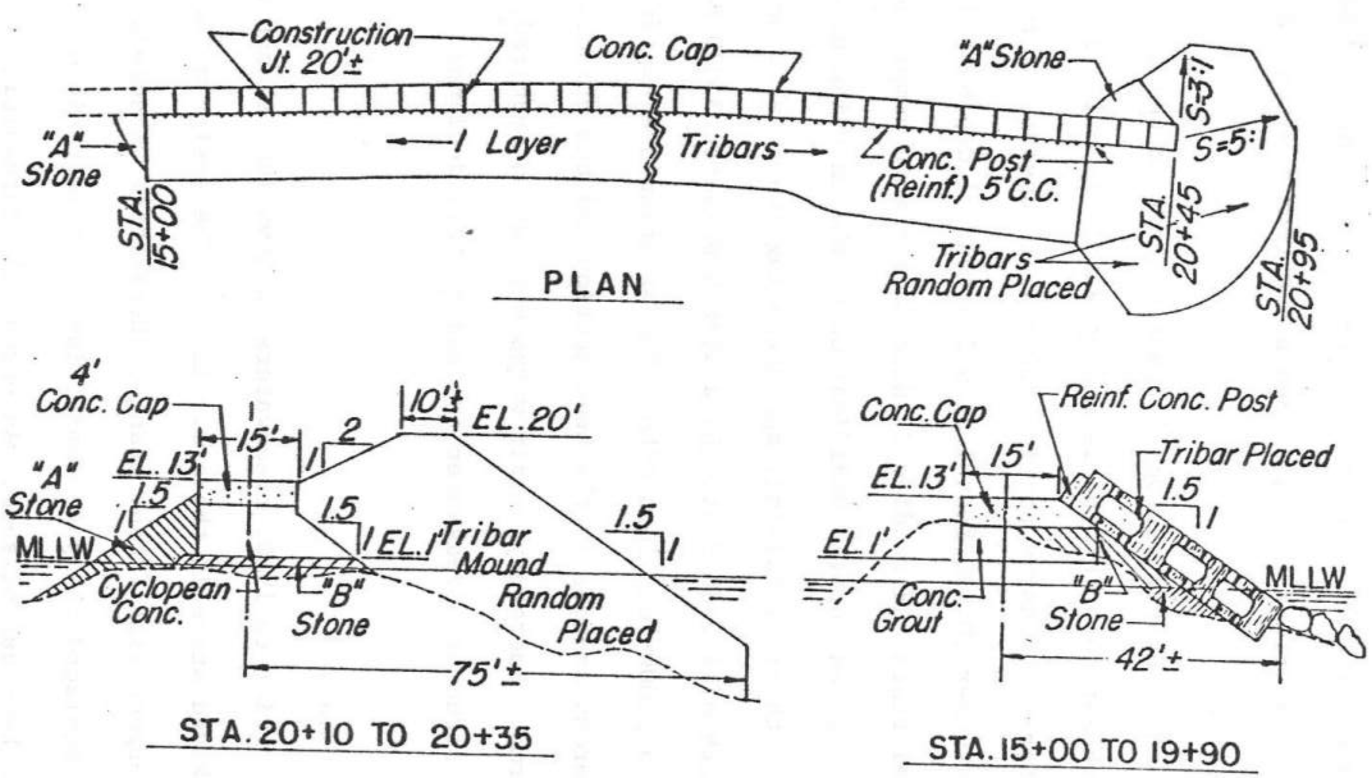


Fig. 2.5 Breakwater at Nawiliwili Harbor

2.1.2 West Coast

A. Crescent City, California

The outer breakwater at Crescent City extends from a point on the westerly side of the city 3,670 feet southeasterly and then continues easterly for another 1,000 feet for a total length of 4,670 feet. This is one of the major breakwaters in the West Coast of the United States. The major portion of the breakwater is stone-rubble-mound. The average size of the armor stone is 12 tons. A 560 foot dogleg section including the head of the breakwater, using tetrapod as armor unit, was completed in 1957. It was the first to use tetrapods in the mainland of the United States. This breakwater has undergone a series of modifications and repairing since 1948.

In November 1948 and 1949, the reach between Stations 30+00 and 40+00 was considerably damaged by two storms. Both storms had wave periods of approximately 12 seconds. The significant wave heights were 16 feet and 19 feet from the west-northwest and west, respectively. Storm damages were repaired and the extension of the breakwater beyond station 40+00 was modified by flattening the upper portion of the sideslopes to 1f on 1.5H on the harborside and 1V on 1.7H on the oceanside. A major portion of the damage was caused by waves overtopping the structure.

From October 26 to November 1, 1950, the Crescent City Area was subjected to successive storms. From wave hindcast it was estimated that during this period wave heights greater than 15 and 20 feet lasted for 56 and 34 hours, respectively. The breakwater seaward above

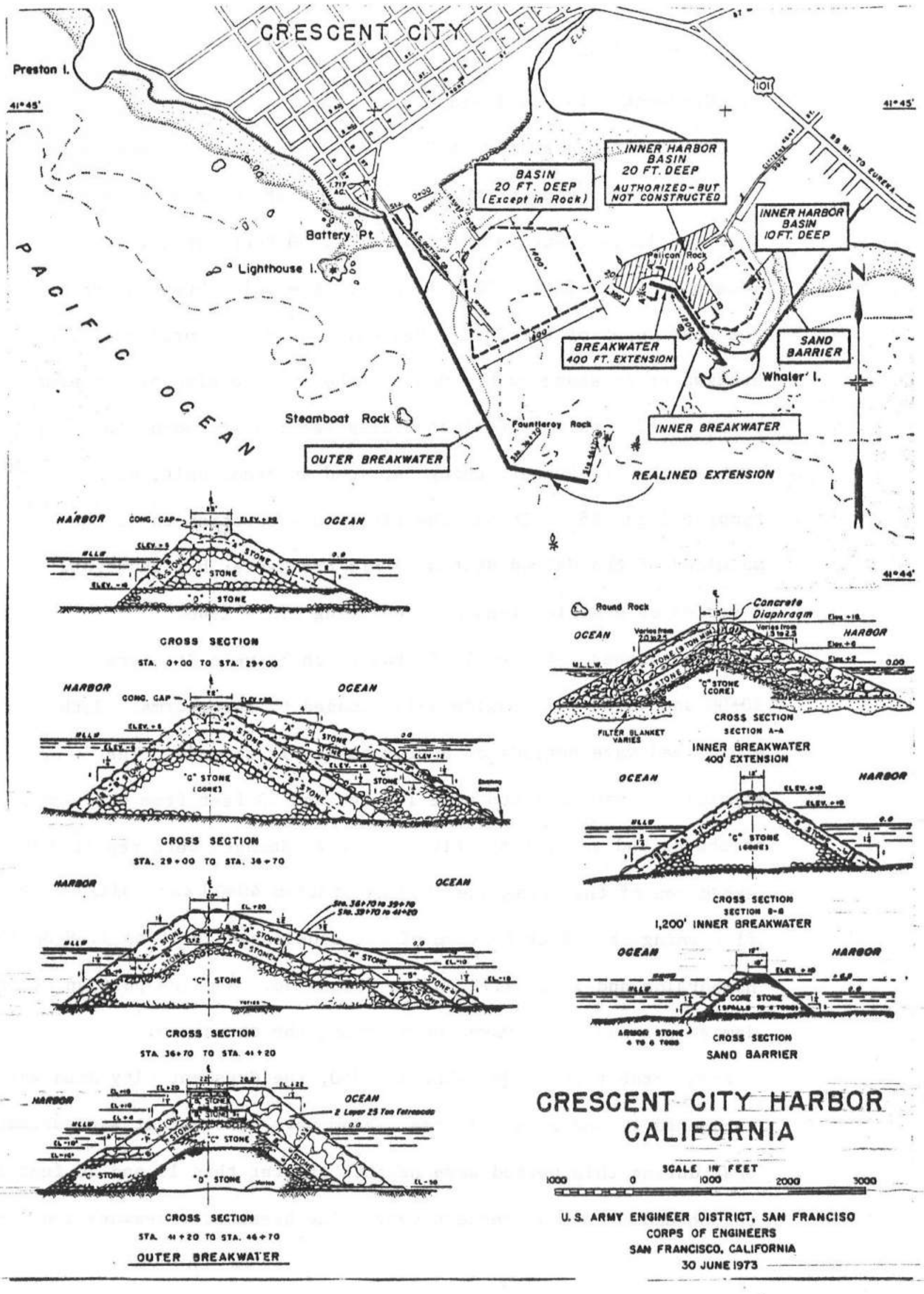


Fig. 2.6 Breakwater at Crescent City, California

elevation three feet m.l.l.w was displaced in the outer 450 feet of the breakwater. Repairs of the breakwater were started in 1951 and consisted of completing the concrete cap between Station 12+30 and 15+30 and repairing the breakwater shoreward of Station 37+00. A realigned 1,000-foot extension starting from Station 36+70 was constructed during 1956 and 1957. Between Stations 36+70 and 41+20, an armour blanket of 12-ton average stone was placed on the oceanside with slopes of 1V on 2.5H landward of Station 39+70 and 1V on 3.5H seaward of Station 39+70. Between Stations 41+20 and 46+70, 25-ton tetrapods were placed on 1V on 1.3H slope on the oceanside and 12-ton rock was placed on 1V on 1.5H slope on the harborside. The crest of this section was constructed at elevation +20 feet m.l.l.w., consisting of a two-foot thick concrete slab overlaying grouted rock. It was built in water of approximately -30 feet from mean-lower-low-water. The design wave height was 23 feet. During the winter of 1956-1957, the reach between Stations 36+70 to 39+10 suffered a loss of armor stone. The reach was repaired using 140 25-ton tetrapods in June, 1957.

Further damages to the outer breakwater were reported in March, 1960. Damages progressed until the 1964 construction season when the breakwater was repaired again. Between Stations 0+00 and 29+00, minor repairs were made. The severely damaged reach between Stations 35+00 and 38+20 was rebuilt with 12-ton rock placed on 1V on 4H slope on the oceanside. Around the conical head, Station 46+70 there was some settlement and consolidation of 25-ton tetrapods. Seventy-five 25-ton tetrapods were added to the conical head. In 1966 relatively minor repairs were made using 12-ton rock.

After completion of the repairs in 1966, no damage to the breakwater was reported until 6 June 1972, when field inspection revealed damage to the oceanside slopes of the breakwater. Damages were evident as:

a. Oceanside slope eroded in various reaches;

b. Concrete cap damaged at a number of stations.

The existing conditions and typical cross-sections are shown in Fig. 2.6.

The proposed repair has the following features:

a. Design wave height was increased to 33 feet for some reaches;

b. Berm will be incorporated in the design to protect the toe area;

c. 30-ton dolos units will be used for repair purposes. The dolosse would be constructed from high density and strength concrete without reinforcement. In general practice concrete armor units weighing over 20 tons are reinforced with nominal amount of steel, approximately 75 pounds per cubic yard of concrete to prevent damage during handling and placing. This amount of steel does not increase the structural strength of the unit, however, it tends to hold the damaged pieces together.

The K_D values used for calculations are based on two layers of dolosse or rough stones placed pellmell. The following K_D^{ψ} values were used: (K_D is the damage coefficient, for definition see p. 164).

Dolosse	11.3
Stone	3.5

B. Humboldt Harbor, California

The Humboldt Bay jetties were constructed beginning 1889 and completed in 1927. There are north and south jetties with location and typical cross sections shown in Fig. 2.7. Originally, the jetties were constructed with slopes of approximately 1 on 1.5. On both sea and channelsides, crest width was approximately 20 feet, and the crest elevation varied from about +12 feet at the shoreward end to +19 feet at the seaward end. Parapet walls about 6 feet in width, and 4 feet high, constructed of both stone embedded in concrete and 20-ton concrete blocks, extended shoreward from a point about 400 feet from the outer end of the jetties, along the south side of the crest.

During the period of 1930 to 1970 the jetty has sustained progressive damages of various kinds, including erosion of trunk sections, broken-off concrete monoliths and damages at head sections. Constant maintenance and repairs were made during this period.

The most recent repairs were completed in 1972. Most of the repair work was made on foundation of stones that were displaced from the breakwater by wave action. In areas where there are no displaced stones, the ocean bottom consists of sand.

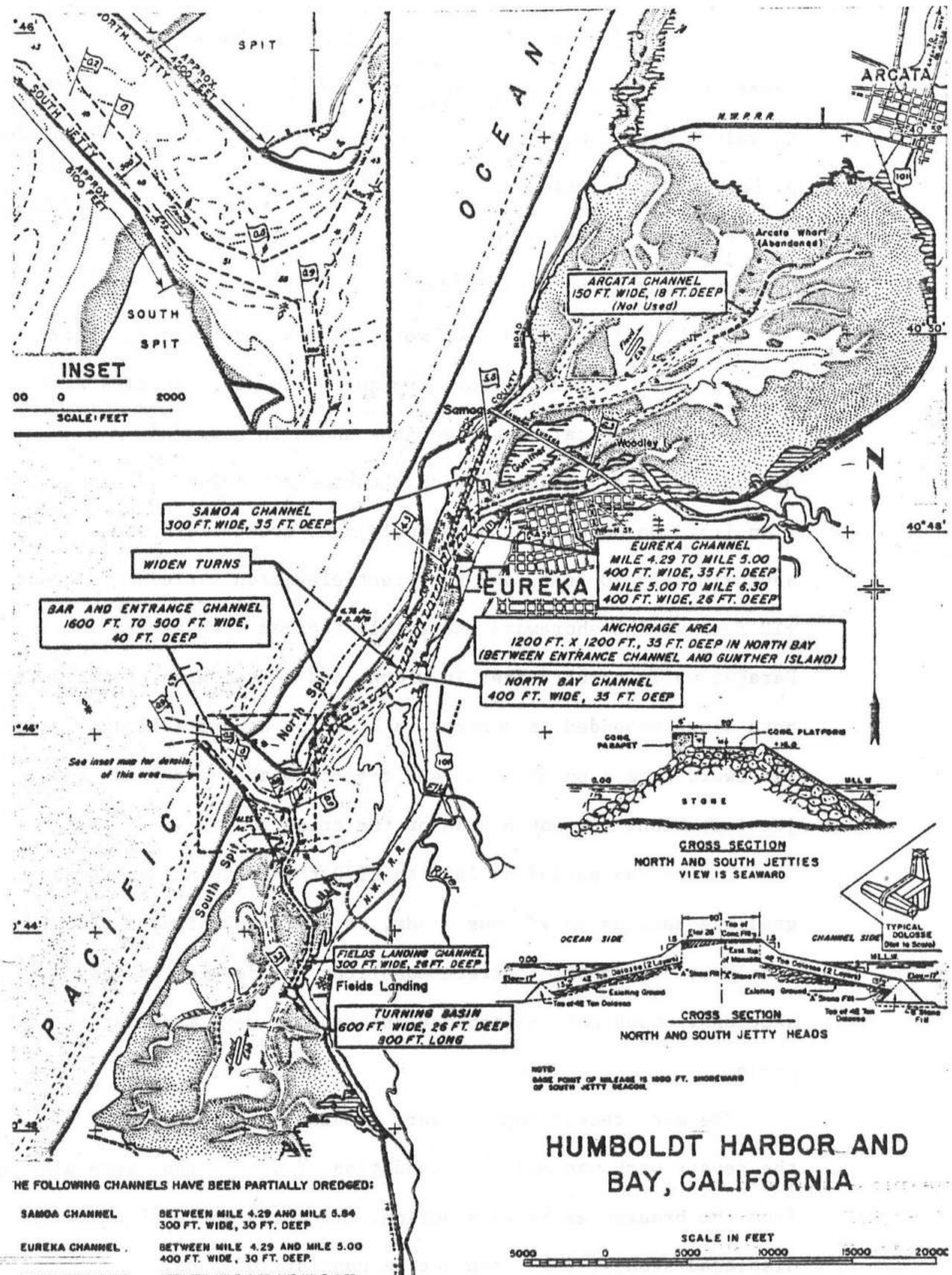


Fig. 2.7 Breakwater at Humbaldt Harbor

The design wave heights acting on jetty heads were determined from refraction studies. The design waves acting on jetty trunks were computed by dividing the depth of water by 1.3 seven wave heights away from the structure. The values of design wave height varied from 40 ft. at structure heads and bayward sections of the jetties to below 20 ft. at landward ends of the jetties.

Hydraulic model studies were conducted at the U. S. Army Engineer Waterways Experiment Station in Vicksburg, Mississippi, to verify the stability of the design section, using various shapes for armor stones. Repair sections were tested using cube-shaped blocks, tetrapods, tribars, tri-longs and dolosse. For protection from 40 feet waves at the heads of the jetties, dolosse were the only shape that could be lifted by presently available lifting equipment to the toe distance required to provide stability of the jetty heads. Due to the adverse conditions and high waves, the design and the repair work are also predicated on placing the dolosse by a 4600 V.C. Manitowac Crane from the crest of the jetty. Limitations imposed by this placing equipment are as follows:

a)	Maximum size of Dolos	42 tons
b)	Maximum distance of placement (from edge of monolith)	163 feet

Dolosse are reinforced to resist breakage and to aid handling. A few unreinforced units were also placed near the waterline for test purposes. A unit weight of concrete of 155 pounds per cubic foot which could be obtained by using locally available aggregate was selected. Stones of 10 to 14 tons were used for trunk repairs.

The K_D values used for calculation are based on two layers of Dolosse or rough stones placed pellmell. The following K_D values were determined by model studies for Dolosse:

Dolosse:	
Trunk	9.0
Head	8.3
Rough Stones:	
Trunk	3.0
Head	2.5

The following steps and sizes for repair section were selected:

a) North jetty

Station	Slope	Stone Size
68+70 to 74+10	1 on 5+	42 ton Dolosse
Channelside:		
67+00 to 68+70	1 on 1.5	42 ton Dolosse
65+00 to 67+00	1 on 3	10-14 tons
55+00 to 65+00	1 on 2.5	10-14 tons
20+00 to 55+00	1 on 2	10-14 tons
Oceanside:		
68+00 to 68+70	1 on 1.5	42 ton Dolosse
67+00 to 68+00	1 on 3	10-14 tons
62+00 to 67+00	1 on 2.5	10-14 tons
54+00 to 62+00	1 on 2	10-14 tons

b) South jetty

Station	Slope	Stone Size
Channelside and Oceanside:		
85+00 to 90+35	1 on 5	42 ton Dolosse
84+00 to 85+00	1 on 1.5	42 ton Dolosse

A recent survey in the Spring of 1974 showed a small percentage of displacement and breakage of dolosse units in the vicinity of water line in both the south and the north jetties.

The unreinforced units also sustained higher percentages of breakage than the reinforced units. Although the evidence was rather limited, some investigators tended to belive that reinforcement is beneficial.

C. Santa Cruz Harbor, California

Santa Cruz Harbor is located at the northern end of Monterey Bay about 65 miles south of San Francisco. The breakwaters were completed in 1963. Typical sections are shown in Fig. 2.8. 25-ton quadripods of two layers were used on the west jetty. This breakwater together with the breakwaters at the Crescent City and Humbolt Bay are the few in the Mainland that ultilized artificial armor units. The rest of the breakwaters in this country used mainly stones as armor units.

D. Other breakwaters in California Coast

The breakwaters are basically rubble-mound structures with stones as the armor units. Cross sections of these breakwaters are quite similar in design with some variance from one to the other. The following Table provides a summary of these breakwaters.

Table 2.1 Breakwaters Along the Caifornia Coast

Location	Length (ft)	Seaward Armor*	Slope	Controlling Depth (ft)
Half Moon Bay, Ca.	3,670 (West) 4,420 (East)	A-3 Stone	1V 1.75H	24
Monterey Bay, Ca.	1,700 (West) 1,100 (East) 3,300 (North Petached)	A-1 Stone	1V 1.5H	36
Morro Bay, Ca.	3,700	A-1 Stone		16
Santa Barbara Harbor, Ca.	2,500 (East) 1,600 (Detached)	10 tons Stone	1V 1.5H	20

Table 2.1
Continued

Location	Length	Seaward Armor*	Slope	Controlling Depth (ft)
Dana Point, Ca.	5,500 (West) 2,250 (East)	A-1 Stone	1V 1.5H	20
L.A. and Long Beach, Ca.	11,200 (West) 18,500 (Middle) 13,400 (East)	A-1 Stone	1V 2.0H	40
Redondo Beach, Ca.	2,800 (North) 600 (South)	A-1 Stone	1V 2.0H	15
Marina Del Rey, Ca.	2,000 (North) 760 (South) 2,330 (Detached)	A-1 Stone	1V 2.0H	20
Newport Bay, Ca.	1,620 (East) 2,860 (West)	A-1 Stone	1V 1.5H	20
San Diego Harbor, Ca.	7,500	A-1 Stone	1V 1.5H	42
Mission Bay, Ca.	3,300 (North) 4,270 (Middle) 2,050 (South)	A-1 Stone	1V 3H	20

E. Breakwaters in the northern part of the West Coast

There are about 11 major jetties along the coast of Oregon. They are summarized as follows:

Location	Length	Construction	Slope	Controlling Depth (ft)
Columbia River	2.5-mile (North) 6.6-mile (South)	Stone Mound	IV 1.5H	49
Tillamook Bay	5,700 ft. (North) 8,000 ft. (South)	Stone Mound	IV 2H	10
Yaquina Bay	7,000 ft. (North) 8,000 ft. (South)	Stone Mound	IV 2H	35
Siuslaw River		Stone Mound	IV 1.5H	13
Umpqua River	8,000 ft. (North) 4,200 ft. (South)	Stone Mound	IV 1.5H	19

Table 2.1
Continued

Location	Length	Construction	Slope	Controlling Depth (ft)
Coos Bay		Stone Mound	IV 2H	32
Coquille River	3,450 ft. (North) 2,700 ft. (South)	Stone With Concrete Cap	IV 2H	14
Port Orford	550 ft.	Stone Mound	IV 2H	2
Rogue River		Stone Mound	IV 1 1/2H	9
Cheteo River		Stone Mound	IV 1 1/2H	8

All the jetties in this area can be classified as high-tide jetties that experience tidal variations up to 12 feet. No major failures have occurred in the past. Figures 2.9 and 2.10 showed two typical examples of the constructions.

Along the Washington Coast, there is only one major breakwater which is the breakwater at Neah Bay (Fig.2.11). This breakwater was completed in 1944 at a cost of over $1.9 millon dollars. It is a rubble mound structure with 1V 1.5H slope. For a period of 30 years after its completion, the breakwater suffered no major damage but some expected deterioration.

2.1.3 East Coast

There are numerous jetties along the east coast, but there are no major breakwaters similar to the ones found in Hawaii or the West Coast. Almost all of the breakwaters in the east coast are rubble mound structures using stone as armor units. Most of the structures were built for navigational purposes and a few were built for hurricane protections. No catastrophe failure has been experienced.

Along the coast of Maine and New Hampshire there are more than 20 jetties and breakwater protected harbors. The jettied entrances include Richmond Harbor, Kennebunk River, Sasanoa River, Saco River, Lubec Channel, Portland Harbor, Wells Harbor, Scarboro River, Eastport Harbor, and Hampton Harbor. Breakwaters are at Bar Harbor, Isles of Shoals Harbor, Matinicus Harbor, Moosabec Bar, Richmond's Island Harbor, Criehaven Harbor, Eastport Harbor, Portland Harbor, Rockland Harbor, Wood Island Harbor, Little Harbor, and Rye Harbor.

Along the coast of Massachusetts, Connecticut, and Rhode Island, the jettied and breakwater-protected harbors totaled more than 45. (Nineteen in Connecticut, Five in Rhode Island, and Twenty-two in Massachusetts.) Most of them are minor structures. Descriptive information can be obtained from the U.S. Corps of Engineers District Office and their publications in water resource development. There is also a detailed technical description on the engineering methods of the hurricane protection barrier that spans the Narragansett Bay at New Bedford (Ref. 97). All of the structures in this area are rubble mound with stone armor units. Major failures have not been found.

Along the New Jersey and Delaware waters jetties and breakwaters were constructed at a number of locations including the recently rehabilitated jetties at Manasquam Inlet (New Jersey), Breakwater Harbor (Delaware Bay), and Indian River Inlet (Delaware).

The only major jetty in Maryland is at Ocean City Harbor. It is constructed in water of approximately 16 feet in depth with a top elevation of 9 feet above mean low water. The project has not been completed. The jetty is another typical rubble mound structure with stone armor units.

Along the coast of Virginia are the following major wave protections:

a. Craney Island Spur Levee. A 2,300-foot long spur levee, capped with heavy riprap, is located at the Craney Island Disposal Area within Norfolk Harbor. The purpose of the levee is to provide wave protection for the rehandling facilities. There have been no major repairs to the levee since its construction in 1957. Pertinent design details can be obtained from the U.S. Corps of Engineers, District Office.

b. Rudee Inlet Jetties. Twin riprap jetties provide navigable access from the Atlantic Ocean to Rudee Inlet in the city of Virginia Beach some seven miles south of Cape Henry and the Chesapeake Bay. The inlet system consists of a north rubble mound stone jetty extending 800 feet into the Atlantic Ocean, a 492-foot long timber sheet pile weir extending into the ocean 510 feet south of the former, with a 280-foot rubble mound stone section coupled at its seaward end. A sand deposition basin, designed to retain 100,000 cubic yards of sand, and a 10-foot by 90-foot navigation channel was also provided. The system was constructed in 1967 and has experienced no damage to date. Further details and drawings of the system can be obtained from the City Engineer, City of Virginia Beach.

c. Little Creek Inlet Jetties. Twin jetties are also present at Little Creek Inlet on the south shore of the Chesapeake Bay. The inlet is basically utilized by naval amphibious craft stationed at the Little Creek Naval Amphibious Base. The overall length of the eastern and western jetties are 1,400 feet and 1,100 feet, respectively. Built around 1927, neither has experienced any major damage.

d. Pier 12 - Aircraft Carrier Breakwater. A rubble mound stone breakwater, roughly 1,500 feet long, is located adjacent to Pier 12 at the Norfolk Naval Amphibious Base. The breakwater protects aircraft carriers based in Norfolk from waves entering Norfolk Harbor from the Chesapeake Bay. Design details of this breakwater may be obtained from the Naval Facilities Engineering Command, Norfolk Station, Norfolk, Virginia.

The only jetty structure on the coast of North Carolina is located at Masonboro Inlet. It is a low weir, rubble mound structure. A full description of the design is given by Magnuson (1966).

Two major jetties existed in South Carolina, located respectively at the mouth of Charleston Harbor and at the mouth of Georgetown Harbor. Both were old jetties. For instance, the construction of Charleston jetties was commenced in 1778 and carried on intermittently as appropriations were made until completion in 1896. These jetties have experienced many storms and hurricanes and have been repaired many times. No major failure, however, has been documented.

The only major jetties in Georgia lie on each side of the entrance to the Savannah River Channel. These two jetties are each approximately 11,500 feet long and were constructed between the late 1890's and 1925. They consist of quarried stone placed on a log and brush mat. No repairs have been made since their construction, and recent surveys indicate that no repairs are necessary. The original drawings of these jetties show that three sizes of stone were used; the largest of which is estimated to be in the 15-20 ton range.

Because of the shallow depths along the coastline of Florida, the breakwaters and jetties are not generally high or large structures. There are two breakwaters and eleven jettied inlets to harbors. With the exception of one jettied inlet the structures are stone construction.

The breakwaters are located at Key West, Florida, and Arecibo Harbor, Puerto Rico. The Key West breakwater was built in 1967 and no maintenance has been done. The crown is elevation +6 and cap or cover stone size 2 to 6 tons. The existing bottom was about 12-foot mean low water depth. The Arecibo Harbor breakwater had maintenance done in 1951. Its crown elevation is +15 and cap stone size 10 tons minimum. The existing bottom is about 15-foot depth.

The eleven jettied inlets or entrances to harbor are twin jetties extending from shoreline to depths of 10 to 20 feet. The jetties are generally low with crown elevations of +6 to 10-foot range. Cap stone size varies from 6 to 14 tons.

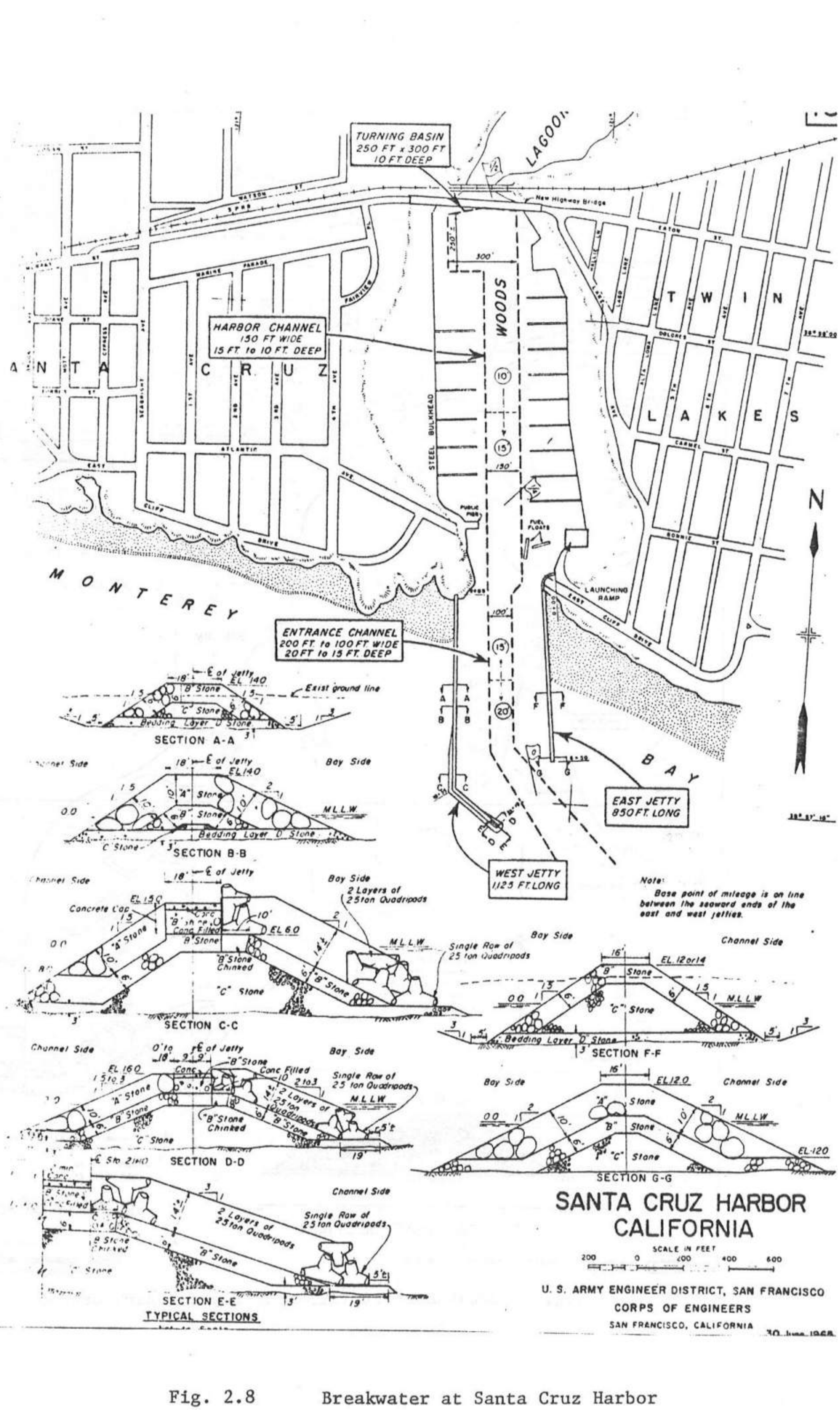


Fig. 2.8 Breakwater at Santa Cruz Harbor

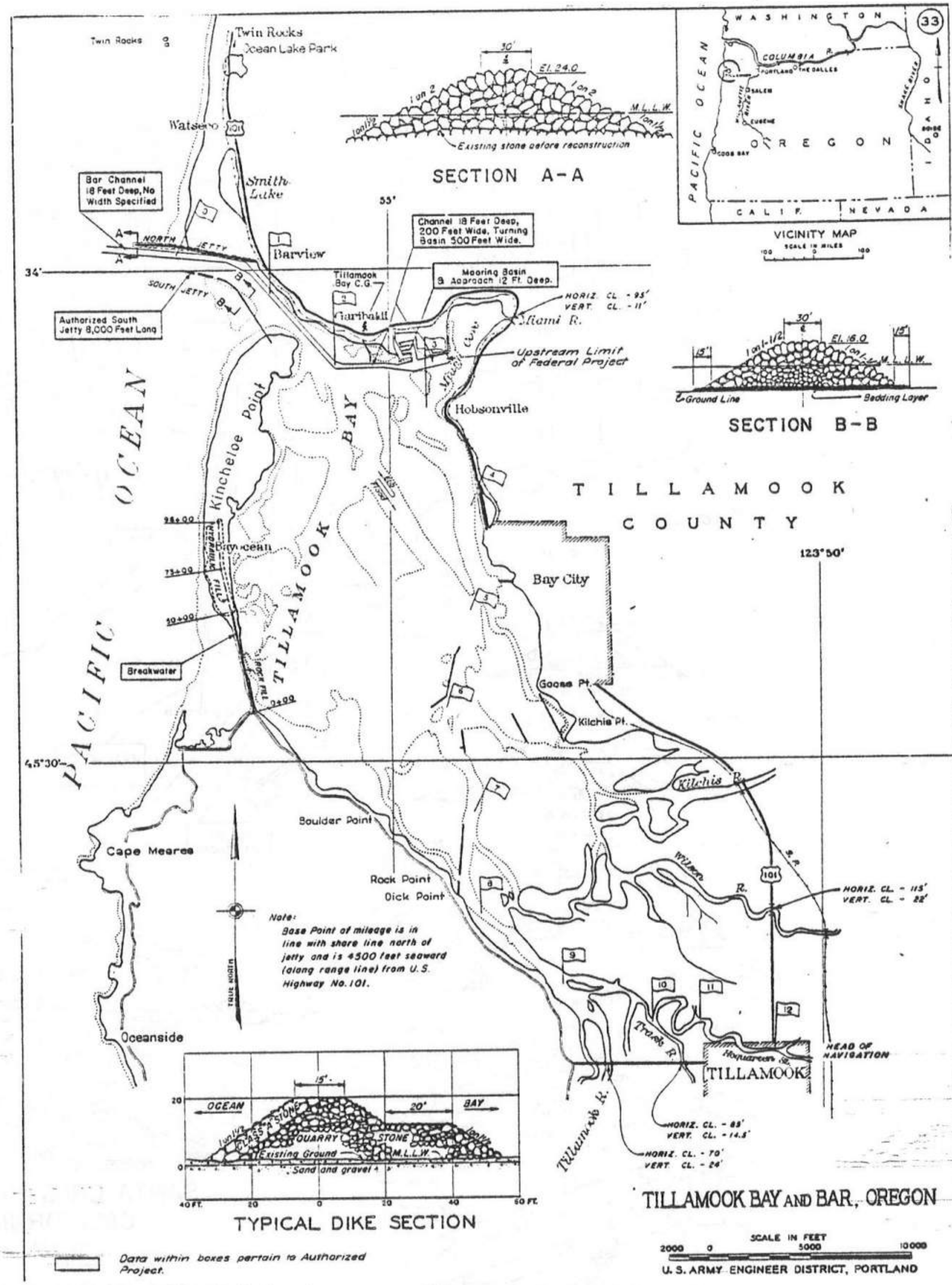


Fig.2.9 Breakwater at Tillamook Bay and Bar, Oregon

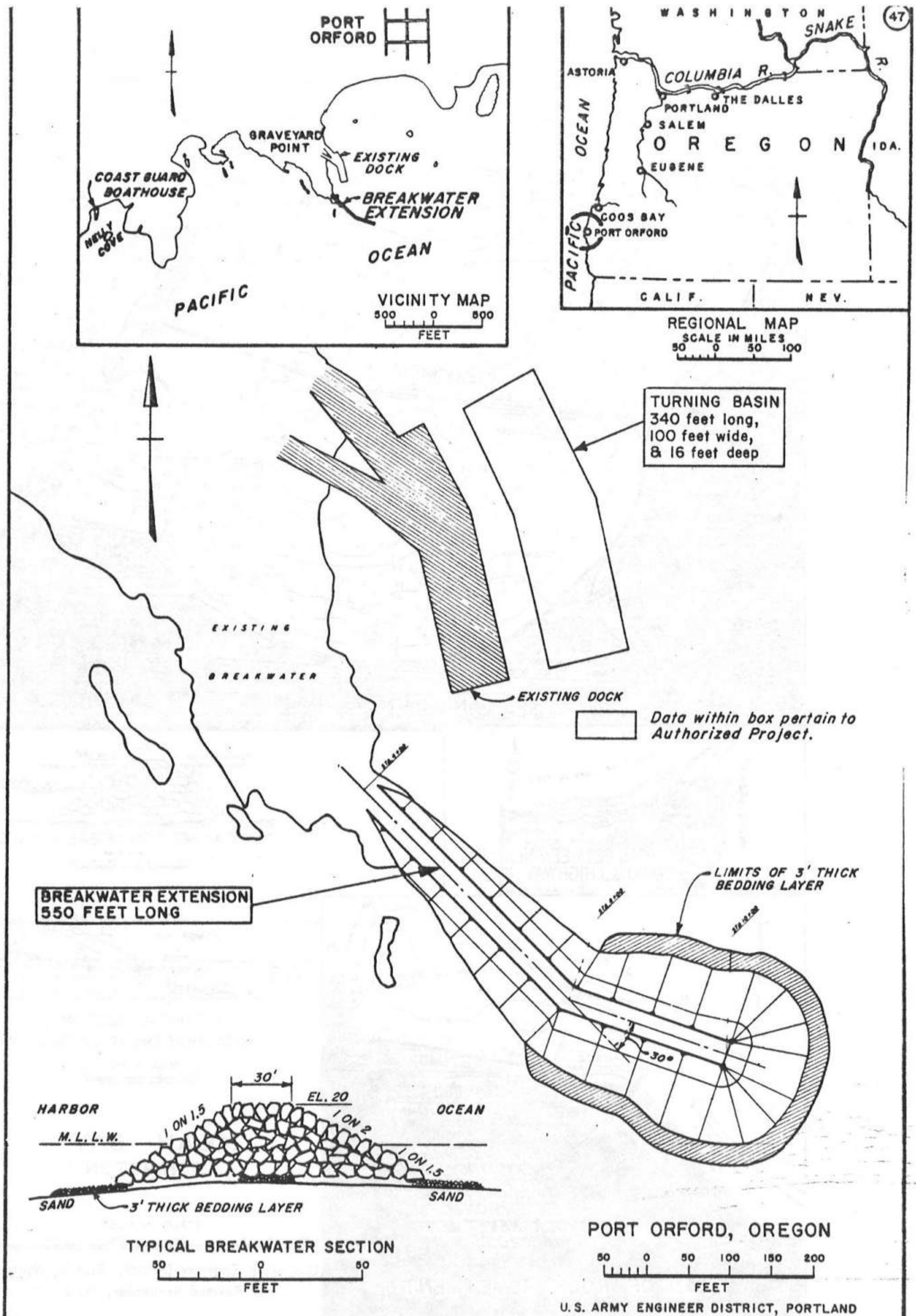


Fig. 2.10 Breakwater at Port Orford, Oregon

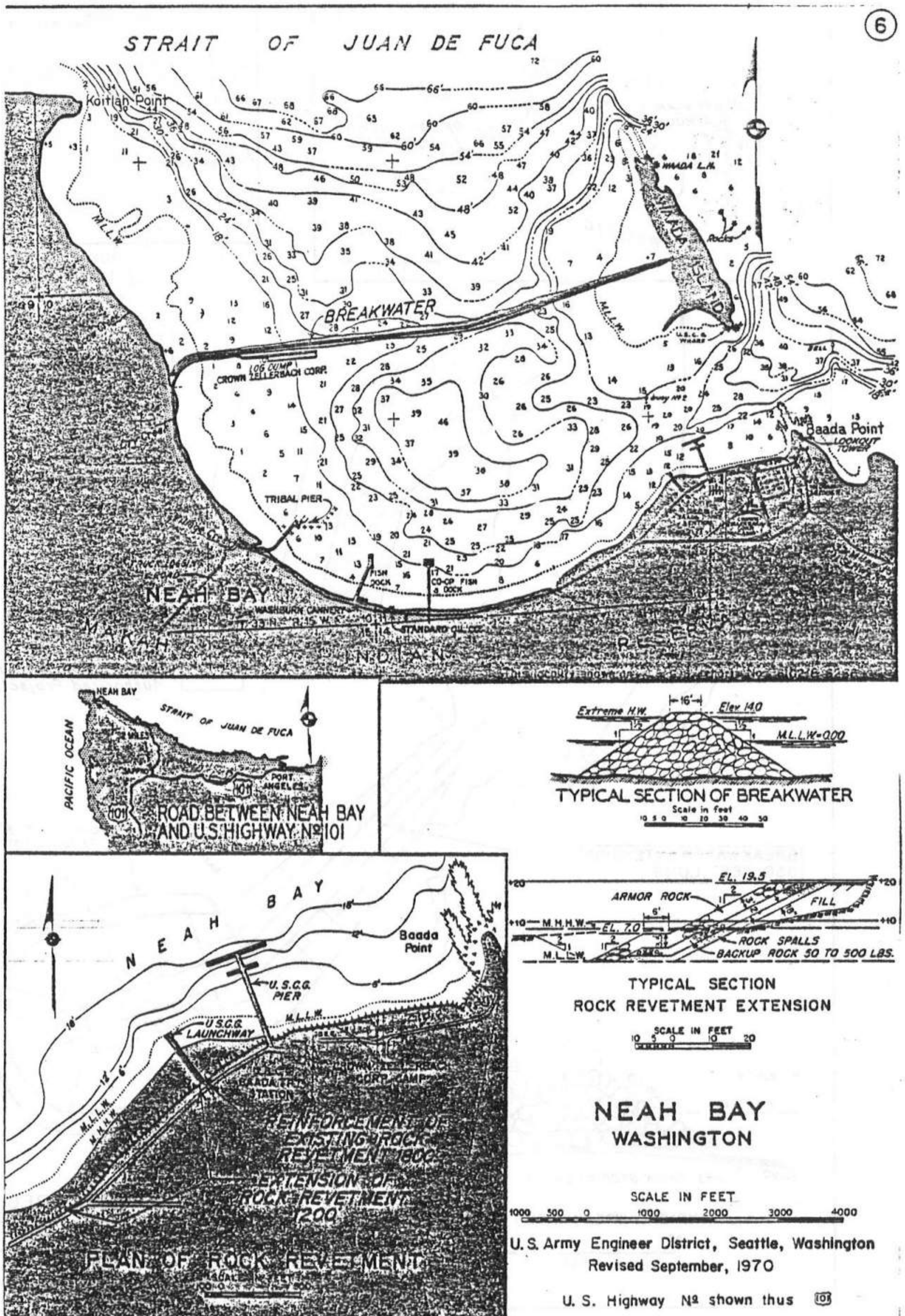


Fig. 2.11 Breakwater at Neah Bay, Washington

The jetties on the Atlantic Coast are at Fernandina Harbor, Jacksonville Harbor, St. Augustine, Ponce de Leon Inlet, Canaveral Harbor, Ft. Pierce Harbor, Palm Beach Harbor, Port Everglades, Bakers Haulover, and Miami Harbor. The jetties on the Gulf of Mexico are at Venice Inlet.

In the Gulf area, only a few jetties were constructed and no major breakwaters. These jetties existed at Southwest Pass, Plaquemines Parish, Calcasieu River and Pass, etc. No major damage has been reported.

2.2 Major Breakwaters in Europe and Africa

Breakwater design information is rather scattered in Europe. Complete design data is rarely disclosed. Through the efforts of a few researchers (in particular, Ref. 5), relevant facts on a number of breakwaters constructed prior to the 1950's have been documented including some cases of failures. Information on breakwaters of recent construction can only be found from various documents published mainly in Dock and Harbor Authorities, International Navigational Congress, Journals of Civil Engineering, Proceedings of Coastal Engineering Conference, and in reports of a few laboratories in different nations. In this section brief accounts on a few of representative cases are given.

A. Breakwaters at Catania, Italy and Algiers, Algeria

In February 1934 a storm of exceptional severity destroyed a length of 401 m. of newly constructed breakwaters at the port of Algiers. In the previous year a storm brought about the collapse of 700 m. of a similar work at the port of Catania. Both of these breakwaters were of the same type, consisting of a vertical wall of superimposed blocks based on a foundation of rubble stone, and the double disaster, following only a few years after a catastrophe of the same nature at Antofagasta, Chile, naturally caused misgivings as to the stability and trustworthiness of such vertical-wall breakwaters - particularly in Italy, where a considerable number of harbors were protected by works of this kind.

It must be remarked that although the breakwaters that failed at Catania and Algiers were of similar design and calibre, the failures were brought about in entirely different ways, due to an essential difference in construction. Both breakwaters were built of massive concrete blocks of cyclopean proportions, in the case of Catania, 12 m. long by 4 m. by 3.25 m., and in the case of Algiers, 11 m. by about 4 m. square. These blocks, weighing respectively 320 tons and 400 tons, were set as headers transversely in the moles, their ends forming the inner and outer faces of the wall, but whereas the blocks at Catania were simply superimposed without bedding or bonding, those at Algiers were provided with internal hollow shafts or wells which, on completion of the wall to full height, were filled with concrete reinforced by steel bars, so as to form a coherent structure from base to coping (Fig.2.12). As might be not altogether unexpected under such conditions, the mole at Catania failed by the sliding of the blocks over one another in successive courses as indicated in Fig. 2.13, which give typical sections; whereas the jetty at Algiers collapsed as a whole, i.e. in intact vertical sections, before fracturing and disintegrating, the rubble mound being first undermined through wave action and the erosion of a deep trench in the soft sea-bed of sand and mud at the foot of the wall. The conditions produced at Algiers are shown by the typical diagrams in Fig.2.14.

At Catania the storm of 26th March, 1933 came from the E.N.E., the fetch being 252 marine miles, the maximum depth of the sea therein being about 2,000 metres, while the maximum height of the storm waves was 7.40 metres, their length being 228 metres with a period of 12 seconds. These waves commenced to break on the adjoining slope of the foreshore in a depth of 10 metres.

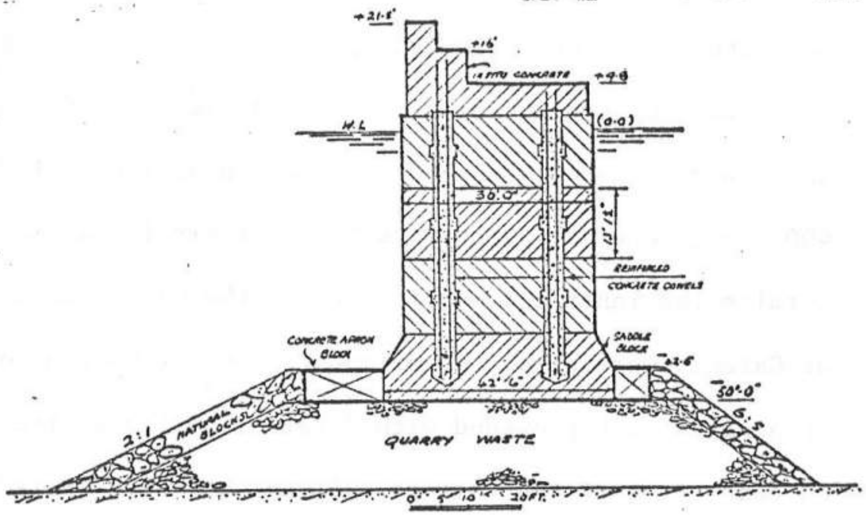


Fig. 2.12 Mustapha Jetty at Algiers

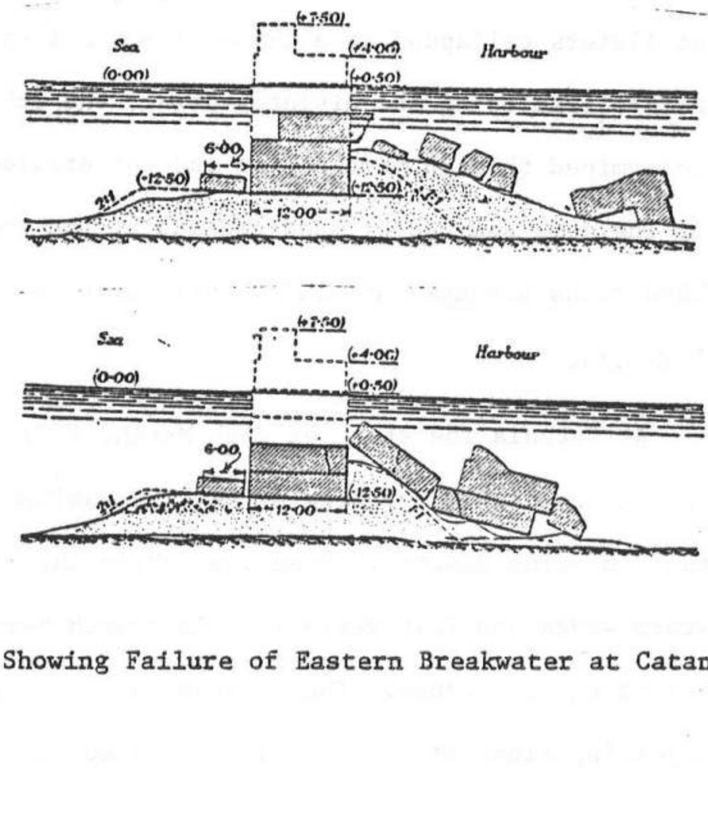
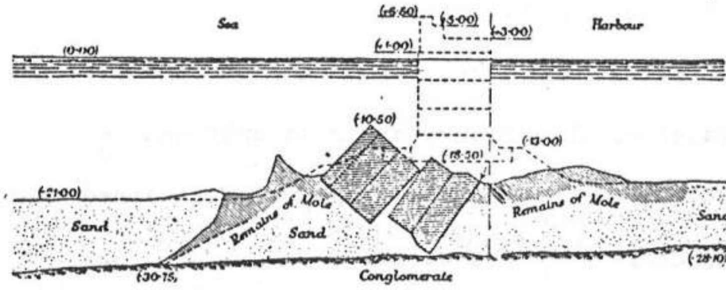


Fig. 2.13 Sections Showing Failure of Eastern Breakwater at Catania



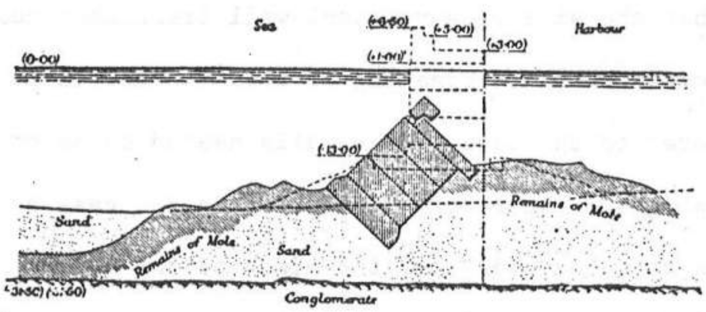


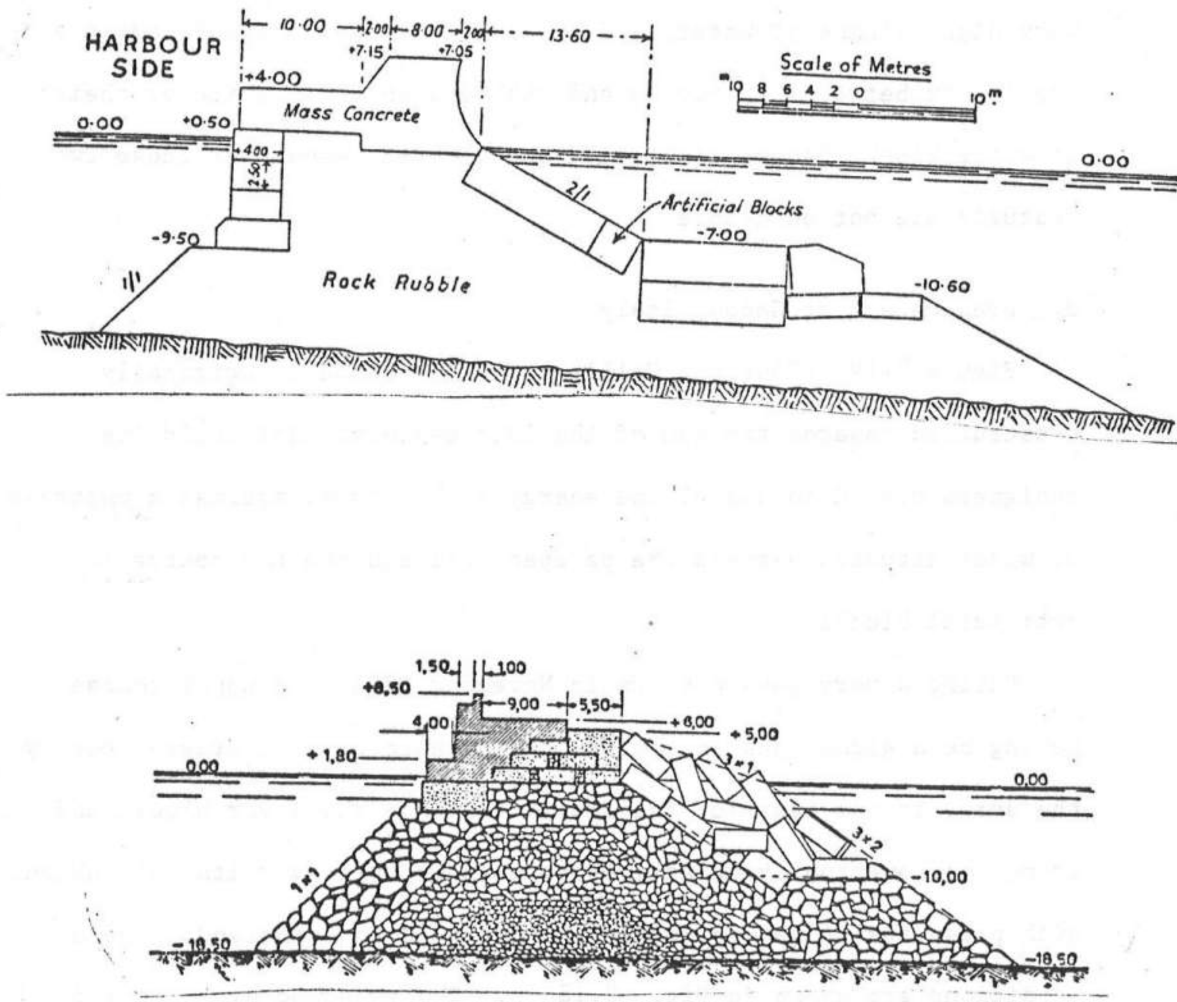
Fig. 2.14 Sections Showing Failure of Jetty at Algiers

At Algiers the storm which caused the disaster there occurred on 3rd February, 1934, with waves of a height of 9 m. and length of 185 m. in the immediate vicinity of the breakwater. The storm originated from the quarter of the compass in which the waves reaching Algiers have the maximum possible fetch, namely, 480 nautical miles, and they reached the wall normally to it. The sea-bed at the site was a very compact bed of fine sand overlying conglomerate of a stable character.

Photographs taken during the storm showed waves passing over the parapet of the breakwater in an unbroken crest. Thus it is evident that, as also at Catania, the waves retained their oscillatory character up to the wall.

The incidents at Catania and Algiers therefore seemed to indicate certain defects which could be remedied without materially affecting the accepted type. It was at once realized, for example, that the site of a vertical-wall breakwater must not be conducive to the complete breaking of storm-waves; in other words the depth of water to the foot of the walls needed to be greater in order to safeguard the rubble mound, and in the case of existing breakwaters, as at Genoa and Bari, the width of the berm or ledge at the top of the mound would need to be increased and the side slopes less steeply inclined

The reconstruction of the Catania breakwater was carried out in accordance with the type shown in Fig.2.15 ; the two courses of cyclopean blocks that had been carried away were left in place, and on the summit of the upper course at the level of - 7.00 a slope



Figures 2.15--Reconstructed Breakwaters at Catania

of 2/1 was constructed, having a revetment formed by other recovered blocks. At sea-level a parapet was built in mass concrete having a parabolic face, a form originally adopted in the Roman epoch at the ancient mole of Pandateria on the island of Ventotene. The smooth slope, combined with the concave profile, led to the formation of very high columns of water, and these, in falling, caused damage at the joints between the blocks and rubble slope. In spite of their size the blocks became displaced, and thus it seems that these two features are not advisable.

B. Breakwaters at Genoa, Italy

Figure 2.16 shows the Galliera Mole at Genoa as originally constructed towards the end of the last century. Evidently its designers wished to dispel the energy of the waves against a mattress of water situated between the parapet wall and the top course of artificial blocks.

During a very severe storm in November 1898, the upper course having been either pushed towards the parapet wall or dragged out by the surf, it was easy for the sea to displace the lower blocks and rubble stone and eventually to strike the parapet wall over its full height with pressures it was obviously not designed to withstand. These conditions are shown in Fig. 2.18 . The repaired breakwater is shown in Fig. 2.17 , the principal amendment being the construction of a heavy concrete revetment over the whole width of 17 m. of the berm. The underwater slope of the rubble mound, which had become flattened from $1\frac{1}{2}/1$ to 2/1 during the storm, seemed to be stable at that slope, which was accordingly adopted. The height of the parapet wall was reduced by raising the level of the berm.

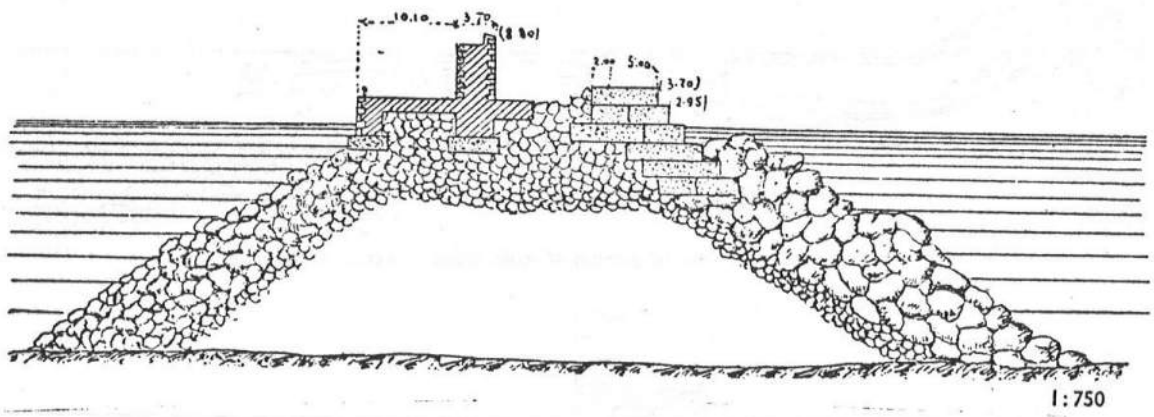


Fig. 2.16 Original Form of Galliera Mole, Genoa

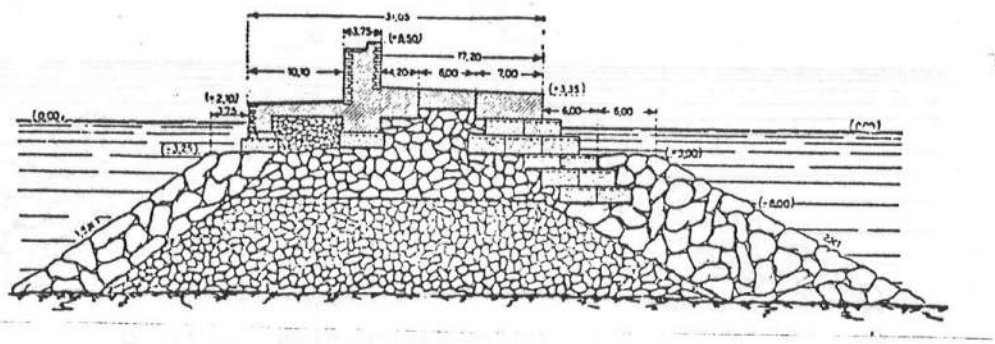
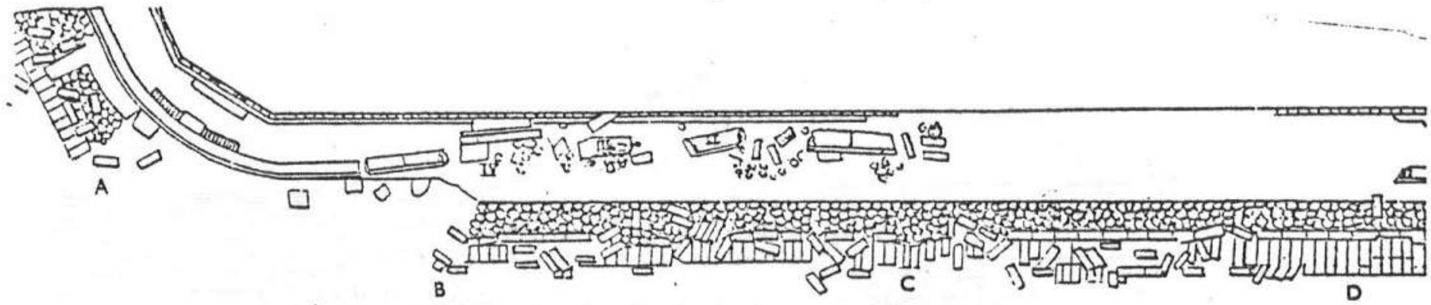
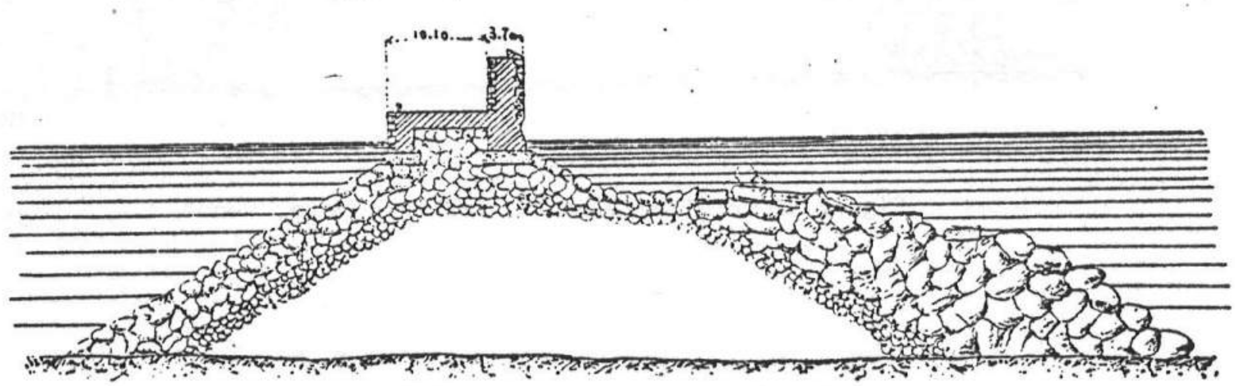


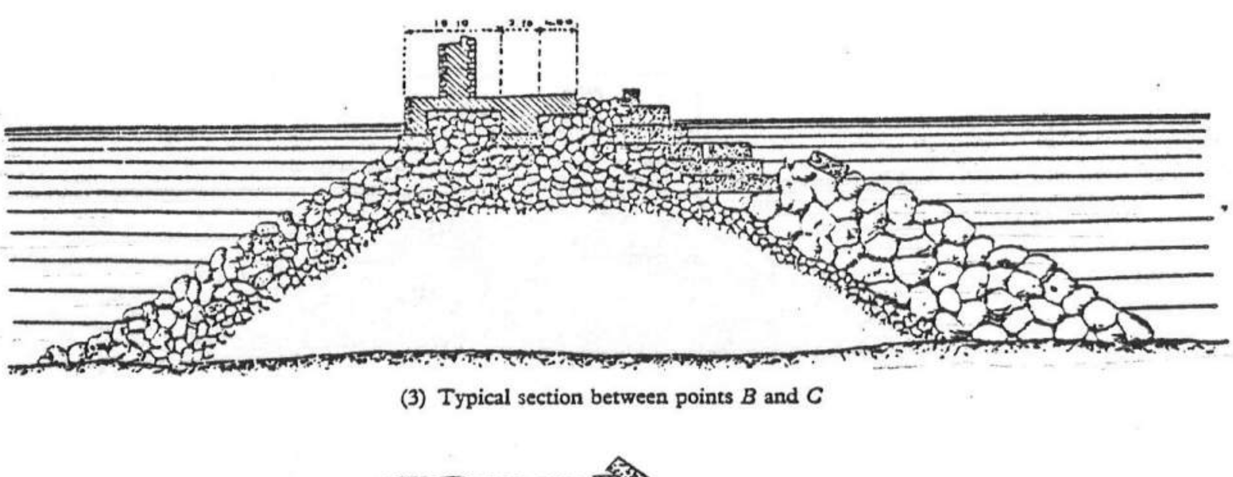
Fig. 2.17 Typical Profile of Repaired Galliera Mole, Composite Type



(1) Plan of portion of mole, showing extent of damage



(2) Typical section between points *A* and *B*



(3) Typical section between points *B* and *C*

(4) Typical section between points *C* and *D*

Fig. 2.18 The Galliera Mole, Genoa, Showing Damage Due to Storm of 26th-27th November, 1898

As a result of further violent storms, the breakwater still suffered heavy damage, whole lengths of the stepped blocks being swept away. These were then replaced by mass concrete from the level of - 3.00 to + 3.35, the face having a slope of 2/1. Even with these alterations the breakwater continued to suffer damage; consequently it was decided that the profile of the breakwater should be altered in accordance with the lines that were followed in the reconstruction of the outer breakwater at the harbour of Crotone in the years following World War II (see later pages).

The last part of the outer breakwater, of more recent construction and 3,869 m. in length, is of several types. That first adopted consisted of a vertical wall built with three courses of cellular concrete blocks (a design later altered to four, with thicker walls), surmounted by a mass-concrete superstructure and standing upon a rubble mound; the amended design is shown in Fig. 2.19 . This breakwater is exposed to the prevailing winds from S.W. to S.E.; the most violent storms come from the S.W., where the fetch extends to the coast of Algeria, a distance of about 700 miles, and during these storms the height of the waves is observed to be 5 m. For the construction of the last section of extension to the west, the design of the breakwater was again amended as shown in Fig. 2.20 , the wall being founded at a level of - 11.50 and constructed of blocks of concrete 12 m. by 4.50 m. by 2.95 m., each weighing 420 tons, with the base protected by blocks forming a berm 5 m. wide.

In spite of this strengthening, the breaking of waves during the most violent storms was apparent, even if they caused no damage to the breakwater, which, on this sector, remained intact until the storm of 1955.

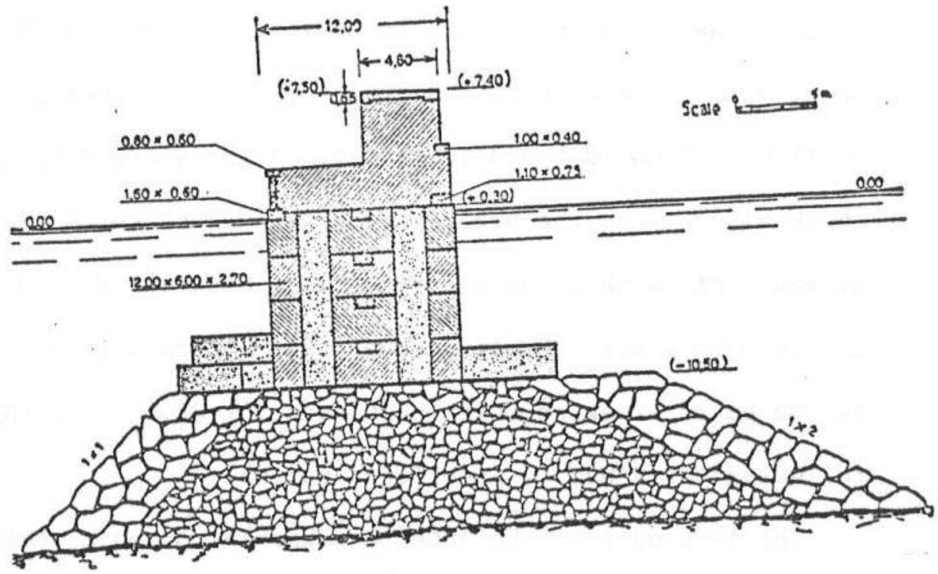


Fig. 2.19 Section of Outer Breakwater, Genoa (Design in well-type blocks)

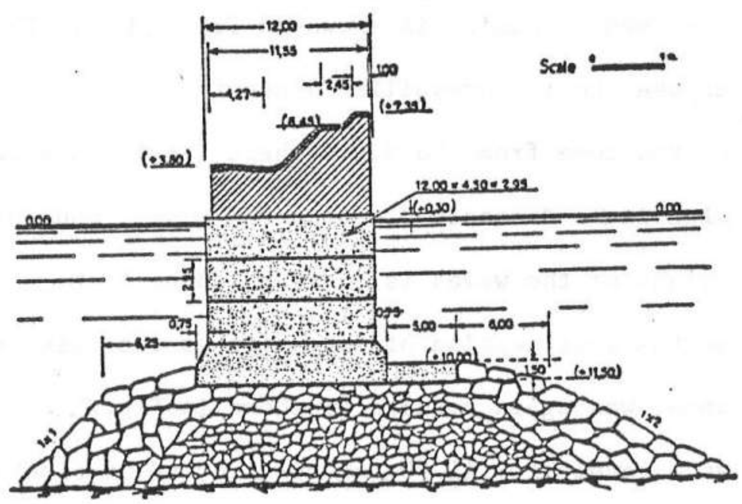


Fig. 2.20 Section of Western Extension of Outer Breakwater; Genoa (Design in Cyclopean Blocks)

A great storm occurred at Genoa on 19th February, 1955, causing considerable damage to the exterior breakwater and to ships in the harbor. Full data are not available, although from the information published in the technical press it appears that sections of each type of the outer breakwater were severely damaged or destroyed, with the significant exception of the oldest part of the Galliera Mole, which was of the composite rubble mound and superstructure type. (Fig.2.17).

The violence of the waves appeared to have been concentrated on the stretch of wall near the western entrance of the Sampierdarena Basin - where the longest breach occurred - in that zone of the sea-bed where the depth contours of the sea form a submerged valley off-shore.

The breaches varied in nature and character for the different types of cross-section of breakwater. The first serious collapse to occur was that mentioned above, when on February 19th at 15.45 hours the heavy seas which had been running for some hours overtopped and sheared off, suddenly and simultaneously, 150 metres of wall of the cross-section shown in Fig. 2.20. This gap widened continuously until it reached 450 m. in length. Later examination by divers showed that the top course of blocks and the mass-concrete superstructure had been cleanly slid into the harbour, the lower three courses of solid blocks and the apron block at the toe remaining in place for the most part undisturbed and undamaged. Two hours later, on the same day with the seas still running high, a first small breach was opened in the east extension of the Galliera Mole, which was constructed of well-type blocks (Fig.2.19), and during the night and following day several more gaps of from 5-80 meters were opened in the same stretch and also in the mole west of the Galliera composite mole, which was constructed of three tiers of cellular-type blocks.

The cellular blocks were not sheared off like the cyclopean blocks because they were held firmly by the central cores of concrete which formed the fill of the wells of the precast skeleton cellular blocks. Instead, the outer faces of both three- and four-course types were shattered and broken up gradually at many points under the heavy blows of the waves. This provoked the undermining but not collapse of the superstructure and the breaking-up of the adjacent blocks and concrete below. It is remarkable that the first damage caused by the storm, and the most extensive in length and depth, was in the most modern and normally least troublesome stretch of the breakwater.

The gale appears to have been the product of a very strong southwest wind and was of a short violent ascending phase and a long descending period which aggravated the damage produced at the peak of the storm. On the morning of the 18th of February the sea had been tempestuous; next morning it seemed to be dying down but then, the gale suddenly backing to the south, it became extremely violent. Such a storm had never before been experienced in living memory at Genoa.

The barometer stood at 739.2 mm. (29.1 in.) and the velocity of the wind in squalls rose to 123 kilometres per hour (76 mph). The waves were most irregular, at one time apparently coming from the dominant S.W., then from the S.E., then a combination of both; their height could not be accurately measured, but those competent to judge computed them conservatively at not less than 7 m. with the possibility of their being higher. They appear to have definitely broken at the walls, throwing up columns of water of great height, some of which, from documentary photographs, were later calculated to have reached 50 m.

It has been mentioned that cellular and well-type walls had been under suspicion for many years, and that where they had failed previously they had been replaced (1926-1927) by the cyclopean block type of wall. This type had withstood all storms successfully, with the exception of minor fractures in the winter of 1953-1954, until seriously damaged by the storm just referred to. All these walls had been built, of course, as reflecting breakwaters, but a combination of reflected and oncoming sea-waves creates clapotis; this condition causes such waves to attain about twice their original height, with great dynamic and static power.

These incidents at Genoa seem to provide another example of the occurrence of meteorological and storm conditions far in excess of estimates.

The main lessons to be learned from this occurrence, together with other disasters that have occurred with vertical-wall types of breakwater elsewhere, are first, that the basis of the present methods of estimation of maximum wave-height and other wave characteristics for design purposes need reviewing; secondly, that the design of such walls to induce free reflection of waves from the vertical face is a delicate experiment with natural forces; thirdly, that technical knowledge of the dynamic effects of the breaking wave is still inadequate; and fourthly, that a combination of partial reflection and energy dissipation at the structure is very dangerous and definitely to be avoided.

It now seems that Italian opinion upon the merits of vertical-wall breakwaters, due to continued failures at various ports, has been modified, for experience shows that the rubble mound, composite, or talus type have given the best results.

C. Breakwaters at Crotone, Italy

The outer breakwater has a total length of 835 m. divided into two arms of which one, rooted in the coastline, has a length of 435 m. in a S.S.W. to N.N.E. direction, while the second, 400 m. in length, is slightly inclined to the north. The sea-bed is sandy and is at a depth of 0.00 at the heel and -16.00 at the head. The breakwater is exposed to waves which come from an E.S.E. direction with a very long fetch and may reach a height of 6 m. with a length of 120 m.

The construction of the breakwater, of a composite type, was commenced some sixty years ago, but the rubble mound and the superstructure were not massive enough. In consequence the breakwater suffered continual damage until finally, in 1941, it was completely destroyed. The reconstruction was carried out as shown in Fig. . The outer slope of the mound is protected by a covering of large blocks having an incline, above water, of 3 to 1 and below water of 3 to 2. The placing of the blocks was done in such a manner that while the surface of the slope, owing to its irregularity, offered a very pronounced friction to the water masses in movement, it contained no sudden projections or deep recesses which lead to the formation of columns of water and cause upward pressures. Along the first leg of the breakwater there is an upper parapet in mass concrete which continues the incline of the 3 to 1 slope terminating with a 1 to 1 slope - not a parabolic profile. Along the second leg this parapet has not yet been constructed for reasons of economy.

The breakwater has withstood, without suffering any damage, several storms of violence comparable with that which destroyed the first structure.

D. Breakwaters at Marseille, France

The main undertaking, begun in 1845, has a length at the present time of 5,000 yards, including an extension for the President Wilson Basin. The same principle of construction has been maintained through more than a century with unvarying success.

A section of the main breakwater, the Grande Jetee, is shown in Fig.2.21 . The core is a bed of small rubble having a depth or thickness of 10 ft., and lying upon the sea bottom at a depth of 55 ft. below low-water level. It is overlain by layers of natural stone of increasing dimensions, ranging from 2 cwt. to nearly 4 tons apiece. The quay shelter wall is a masonry structure founded upon the topmost layer of blocks.

The exterior slope is 4 to 3 for its lower portion, extending from the foundation to low-water level. At this point it flattens abruptly to nearly 3 to 1. The effect of this sudden transition is to create a sharp ridge at the water-line, with the result that the waves are cut at the point where their action is most potent. The upper part of a wave, therefore, falls dead upon the flat slope above, or at the worst upon the masonry apron in front of the shelter wall, in neither case being capable of producing deleterious results. The parapet thus receives no appreciable shock, and spray alone passes at times over its crest to fall upon the interior quay.

The blocks forming the flattened slope referred to are huge monoliths, rectangular in shape, with a length twice as great as their width, having a volume of 500 cu. ft. and a weight of about 33 tons apiece. They are deposited so as to lie longitudinally in the direction of the onset of the waves.

The external profile of the breakwater has proved to be extremely stable and has been kept up at trifling expense in the way of repairs. For a length of 1,200 yards constructed prior to 1865 the cost of maintenance is still very little. The remaining and later portions of the breakwater also cost practically nothing for upkeep.

Thus the Grande Jetee is a most efficient example of its type. Only one objection can be laid against the design, and that is the narrowness of the uppermost outer slope flanking the masonry apron. The existing width of 27 ft. seems to be insufficient to prevent the protection blocks from being occasionally rolled off by the waves into deep water.

Settlements in the mass of the breakwater, though they have been by no means inconsiderable in themselves, appear not to have given rise to any serious dislocation of the parapet wall. Indeed it is said that only here and there can a few vertical cracks be observed, having widths of mere fractions of an inch. The shelter wall and its apron are not bonded together: they are simply in contiguity. Separation was inevitable since they rest upon distinctly different foundations.

E. Alderney Breakwater, England

The composite breakwater at Alderney Harbor which has the general cross-section shown in Fig. 2.22 , offers an interesting comparison to the breakwater at Catania for it seems to have failed through

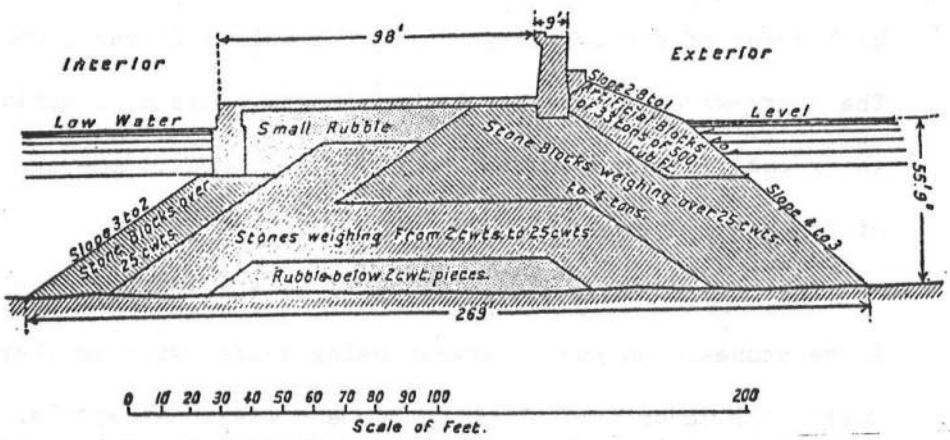


Figure 2.21--Section of Grande Jetée, Marseille

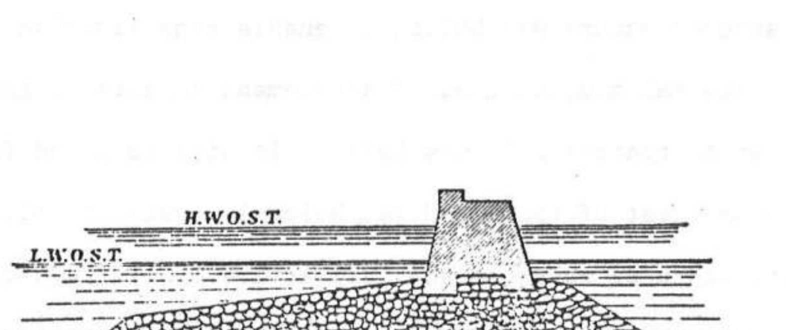


Figure 2.22--Typical cross-section of Alderney Breakwater

almost precisely similar causes - disruption of parts of the structure by falling masses of water.

The site of the breakwater is exposed to Atlantic gales with an unlimited fetch; the tidal range is 17 ft. and the depth of water at high water of spring tides, where the wall collapsed, was 130 ft. The storm-waves of any height and length could manifestly maintain their oscillatory character until quite close to the superstructure of the breakwater.

The rubble mound was constructed by first depositing two banks of large stones, the space between being filled with smaller graded rubble, during the deposit of which settlement was considerable. Heavy seas that occurred during construction caused little disturbance of the rubble when the top of the mound was 15 ft. or more below low-water level with no superstructure upon it. Three years elapsed before the superstructure was built, to enable consolidation and settlement to reach its maximum,but unequal settlement appears to have occurred after the superstructure wall was built. It will be noted from the figure that the crest of the mound was below low-water level; actually there was 11 ft. depth of water at the wall at L.W.O.S.T., and 27 ft. at H.W.O.S.T. The upper slope of the mound was such that in effect a shoal was formed about 100 ft. from the wall, at which distance the water was some 7 ft. deeper. This depth of water meant that the waves - oscillatory up to this point - were converted into a succession of breakers, the impact of which on the wall caused masses of water to be projected to a considerable height, recorded to have been, at times, upwards of 200 ft.

The blows of the waves dislodged some of the masonry blocks of the superstructure wall, and the masses of water, falling from the great height to which they had been projected, tore out the rubble at the base of the wall, the bottom courses of which dropped down and enabled the sea-waves to reach and bring out the rubble hearting, and within a short time to reach the inner portion of the wall which was soon breached.

Four engineers of repute investigated the causes of the failure of the breakwater, and the above remarks are based upon their reports. The profile of both the rubble mound and the seaward face of the walls were criticized, and one of the main recommendations for reconstruction was the raising of the slope of the mound, so that at the wall it reached the level of high water of spring tides.

In the light of modern knowledge and practice, the defects of this breakwater would probably be reckoned to be:

a) The profile of the mound and the depth of water arising from it were ill-chosen, i.e. the latter was too shallow, causing waves to break the wall. If the depth had been greater, the oscillatory pattern of the wave would have been retained, resulting in a "clapotis" at the wall. If on the other hand the rubble mound had been higher and above high-water level at the wall, and the slope had been wider, all waves would have been forced to break before reaching the wall.

b) The sizes of the stones of the outer covering of the rubble mound were too small.

c) The dimensions of the blocks of stone with which the superstructure walls were constructed were also too small, and the foundation level in the mound was not deep enough.

d) The covering of the roadway was not adequately robust.

F. Leixoes Harbor, Portugal

The port of Leixoes, the artificial harbor of Oporto, is situated at the mouth of River Douro (Fig. 2.23). Two old breakwaters, the northern and the southern, were built between 1883 and 1892.

The construction of the new shelter-breakwater was commenced in 1933 by building the first 330 m. of the planned 1,000 m. long breakwater in the form of a vertical wall founded on the rock below the sea bottom. During the work a storm of exceptional severity ruined part of the structure. It was decided that with the depth of the water and the great exposure of the site, a vertical wall would not be stable. In 1936, laboratory studies came up with the recommendation of using a submerged breakwater of rock-fill for the 678 m. of the most exposed part of the structure. The cross-sections of the two tvpes of breakwaters finally adopted are shown in Figs. 2.24 and 2.25 .

The prevailing winds at Leixoes come from the quarter lying roughly between N.W. and S.W., and it is from this region that the greatest storms arrive from the Atlantic with its almost unlimited fetch. Actually, waves of the following characteristics have been recorded at the harbor: height 8 m. and length 350 m., with periods of 10 to 15 seconds. Based on model studies, 80-ton blocks for the outer covering and the inclusion of berms seen in Fig. 2.24, were recommended.

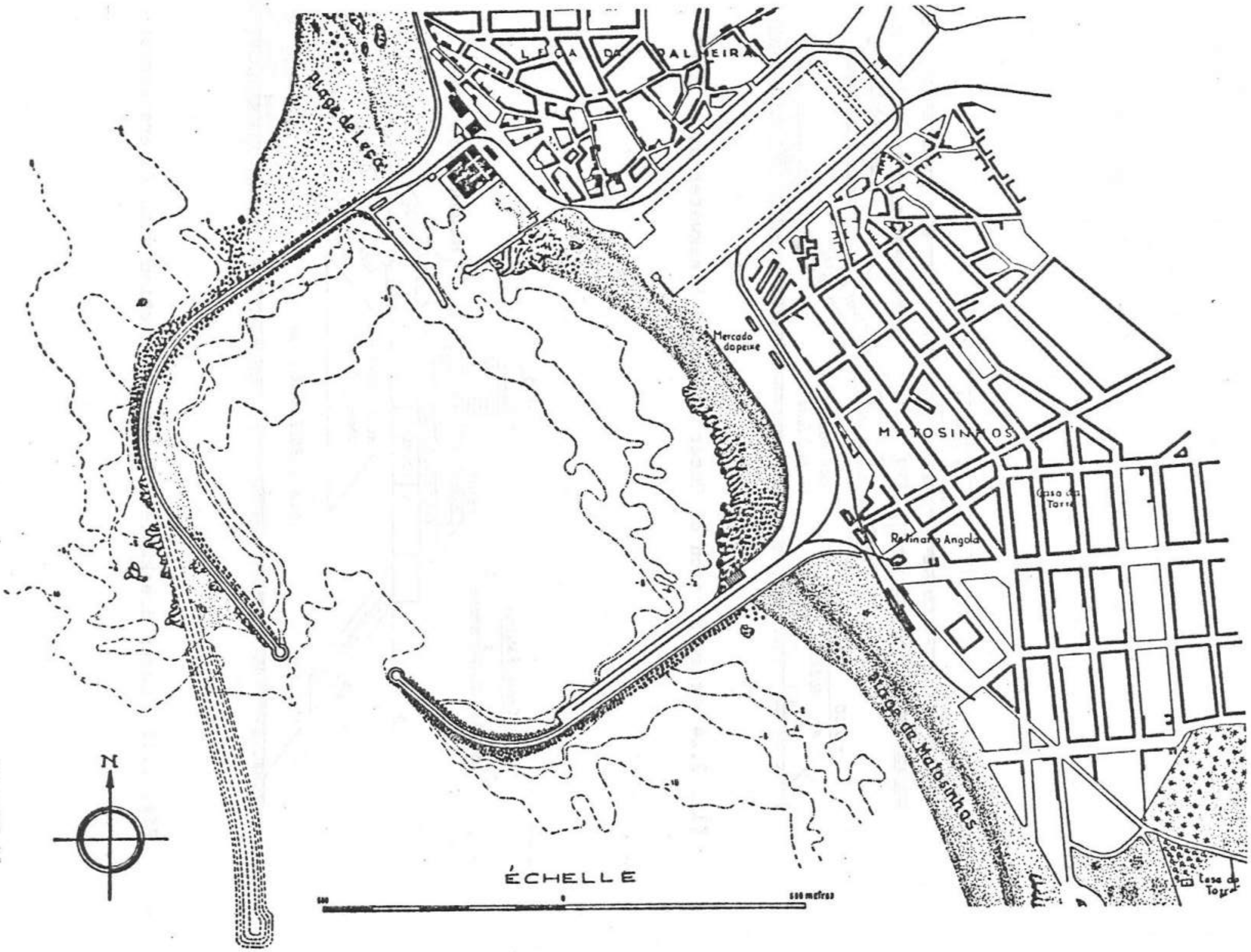


Figure 2.23--The Harbour of Leixões

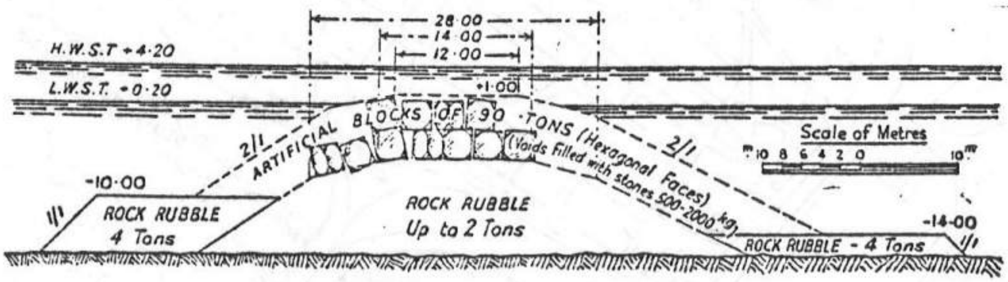


Fig. 2.24 Cross Section of Outer Submerged Breakwater at Leixoes

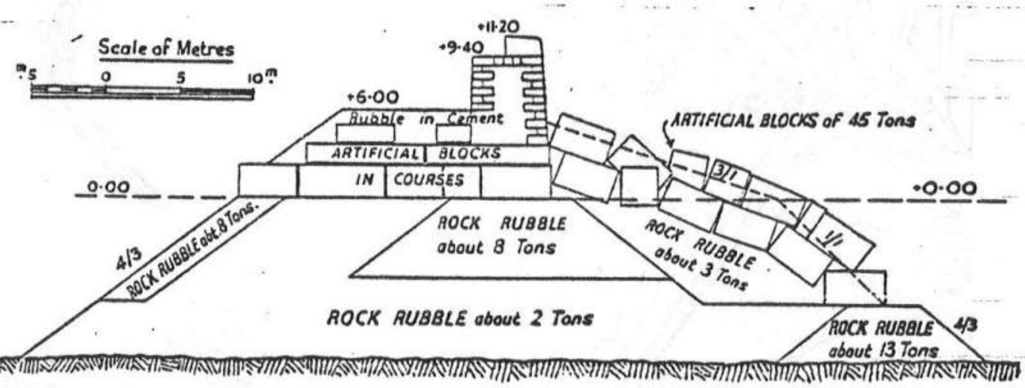


Fig. 2.25 Outer Breakwater, Leixoes: Portion with Superstructure

In practice, the artificial blocks of the outer covering were made 90 tons in weight and were cubical in shape with the corners greatly chamfered.

G. Casablanca Breakwater, Morocco

At this harbor the tidal range is 3.80 m. and the coast is exposed to violent storms; waves with a height of 8 m. are not uncommon and they sometimes break in depth of water of 20 m. From Fig. 2.26 it will be seen that the artificial blocks of 45 tons in weight cover the underlying riprap entirely in pell-mell fashion, while under the superstructure they are built into position. At high tide the mass-concrete shelter wall is liable to bear a great proportion of the impact of breaking waves, though it is sufficiently low in height to allow waves to go over the top in exceptional storms. During such a storm in 1924 parts of the shelter wall weighing some 260 tons were thrust inside the harbor, but the main body of the breakwater stood intact.

H. Dunkirk Breakwaters, France

The western arm of the two harbor breakwaters faces roughly N.W., while the eastern arm faces N.E., the entrance between the pierheads facing due north. While the maximum fetch is in a northerly direction, the shoals off the shore and the Goodwin Sands reduce the violence of any gales from the north and northeast. The shortest fetch is in a northwesterly direction, but owing to severe gales from the Atlantic Ocean advancing up the English Channel and a not-infrequent change of wind to the northwest it is from this direction that the most violent seas arrive.

Two forms of breakwater are in existence, shown in Fig.2.27 and 2.28 . The first mentioned was employed for some 700 ft. from the roots of the breakwaters, and the latter was used in deep water. The sea-bed is composed of sand, and in view of the strong possibility of scour being caused by tidal and littoral currents due to the projection of the breakwater into the sea, a feature of the design of the mounds was the use of brushwood fascine work in order to form a mattress 6 to 7 ft. thick upon which the rubble mounds were deposited.

The mattresses were constructed ashore of elm and birch branches made into faggots about 12 in. diameter and 50 ft. long, placed in layers, checkerboard fashion, all being lashed together. Sections about 130 ft. long by 80 ft. wide were constructed, towed to the site, and sunk by means of rubble stone deposited into the cells formed by the checkerboard construction. The sinking was accomplished by working from the end meeting the current, the portion still floating thus being kept taut.

I. Praia de Vitoriz, Portugal

Práia da Vitoria is a harbor in the eastern coast of Terceira, in the Azore Islands, Portugal. The rubble mound breakwater, about 600 m. long, extend from Ponta do Espirato southward into the Praía Bay (Fig. 2.29). Given the location and orientation of the breakwater, storms in easterly direction are the only kind that can hit the breakwater with full strength. For some unclear reason the breakwater was designed based on a design wave of 5.2 m. where local observations recorded wave height of 5.5 m. during an easterly storm in 1955 and of 5.5 to 6 m. in a storm of 1956. The typical

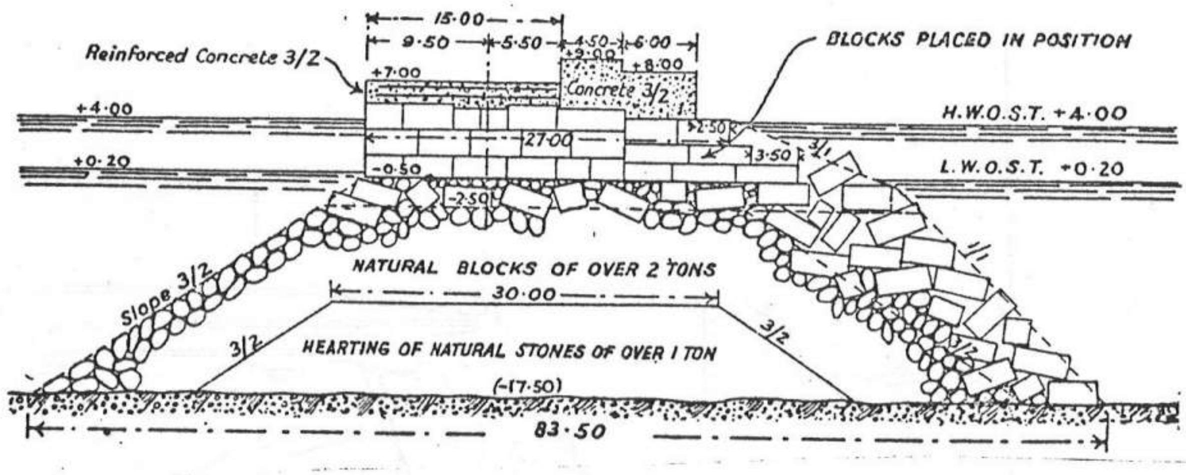


Fig. 2.26 Section of Breakwater at Casablanca

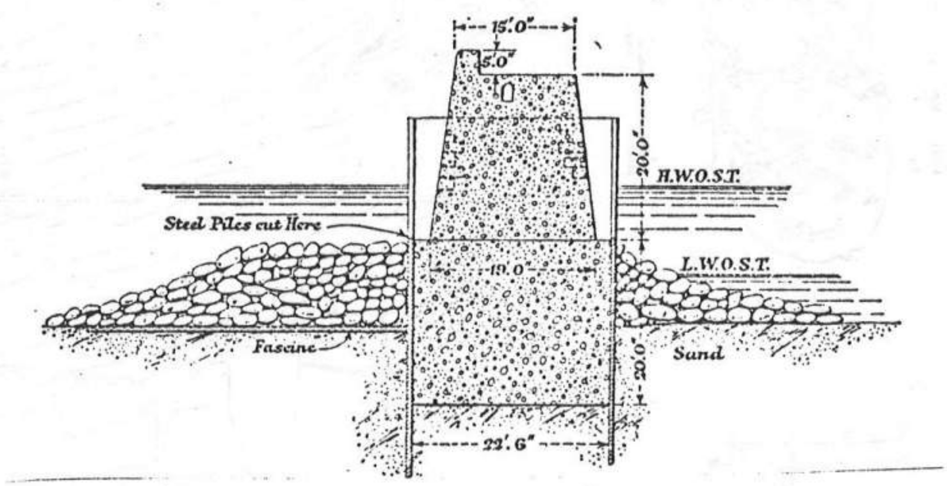


Fig. 2.27 Dunkirk Breakwater: Typical Form in Shallow Water

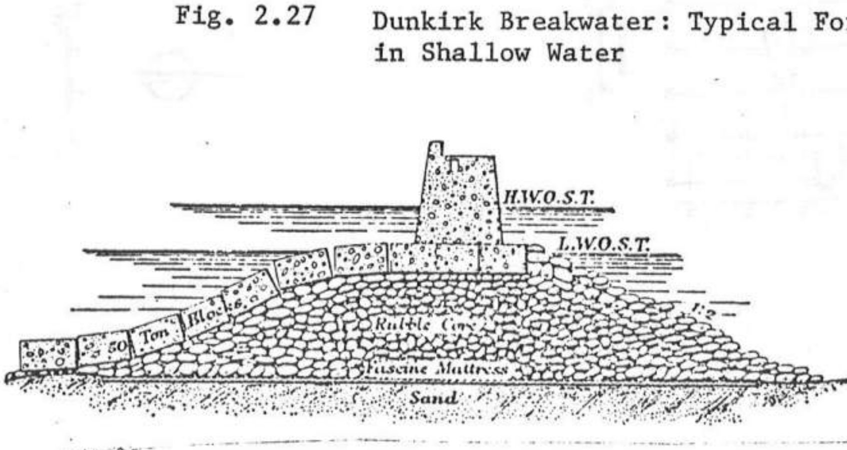


Fig. 2.28 Dunkirk Breakwater: Typical Form in Deeper Water

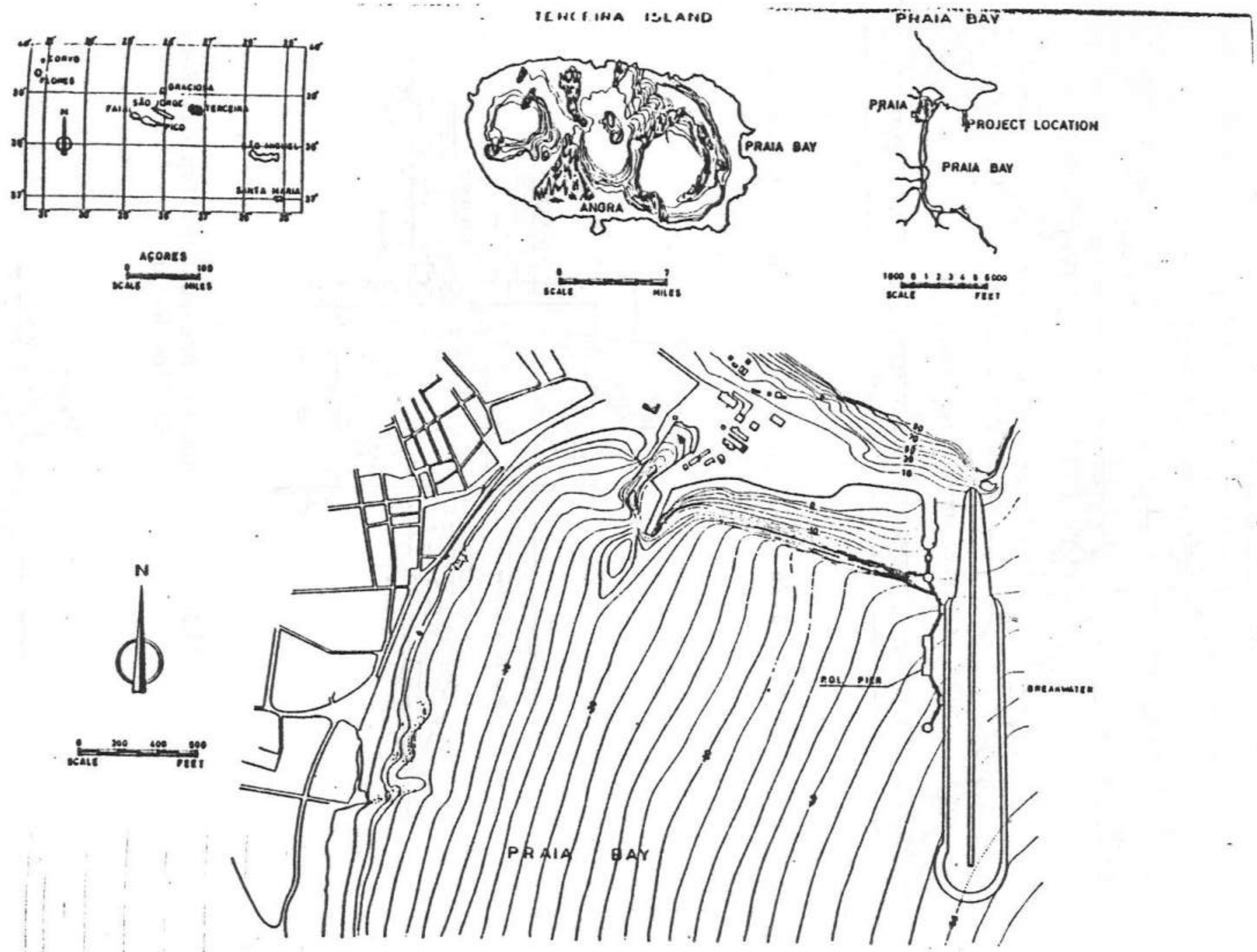


Fig. 2.29 Praia Da Vitoria Harbor: General Arrangement

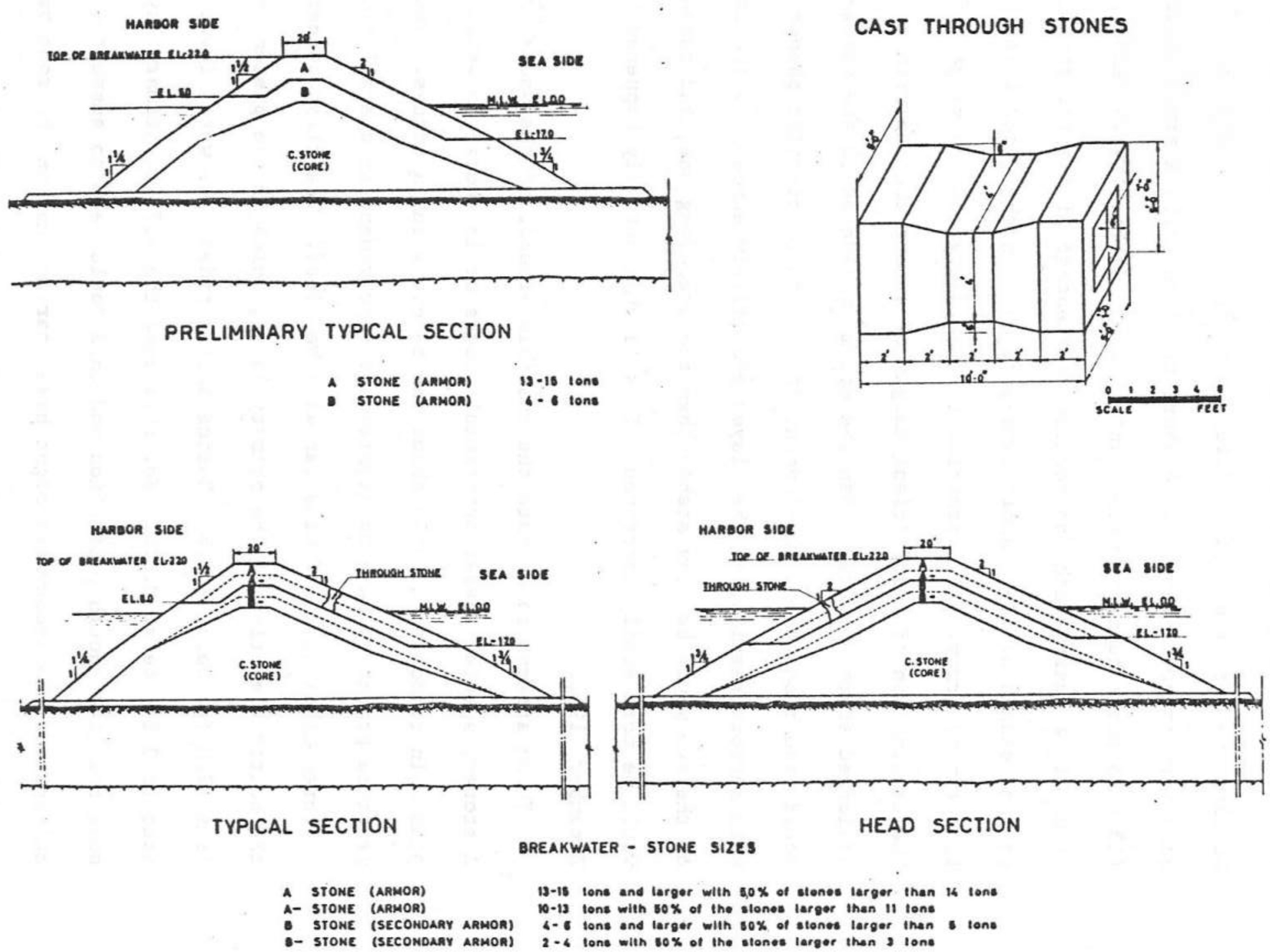


Fig. 2.30 Original and Alternate Sections

cross section based on the 5.2 m. design wave is shown in Fig. 2.30 . It consisted of type C stone core (up to 8,000 lbs.), type B secondary armor layers (4 to 6 short tons) and type A armor layers (13 - 15 short tons). During construction, difficulty was experienced to obtain a quarry with the required percentage of A armor stones, so that stones of 10-13 short tons were used in the inner layer of the primary armor. Such practice further decreased the margin of safety. The combination of insufficient margin of safety coupled with a prolonged storm, severer than the storm considered in the design, could give rise to the development of a chain destruction phenomenon with serious results, as the layer excessively exposed to the action of the sea would be less stable than the preceding one, and consequently would be more easily destroyed. This is what actually happened late in December, 1962.

In an attempt to obviate the possible effects of the undersized A stones, special shaped artificial blocks of 16 short-ton weight were placed in ramdom at a ratio about one to every ten A stones. Such a practice proved to have no significant improvement in overall stability.

Three other design faults can also be cited. The first concerns the bottom elevation of the primary layer, which in the present case lies 17.0 ft. below M.L.W. During spring tides, the water level can drop to 3 ft. below M.L.W. For this case the -17 ft. primary layer does not give enough protection and this could lead to suspect that the collapse of the breakwater might have started just in the zone between A and B armor stones. Secondly, in the harbor side the A armor stone cover extends down to +5 ft. to M.L.W., the B armor stone layer being, therefore, exposed for all the tide levels except high water spring tide. Finally, there is no provision to make the head section of the breakwater more stable than the trunk section.

The structure sustained extensive damage during a violent storm in December, 1962. According to hindcast, waves as high as 8 to 9 m. were experienced. It was reported that the storm arrived at Santa Maria island after 00 hours on 25th December and sustained through 27th with a reduced influence thereafter. The structure damage appeared to be highly irregular (i.e., sections of breakwater sustained extensive damage where other sections remained basically intact or suffered very light damage).

Based on post-storm survey, it was reconstructed that the damage began due to insufficient stability of the sea-side slope and not to overtopping. When the storm reached its maximum intensity, the damage gradually extended to the whole structure. First, the B armor stones became exposed and displaced, then the C core stones were attacked by waves. Finally, sections were broken and profiles above elevation (-0.00) were completely ruined. From this point on, the overtopping waves pounded on the harbor-side slope and caused removal of stones on that side. The destruction went on to the extent that berth structures were endangered. Fortunately, the storm abated before total collapse of the structure.

Figure 2.31 shows the breakwater sections after the storm. The irregular distribution of damages is clearly illustrated. Two factors could have caused this irregular distribution of damages: marked changes of the wave height along the breakwater and variable construction details from zone to zone.

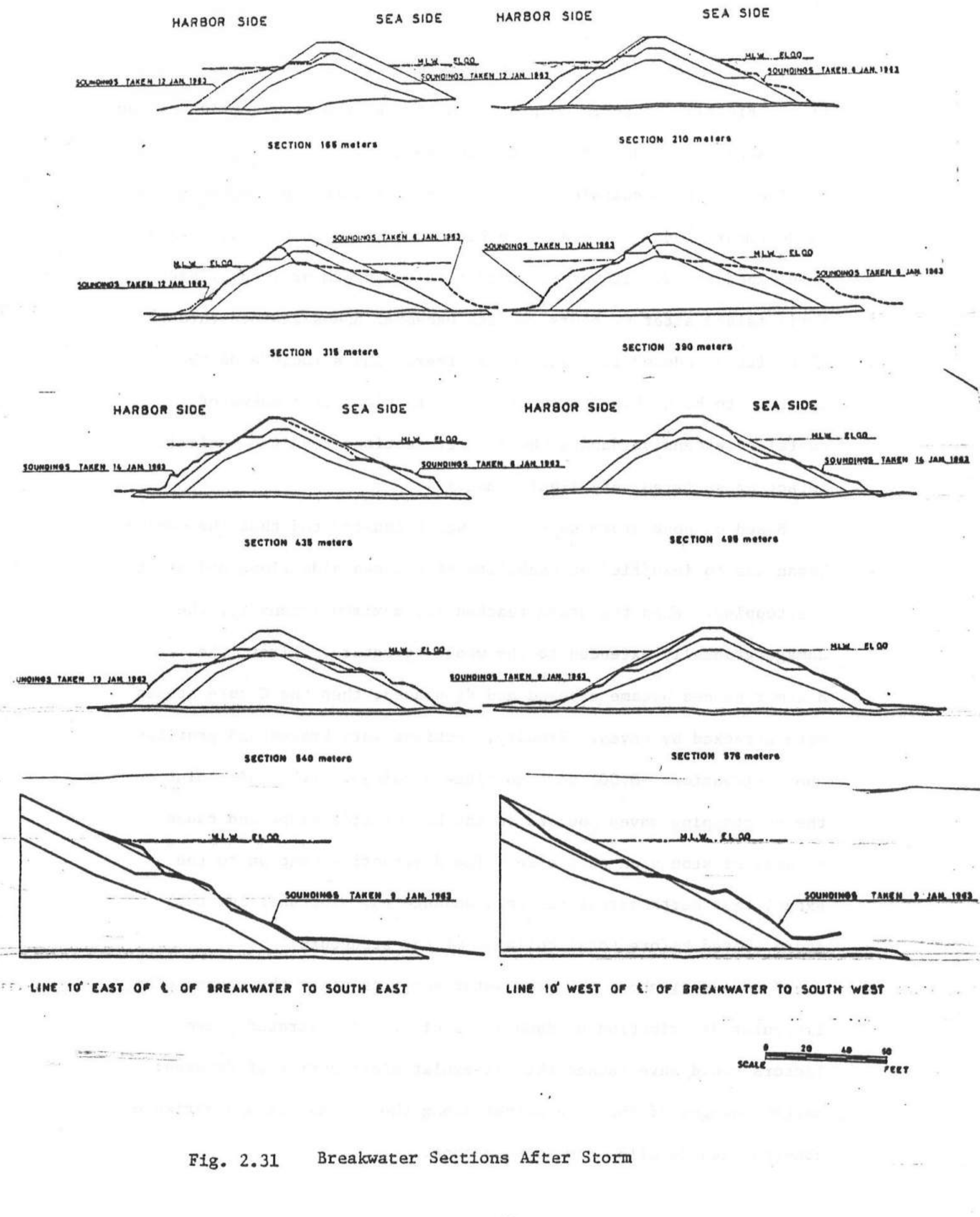


Fig. 2.31 Breakwater Sections After Storm

Later on, model tests were performed at the National Laboratory of Civil Engineering in Portugal. The model tests seemed to agree with the observations in nature. It was found in the model tests that the damages in the sea-side slope were extremely sensitive to water level and wave heights.

J. Breakwaters at Scheoeningen, Ymuiden and Hook of Holland, Netherlands

Figure 2.32shows a typical cross section of a breakwater protecting a pier at Ymuiden. The breakwater has a main body of concrete blocks situated on a rubble mound foundation. The seaward side of the breakwater was protected by solid cubes of concrete placed in pell-mell fashion. These concrete blocks have a specific gravity of 2.2. The breakwater suffered damage when water masses fell down on them and caused the cubes to become displaced or lifted from their original location. Subsequently, the cubes were modified by using lead slabs as aggregates to increase the specific gravity to 2.85. The increase of the specific gravity has led to some improvement, but will not solve the problem satisfactorily.

The breakwater at Hook of Holland is often referred to as an "open" breakwater and was constructed in water of about 50 ft. deep. The breakwater has composite slopes in its cross section. It can be seen from Fig. 2.33that, on the seaward side of the breakwater, the main breakwater cross section has a $1.1\frac{1}{2}$ slope for the top of the breakwater to a depth of approximately 20 ft. It was then connected to a horizontal terrace with a sharp break. The final section was actually a foundation with a 1.6 sloped toe at the seaward side. Solid cubes of concrete were used in the armor layer. The structure suffered similar damage as that of Ymuiden due to falling water mass.

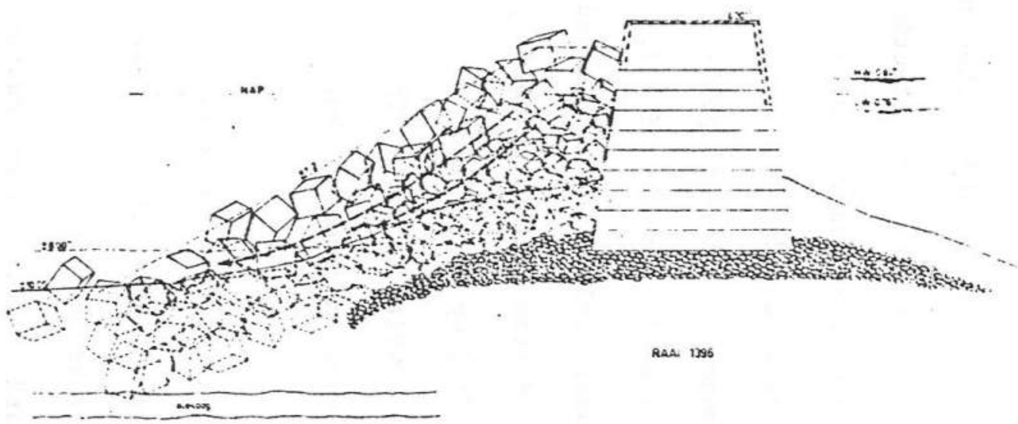


Fig. 2.32 Cross-Section of the Pier at Ymuiden (Old Construction)

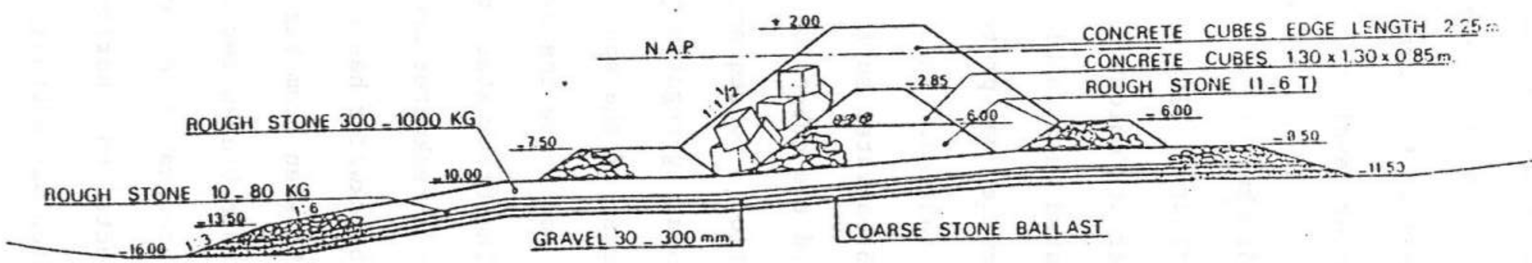


Fig. 2.33 Cross-Section of the Breakwater at the Hook of Holland

The unique feature of breakwater at Scheoeningen is the explatory application of hollow cubes as armor units. These hollow cubes also known as Stolk cubes have been designed by a firm of that name in the Netherlands - J. Stolk & Sons. This cube is provided with internal passages, each two opposite faces of the cube communicating with each other by a single passage. The axes of the three passages intersect in the center of the cube. These cubes were claimed to have the advantage of absorbing wave energy, and slowing down fast-moving current. The southern pier at Schoeningen (Fig. 2.34) has used 2,500 Stolk cubes as the armor layers. The typical cross-section of this breakwater is shown in Fig. 2.35, although it is believed that the perforated breakwater cube will solve the problem of reducing the impact of huge pressures exerted by violent wave action. The information, up to date, is insufficient to make concrete conclusions.

K. Port Talbot, England

Port Talbot is situated on the North side of the Bristol Channel east of Swansea at the mouth of the river Afan. The coastline at this point runs southeast/northwest and is thus facing directly towards the southwest and the open Atlantic with a fetch of upwards of 3,000 miles in this direction, to the nearest land.

Thus exposed, Port Talbot is subject to severe southwesterly gales and on occasion unpredictable swell waves which can come up out of what appears to be a calm sea.

As is well known, the tides in the Bristol Channel are among the highest in the world, the mean spring tidal range at Port Talbot being some $28\frac{1}{2}$ feet and the range at the equinoxes over 33 feet.

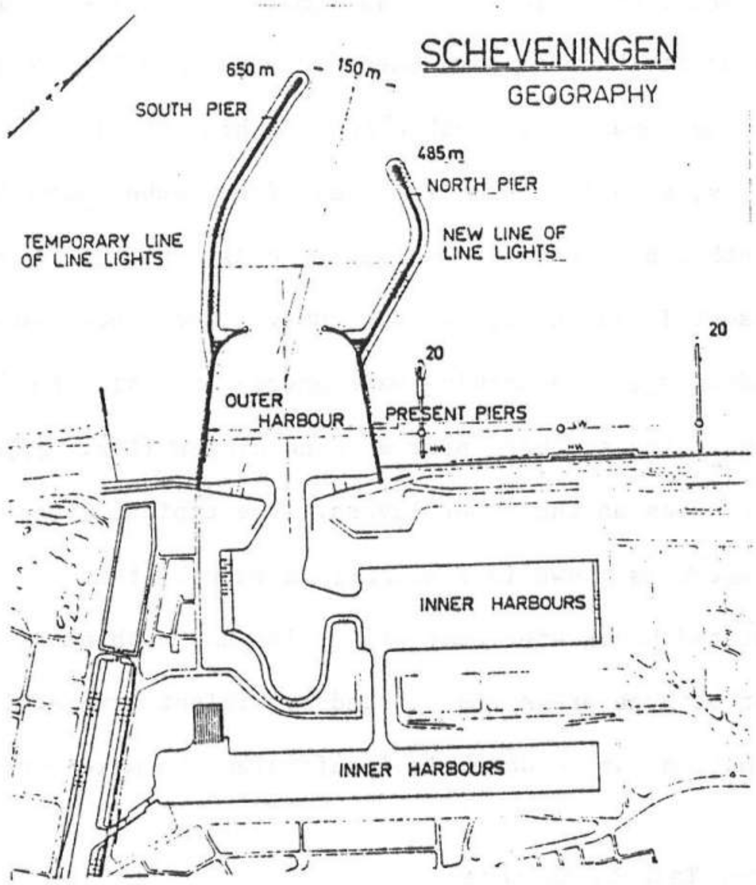


Fig. 2.34 Scheveningen Harbor

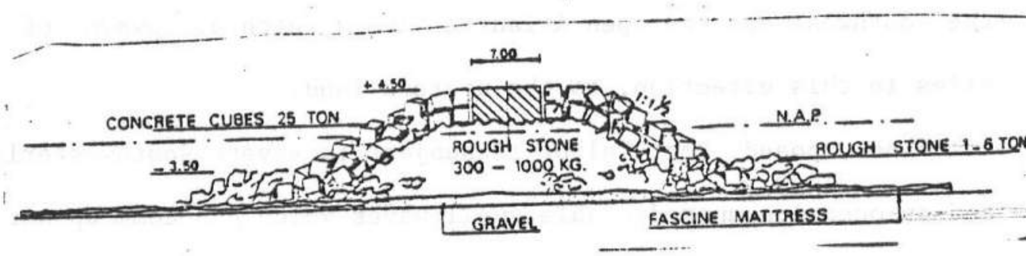


Fig. 2.35 Typical Cross-Section of South Pier at Scheveningen Harbor

The port of Talbot was started in 1834 and a breakwater was built on the south side of the entrance channel for a length of some 1200 feet in approximately an east-west direction. In 1897 an additional 1,000 foot extension to the main south breakwater and a new north breakwater were constructed (Fig.2.36).

The extension to the south breakwater was of much more massive construction than any of the work to that date. The breakwater was a composite type formed of mass concrete faced with concrete blocks, headers, and stretchers laid upon a foundation dredged to a depth of some 8 ft. and filled with heavy copper slag. The structure was keyed concrete blocks laid on sloping land to allow for possible unequal settlement. The seaward face of the breakwater was further protected by the provision of a 22-ton unit weight concrete wave breaking block laid pell mell. In the 70 years or so which elapsed since the strengthening work, it has weathered many very severe storms and required only limited maintenance.

The north breakwater was built at right angles to the shore line on a bearing of 208° E. of N., for a length of some 1,500 ft. from the high water mark and was thus considerably inshore of the head of the main or south breakwater. Possibly because of this, it was looked upon from the first as a "lee" breakwater. This is a misconception for while undoubtedly the north breakwater is protected from the weather by the south breakwater in a direction from east running through south to a bearing of 205° E. of N., the preponderance of fierce gales and extreme waves come from the open Atlantic from a

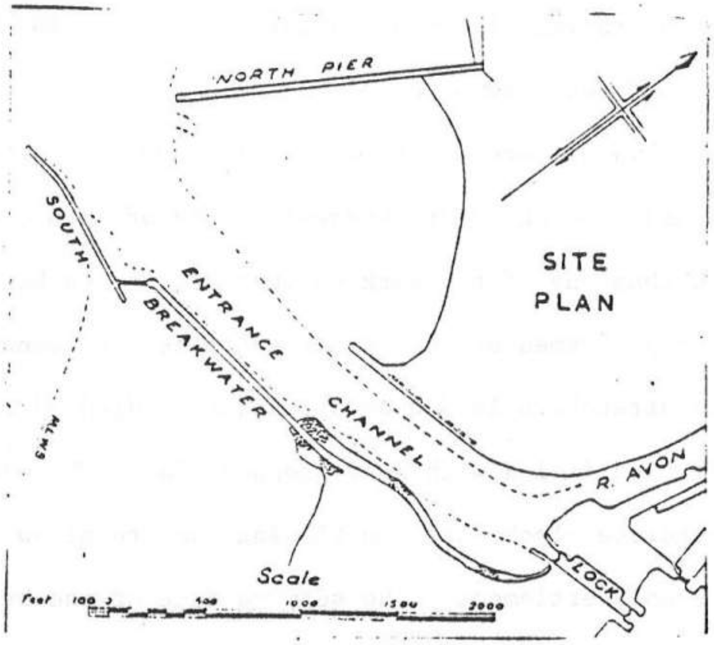


Fig. 2.36 Site Plan Showing the Breakwater at Port Talbot

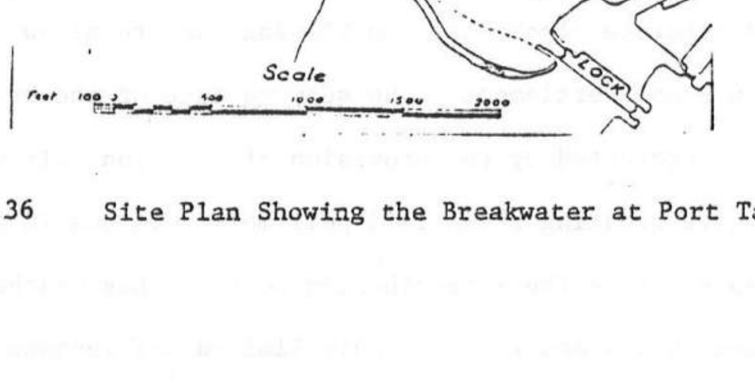


Fig. 2.37 Design of Reconstructed Breakwater

southwesterly direction between 219° E. of N. and 258° E. of N., from which direction both breakwaters are completely unprotected.

Be that as it may, the design adopted in the 90's was of lighter construction than the main breakwater and was of the timber crib type of jetty which was common practice at that time. This consisted of timber piled frames or "bents" driven at 8-ft. spacing and cross braced with half timbers and aligned longitudinally with whole timber walings on each side. The piles were driven to penetrate well into the underlying clay and records show this to have been (or assumed to have been) some 5 ft. below the thin level of the sand surface at the head, the piles penetrating a further 8/9 ft. into the clay at this point.

Short half timber piles were driven between the main bents and attached to the intermediate waling (about 4 ft. below H.W.S.T.) toeing into the underlying clay. From the lowest waling (beach level) 4-in. sheeting was driven to toe into the clay bed. From the bottom waling, an 18-in. thick concrete wall was built up to approximately H.W.S.T. level inside the main piling on each side. The space between these walls was filled in with a hearting of copper slag finished with a layer of grouted slag at about H.W.S.T. The top cross tie of the bents and longitudinal waling were finished with a 4-in. planking deck set at 8-ft. 6-in. above H.W.S.T.

It should be added that the outer 47 ft. of the breakwater forming the head, which was finished so that it could be extended in the future if required, was of a stiffer design and was filled in solid with heavy copper slag and 12:1 concrete.

The north breakwater has frequently sustained extensive damages during the last 50 years and constant repairing was required. Finally in 1960, the British Authority came to the decision to reconstruct the north pier. Various schemes were examined. These schemes can be broadly grouped under two headings:

1. Vertical wall breakwaters with roadway on top;
2. Rubble mound breakwaters with roadway on top.

In designing a vertical wall breakwater, the essentials are, of course, to obtain a stable foundation and, secondly, to build upon it a structure solid enough or suitably braced to withstand the breaking forces of the waves. The first would be difficult and very expensive to achieve as there was no question of forming a raft owing to the presence of the old structure. Furthermore, the sand beach was of such fine grain that it was not suitable to stabilize by cement grouting and, of course, any question of coffer damming was out owing to the extreme tide range. As to stablizing the structure in itself by the introduction of cross ties and braces, this would be very costly as such ties would have to be drilled through the existing structure on low spring tides in reasonably calm weather. It was considered that the actual construction of the vertical wall of the breakwater would be both difficult and costly and the decision was therefore made to examine the possibility of converting the breakwater into a rubble mound.

The characteristics of this form of breakwater are a sloping mound of heavy stone or rubble protected from wave action by a skin or facing of stone or concrete block armoring which is placed in random fashion so as to produce ample voids to absorb the energy of the breaking waves.

The design wave was selected as 15 ft. high. Hydraulic model tests were performed at Grenoble before arriving at the final design. The final design of the reconstructed breakwater has a typical cross section shown in Fig. 2.37 . The existing structure was used as a core or spine for the new work and was covered and partly encased for its entire length of some 1,500 ft. by a 15-in. thick concrete deck slab with skirting walls each side running down to as much as 10-ft. below the new deck level, as a protection to the existing face. This cap was reinforced transversely with old bullhead rails as cramps, interconnected with longitudinal rails and m.s. reinforcement. To allow for uneven settlement or other movement, the capping was constructed in 32-ft. lengths with the idea of forming the core into monolithic units of not less than 100 tons apiece.

From a point 500 ft. from the root or landward end of the breakwater a foundation for the toe of the embankment was formed at a beach level as described.

On the seaward face of the breakwater from the root to c.s. 7 (600 ft.) the exposed toe of the existing work was protected against erosion by Pennant stone blocks Class C (7 cwt. to 15 cwt.) laid on a slope of approximately 1 on $1\frac{1}{2}$.

From this point to approximately 1,260 ft. from the root, the embankment consists of an inner core of B class fill (quarry run to 1 cwt.) covered by 4-ft. thick of C class fill and protected from wave action by a double layer of 2.5 m^3 Tetrapods with, however, a toe row of 5.0 m^3 Tetrapods and finished with a berm at the top of the slope with 5.0 m^3 Tetrapods laid to a density of 4 No. per 8-ft. measured longitudinally. The transverse slope of the embankment was 1 on 1.33.

For the outer 250 ft. and round the head to the lee side, the transverse section of the embankment is armored with a double layer of 5.0 m^3 Tetrapods throughout, laid on a slope of 1 on 1.33. The Tetrapod armored embankment is carried as a wrap round the head to a point 100 ft. shoreward of the head on the lee side. The embankment slope of the lee side is flattened to as much as 1 on 3 to form a haunching or buttress to the armor directly in front of the head and strengthen it against attack from the southwest.

On the lee side, protection against erosion was given to the face of the existing structure, which was in much better condition, by Pennant stone random rubble (7 cwt. to 1 ton) laid at a slope of 1 on $1\frac{1}{2}$ from a narrow berm, level with the bottom of the capping skirt. This was stiffened up with Class S fill (Penant stone 1 to 3 tons) on a considerably widened slope in the last 200-ft. of the outer end to form a buttress to the Tetrapods. The work was completed in 1964.

Several severe storms have been experienced in this time but the general effect seems to have been to lock the fill and armoring more closely than it was possible to place it, and flatten the slope of the unprotected stone armoring on the inshore seaward face to 1 on 3 or 4. During a force 8 gale on the 16th/17th of January when gusting up to 70 mph was recorded, three small Tetrapods were moved down to the beach, close in against the toe of the slope, at a position about 25 ft. back from the change of section in the armoring. Apart from this, the Tetrapods appear to be doing their job very well.

L. East London Harbor, South Africa

The breakwater at East London was the first test ground for the much publicized artificial armor unit--Dolosse. The original breakwater was constructed as a mound formed by rectangular blocks weighing from 15 to 30 tons each, topped with a 36-foot wide concrete cap reaching to 16 feet above Low Water Ordinary Spring Tide and a seaward parapet of 5 feet 6 inches high. By 1884, 1,500 feet of breakwater had been completed and the structure was ended off with a round head. Between 1911 and 1917, the breakwater was extended a further 776 feet using 40-ton rectangular blocks placed at random while the end portion was raised to 19 feet above LWOST. In 1935, the third and final stage of construction commenced. The breakwater was extended by a further 1,000 feet, also to 19 feet above LWOST using 33-ton blocks. This work was completed in 1939 and the breakwater is now 3,276 feet long.

The seaward face of the breakwater was at one time protected with a random layer of 33-ton rectangular blocks over a length of 1,000 feet on the seaward end of the breakwater and with 41-ton blocks over the remainder. During 1944, a severe storm breached the breakwater some two hundred feet from the end, carrying away a considerable number of

33-ton protective armour blocks. The breakwater was repaired and the whole seaward face protected to a height of 24 feet above LWOST with 41-ton rectangular blocks placed at random to an approximate slope of 1 1/4 horizontally to 1 vertical. In 1963, i.e. nineteen years afterwards, it was estimated that the outer half of the breakwater had lost at least fifty percent of its seaward random block protection, while a few sections were almost stripped bare to the original mound core. It was, therefore, evident that the existing rectangular 41-ton armour blocks did not provide a stable protection.

A small number of the 19 3/4-ton Dolosse were placed in a line (not interlocked) on a section of the foreshore near the root of the breakwater to test the individual characteristics of the blocks. They were subjected to breaking waves up to 18 feet in height and, although only seated on small loose round boulders, they moved very little by swinging sideways and tending to "dig in." They showed no tendency to roll or glide away as happens to rectangular blocks.

By the end of 1965 approximately 450 Dolosse had been placed at random around the end of the breakwater and along a short section of its seaward face. It was found during the first severe storm that Dolosse, which were not completely stable yet, moved into more secure positions and a general "settling down" of the Dolosse occurred, forming a permanent and better packed group. After this initial settling no subsequent movement has been observed and the blocks have now withstood the severest storms, with estimated wave heights of up to 25 feet, of two winters, while during the first winter (1964), five 41-ton rectangular blocks were swept over the breakwater cap, at a section where there was no Dolosse protection.

During a storm or 'heavy seas,' and particularly when the wind is blowing in the same direction as the waves, it is quite impossible to traverse the breakwater due to large amounts of water splashing over the top, and due to strong clapotis. On one occasion when the waves were estimated to be of the order of 20 feet high, the only manner in which the light at the end of the breakwater could be reached was by means of a steam locomotive. At the round head, which is protected by Dolosse, it was possible to walk about the breakwater deck with perfect safety, and only a light spray brought over by wind was experienced.

No damage of any sort, including erosion, has been observed in any of the Dolosse over a period of two years and, although many blocks fell and slid four to five feet during placing, none of them suffered any damage except for minor clipping of the edges. This seems to be the first happy story of Dolosse application.

2.3 Major Breakwaters in Japan

There are more than 1,000 ports in Japan; most of them are protected by breakwaters or jetties of one kind or another. Figure 2.38 shows the major ports in Japan. Since Japan is a country continually battling with storms, tsunamis, typhoons, and earthquakes, they have constructed and are continuously constructing many marine protection structures. In 1966 alone, Japan spent $60 million in levees, breakwaters and jetties. Usually, extensive theoretical and model test work is conducted prior to construction. The most typical breakwaters which have been constructed in Japan are caisson-type composite breakwaters. Rubble-mound structures are also common. A few of the important ones are described here.

A. Nagoya, Ise Bay

The Nagoya breakwater is a large-scale breakwater built as a link in the general Ise Bay Storm-Tide-Prevention Project. The entire length of this breakwater is about 8,250 m (27,000 ft), which extends from the mouth of the Nebeta ruin, transversing the northern part of Ise Bay, to the Chita Town (Fig.2.39). The purpose of this breakwater is to prevent damages from storm-tide and waves caused by typhoons. This project was completed in September, 1964 at a cost of about $30 million. Based on past oceanic meteorological data and computer analysis of storm tides, the following design conditions are established:

a) Waves: 3.0 m (12 ft.)

b) Storm Tides: 2.3 m (9.1 ft.)

c) Meteorological Tides: 3.55 m (9 ft.)

d) Crown Elevation: 7.5 m (19 ft.)

e) Soil Condition: sand to silty clay

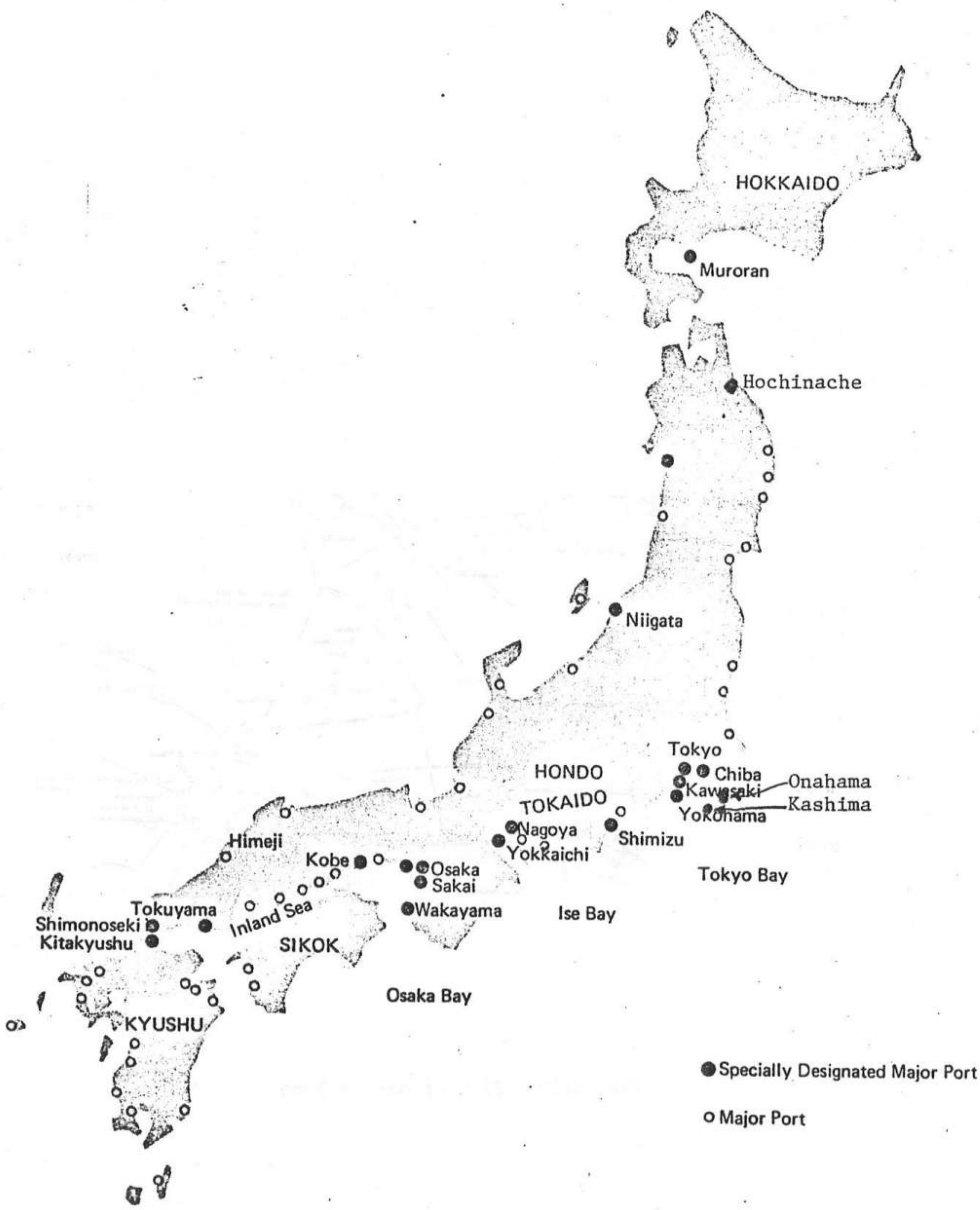


Fig. 2.38 MAIN PORTS OF JAPAN

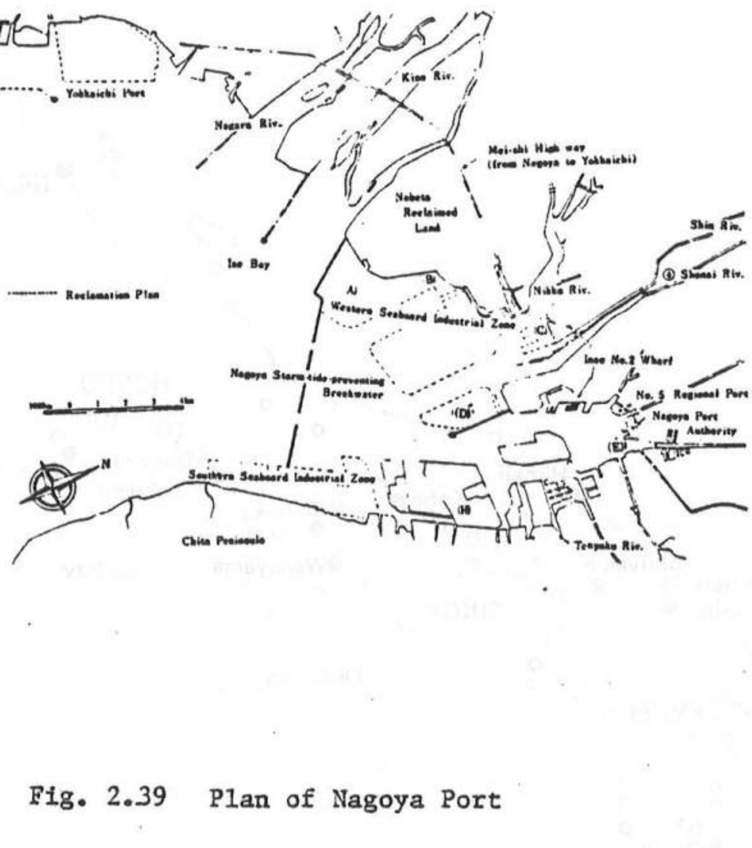


Fig. 2.39 Plan of Nagoya Port

Since the soil condition along the line of the breakwater varies from place to place, the entire breakwater was divided into four sections as shown in Fig.2.40. In the exchange section, the soil is very poor for foundation purposes and is therefore exchanged through dredging and replacing with sand. The breakwater in this section is of composite type as shown in Fig.2.41 . The drain section is a section where the soil condition is marginal and is improved through sand-draining. Typical breakwater cross section in this section is shown in Fig.2.42. On the western part of Naleta Bank where the soil is good and the water is shallow, stepped rubble-mound breakwater, such as that shown in Fig.2.43, was chosen. Finally, a transient section of concrete block (Fig.2.44) was chosen to link the mound section and drain section.

The caissons are of ordinary type where the side walls are designed as a fine slab of three sides. The armor layer is composed of "A" stones of weight determined by the Iribarren-Hudsen formula.

In summary, the Nagoya storm-tide-preventing breakwater is one of the major protection projects in Japan. It was well planned, designed and executed. It is one of the unique structures that consist of a variety of structure types linked together.

B. Kaga-Sanko Jetties

The Kaga-Sanko jetties were constructed in 1965 at the outlet of a diversion channel of the Kaga-Sanko reclamation project (Fig.2.45). The west jetty is 130 m (512 ft) long and the east jetty extends 90 m (354 ft). This is not a large structure by any means but is one of the few recently constructed protection

NAGOYA BREAKWATER

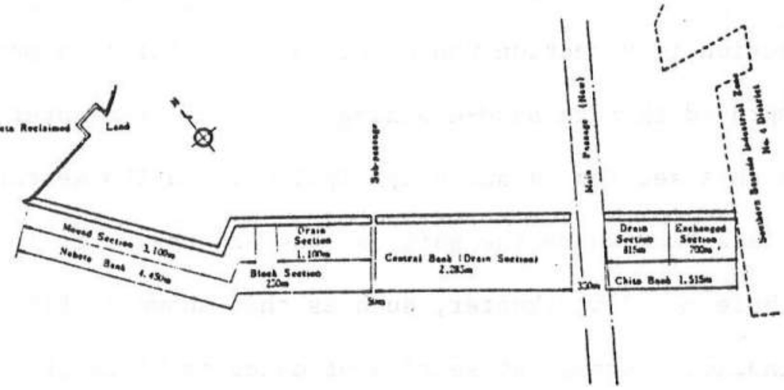


Fig. 2.40 Plan of the breakwater

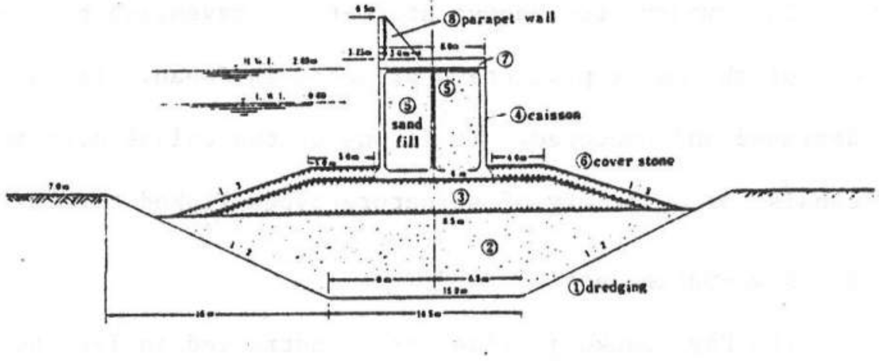


Fig. 2.41 Standard Cross Section of Exchanged Section

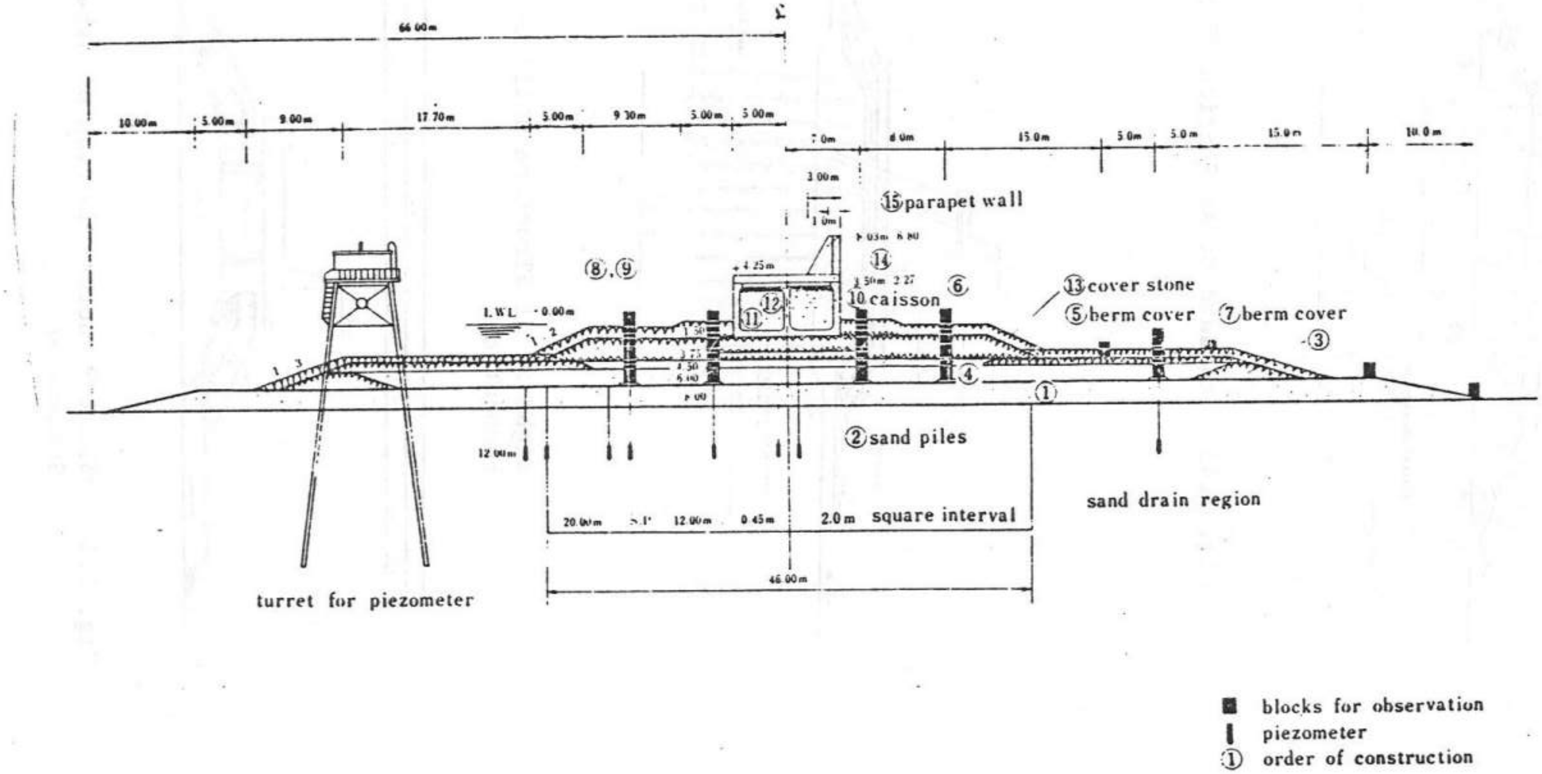


Fig. 2.42 Standard Cross Section of Drain Section. (Arrangement of Investigation Devices at 900m point) (Nagoya Breakwater)

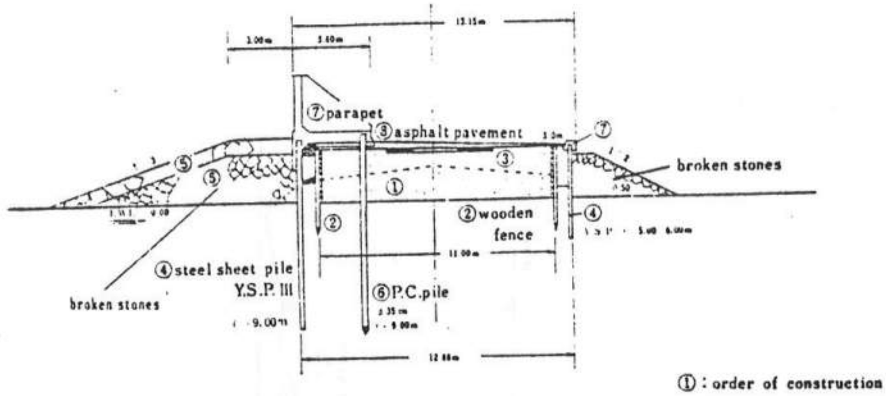
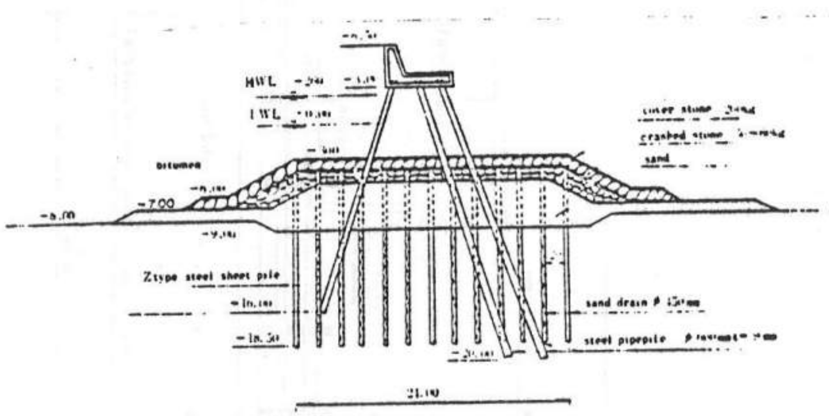


Fig. 2.43 Standard Cross Section of Mound Section



Light Steel STructure Build on the Sand-Drained Foundation.

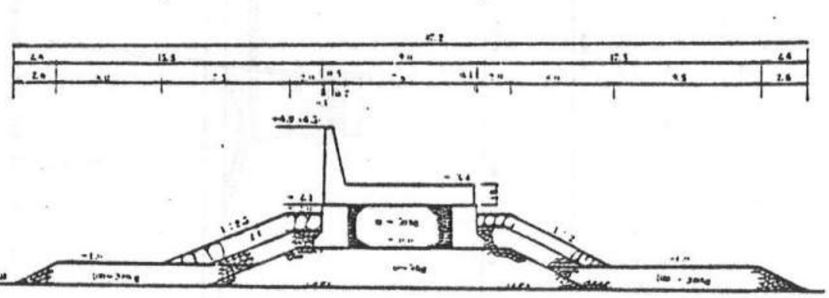


Fig. 2.44 Block Structure Section of Nabeta Bank of the Breakwater

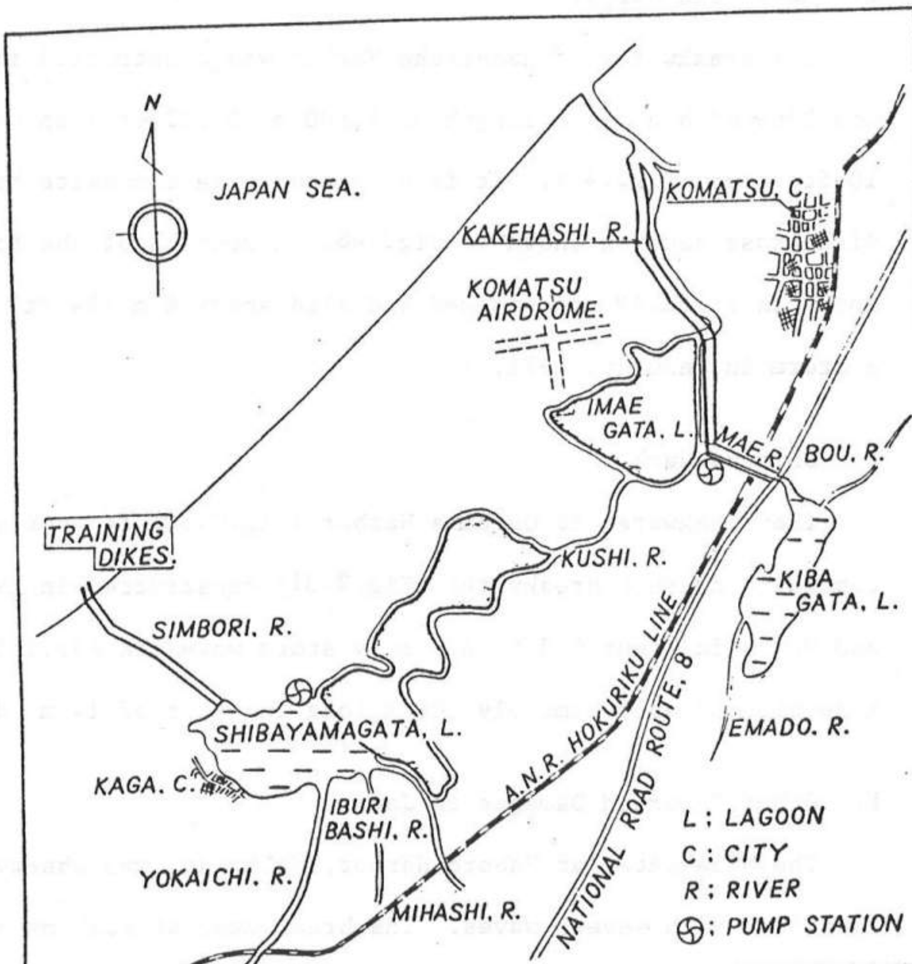


Fig. 2.45 General Plan of Kaga-Sanko Reclamation Area

structures in Japan using tetrapod as armor unit of 2 to 6 layers and having a steep face slope of 1 vertical to 1 horizontal. Typical cross-sections are shown in Fig.2.46. It is another example of composite-type breakwater commonly seen along the Japanese coast.

C. Hachinoke Harbor

The breakwater of Hachinoke Harbor was constructed in 1968 and 1969 with a total length of 1,400 m (5,512 ft.) in water to 10 ft. deep (Fig2.47). It is a caisson-type composite breakwater with cross section shown in Fig2.48. A section of the breakwater shown in Fig.2.49 was damaged and slid about 6 m (24 ft) during a storm in January, 1971.

D. Onahama Harbor

The breakwater at Onahama Harbor (Fig.2.50) is another composite caisson breakwater (Fig.2.51) constructed in 1967 and was slid about 0.3 $\sim$ 0.9 m by storm waves in April 1971 for a section of approximately 300 m long in water of 14 m deep.

E. Other Recorded Damages in Japan

The breakwater at Haboro Harbor, Hokkaido, was observed to oscillate with severe waves. The breakwater at Kashima suffered local scouring and slid about 0.2 m to 5 m in a storm of January, 1972.

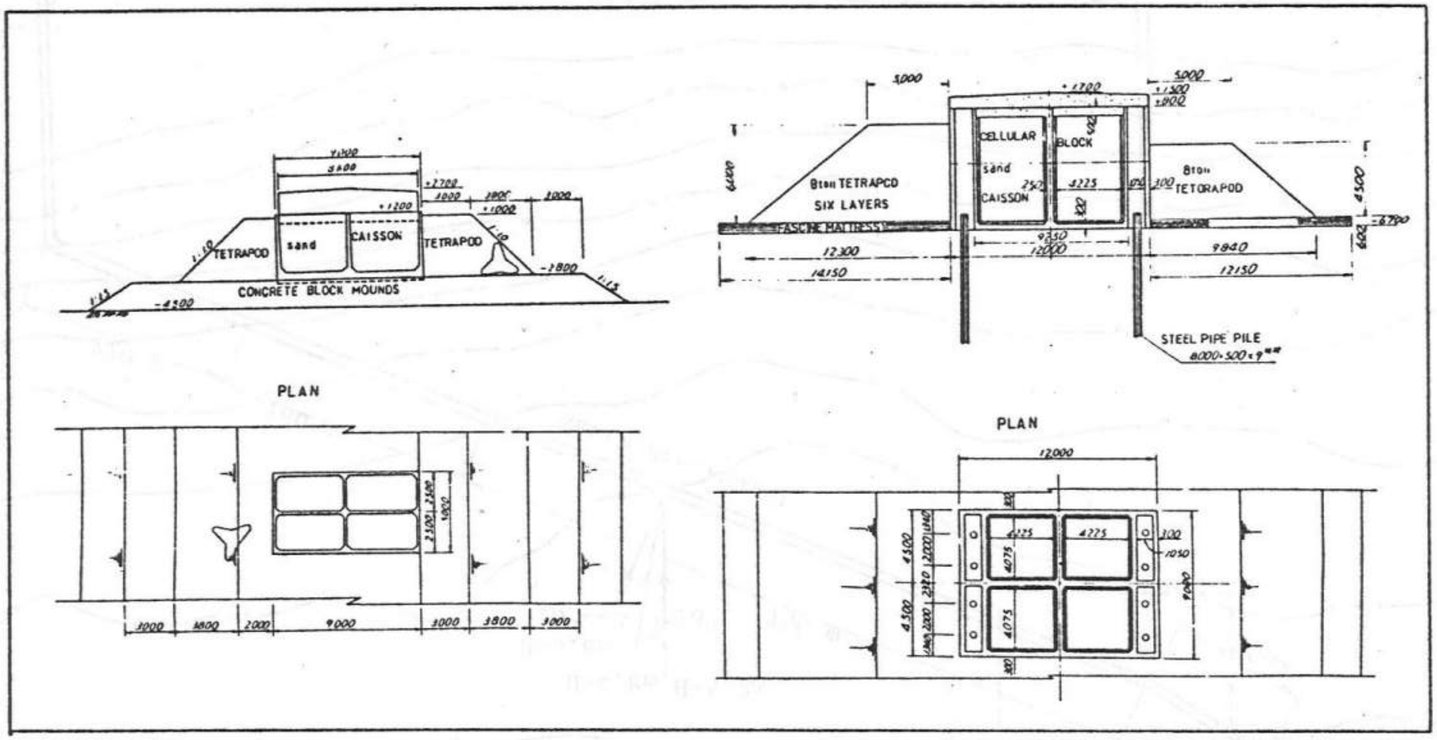


Fig. 2.46 Standard Cross Sections of Training Dike at Kaga-Sanko

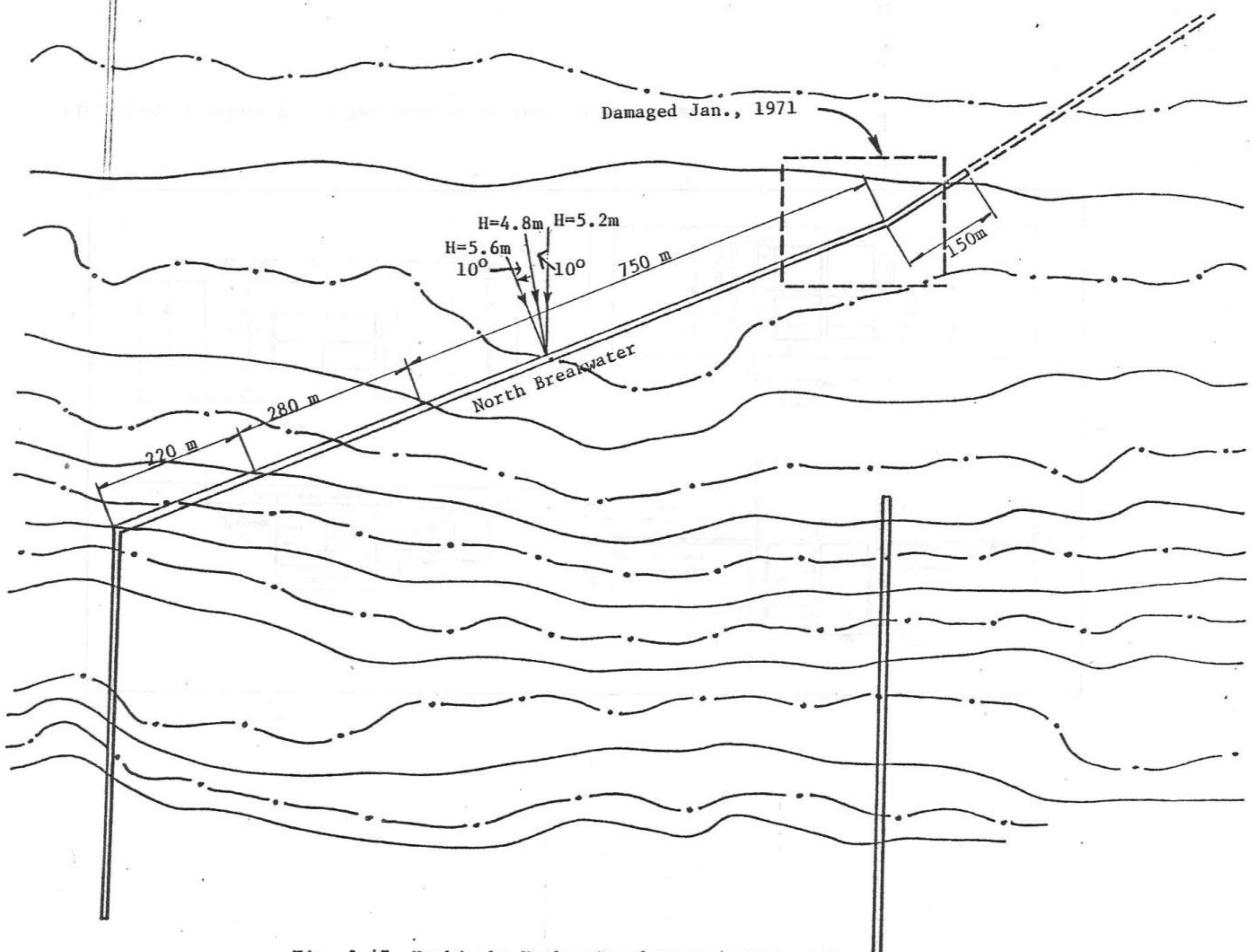


Fig. 2.47 Hachinohe Harbor Breakwater Arrangement

Dead Weight	820.8 tt
C.G.	3.72m
Draft	4.74m
Center of Buoyancy	2.37m
MG	0.59m 0.05d
Sliding	1.19
Over Turning	2.77
Tip Pressure	P_1=268 t/m^2
	b^1= 15.36m

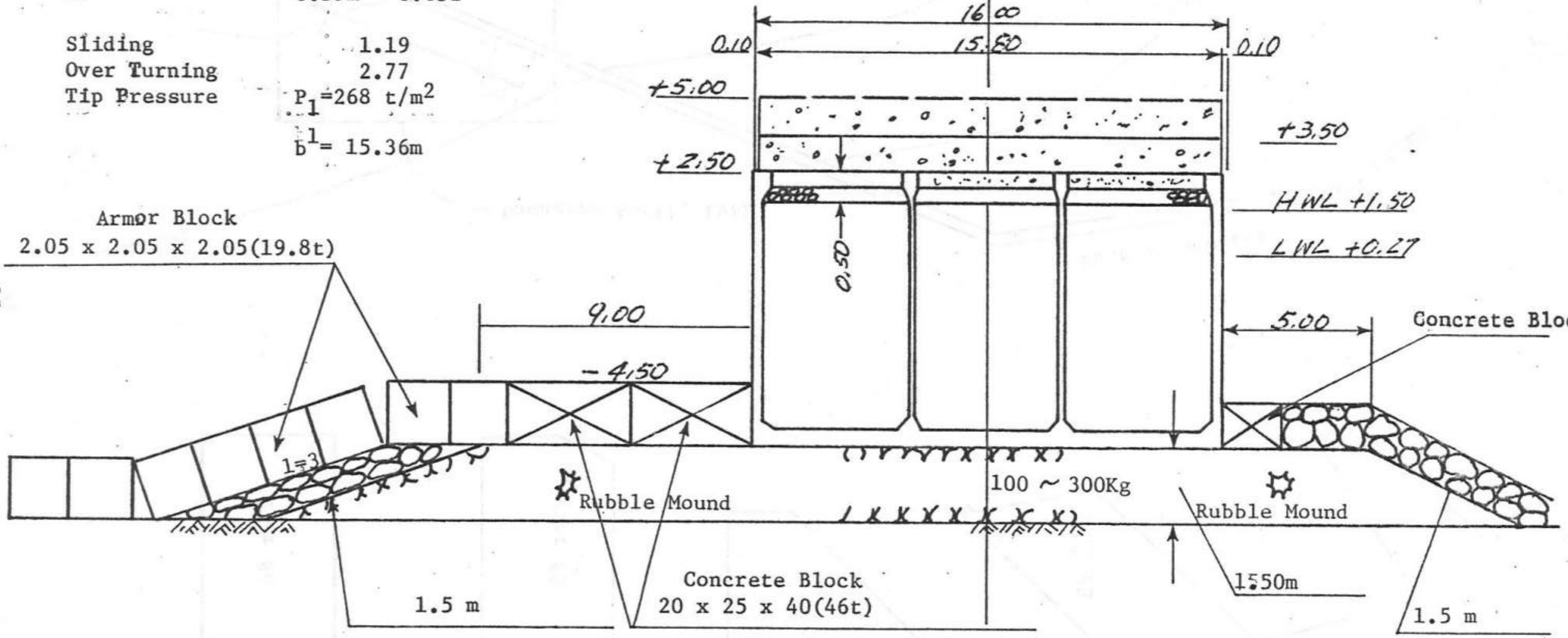


Fig. 2.48 Hachinohe Harbor Breakwater Typical Cross Section

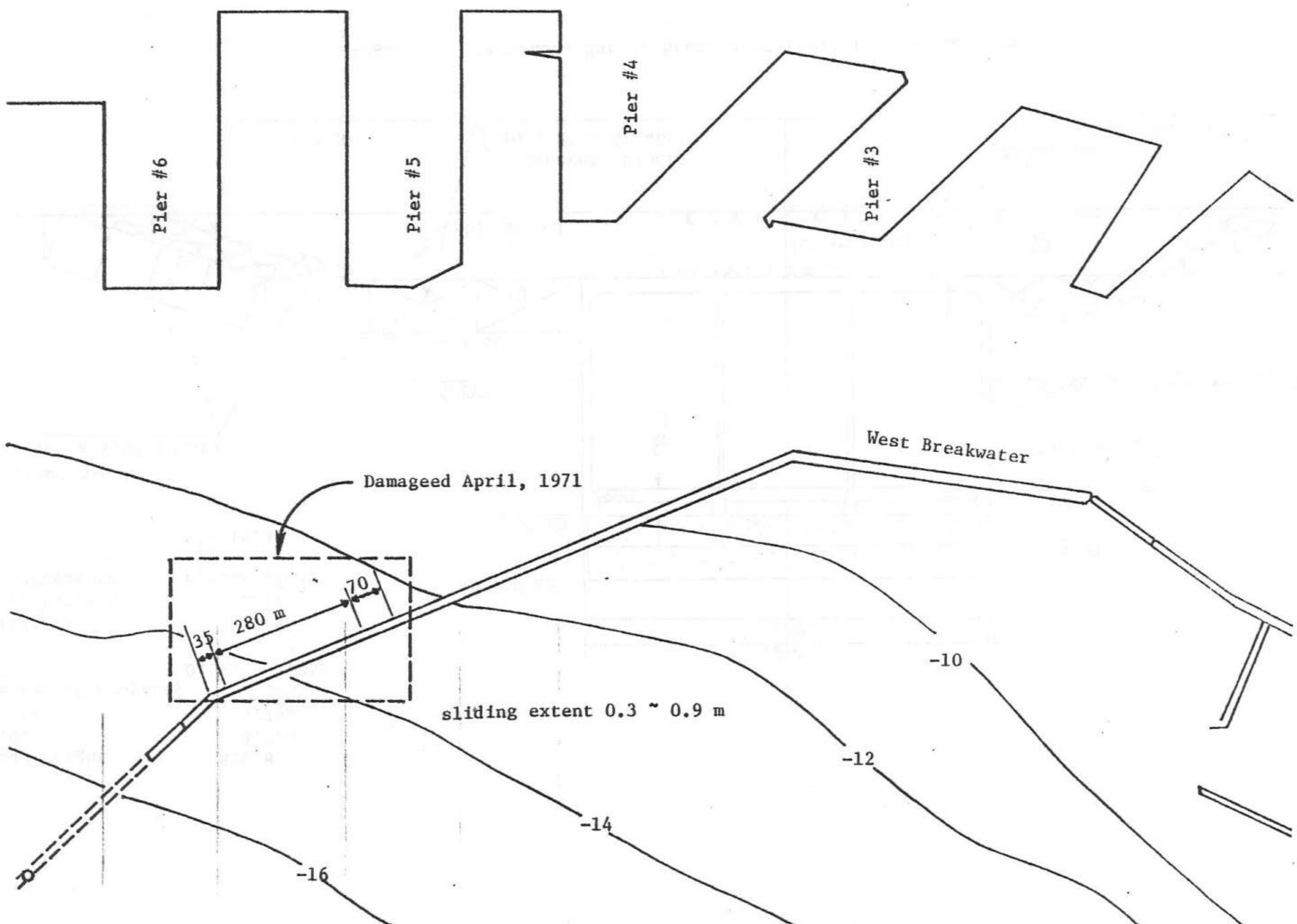


Fig. 2.49 Onahama Harbor Arrangement

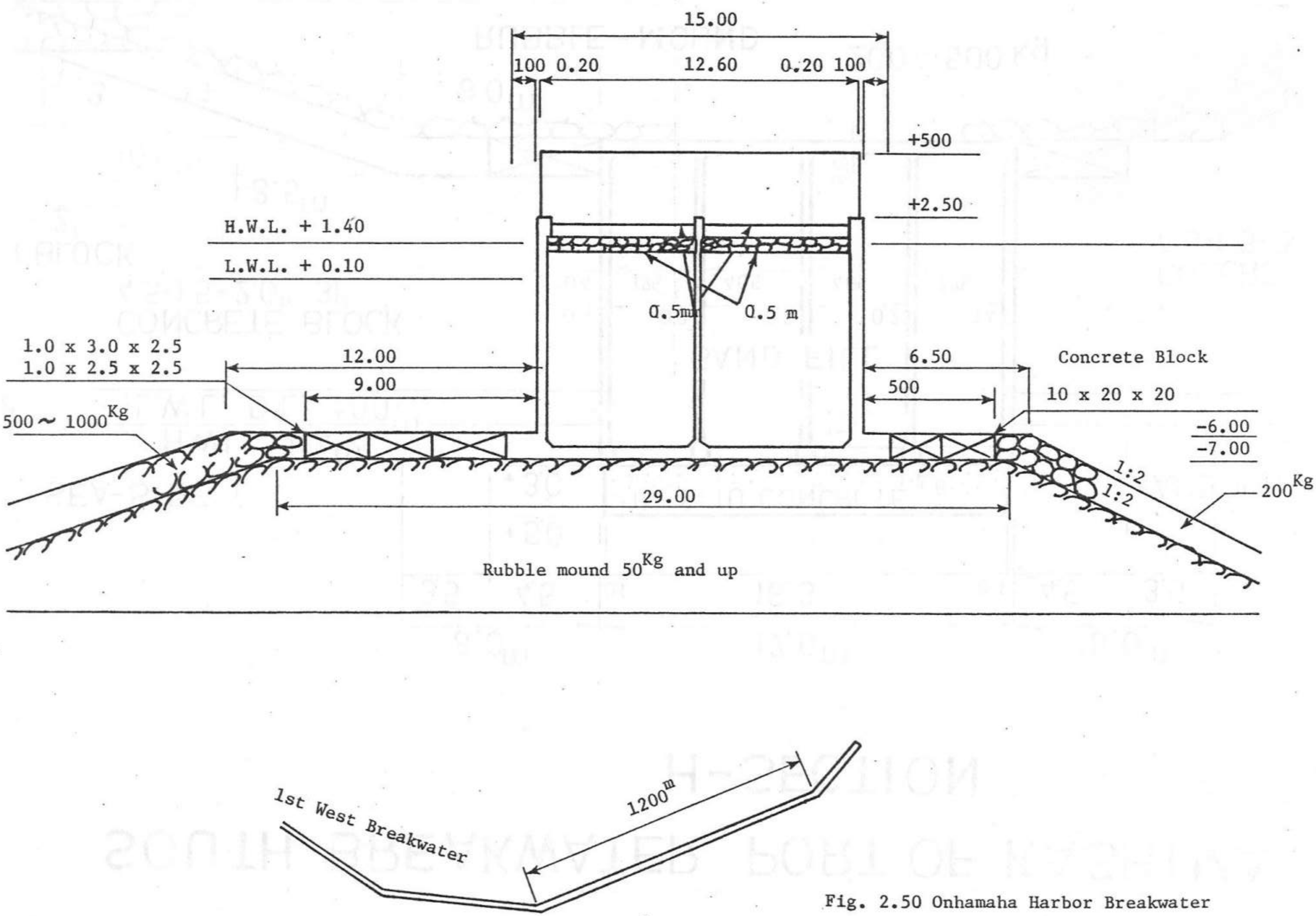


Fig. 2.50 Onhamaha Harbor Breakwater

SOUTH BREAKWATER, PORT OF KASHIMA
H-SECTION

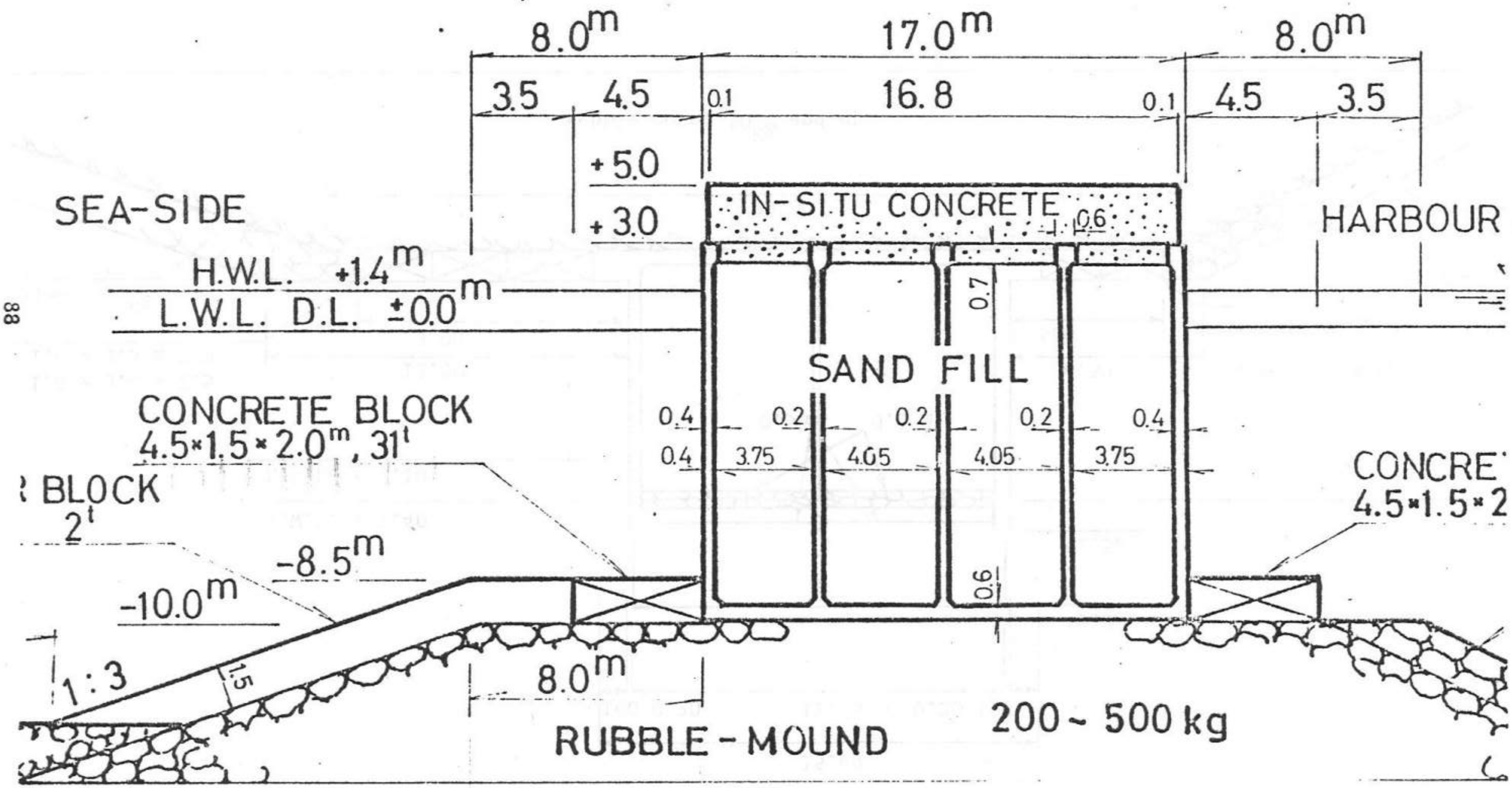


Fig. 2.51South Breakwater, Port of Kashima

E. Breakwaters at Madras, India

A section of the original breakwater is shown in Fig. 2.52; each block, of which the vertical wall was constructed, weighed about 27 tons and was built into the wall without bond. The sea-bed consisted of a large bed of shifting sand extending some 600 feet from high-water marks into about 15 feet depth of water, beyond which the bed of the sea was stiff blue clay with a small depth of firm sand upon it. The beach slope was steep, and there was always a heavy surf breaking on the shore.

In November, 1881, a cyclone originating somewhere near the Nicobar Islands travelled across the Bay of Bengal in a north-westerly direction and struck the Coromandel coast some 100 miles south of Madras. The waves came with great regularity from a direction a little north of east, the breakers being parallel to part of the north face of the old north wall. The swell reflected from the face, meeting the waves direct from the sea, set up large confluent waves which appear to have closely hugged the elbows (the sheltering arm being, of course, not yet in existence), following them round and dislodging the blocks so that the outer row fell outwards, and the inner row inwards. On the faces CD and HG (Fig. 2.53) the walls rocked and fell inwards. The north and south pier or arms were undamaged, but water poured over them and caused a current from the harbour outwards through the entrance. Some sections of the vertical walls were undermined in places to the unprecedented depth of 22 feet below water, and some of the work even fell outwards under pressure of the water pent up within the harbour. A report on this damage was submitted by G. L. Molesworth, who made a close underwater inspection, and gave his opinion that the following effects of wave action and ground swell were the causes of the catastrophe:

1. Direct blows of waves on the outer blocks, the force of which may have varied from 1 to 3 tons per square feet.

2. Percussive action communicated from one block to another; this might cause the last or inner row of blocks to be driven in, while the outer row and intervening rows were untouched.

3. Compression of air in the joints; this is caused by the wave blows, and the concussion is communicated to the water also in the joints--an action similar to that of the hydraulic ram.

4. Dragging action of the waves on the top block.

5. Vacuum formed behind the wall as the waves overtopped the wall and receded into the harbour (Fig. 2.52).

In consequence of this disaster the walls, after repair, were raised an additional 9 or 10 feet in height, with improved bonding and jointing, together with protection in certain places composed of 30-ton blocks; the wall as redesigned in 1883 is shown in Fig. 2.54.

Unfortunately it has to be recorded that during a cyclone of exceptional severity in November 1916 the monolith pierhead, 42 feet square and 50 feet in height, and founded on rubble on the sea-bed (which at this point lies at a depth of 42 feet below low water), was tilted over into a hole about 10 feet deep caused by the scouring action of the storm waves on the northern and western sides of the base. Deprived of the protection afforded by the pierhead, a contiguous length of 130 feet of the new sheltering arm was destroyed piecemeal. This damage was made good and a new pierhead installed after World War I.

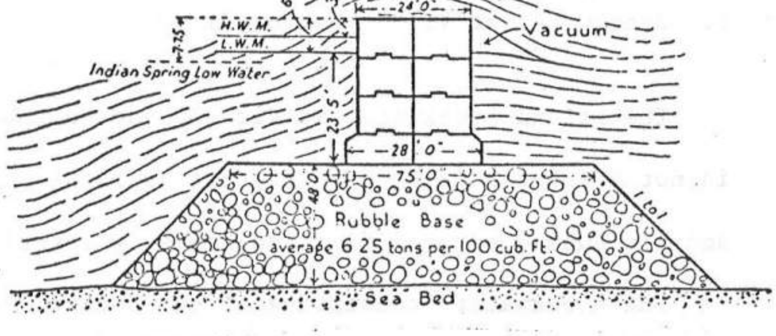


Fig. 2.52--Section of original breakwaters at Madras

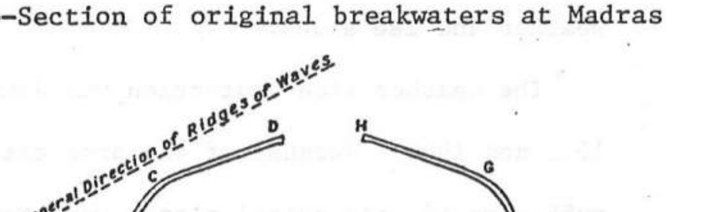


Fig. 2.53--Madras Harbour: original breakwaters, showing damage due to cyclone of November, 1881.

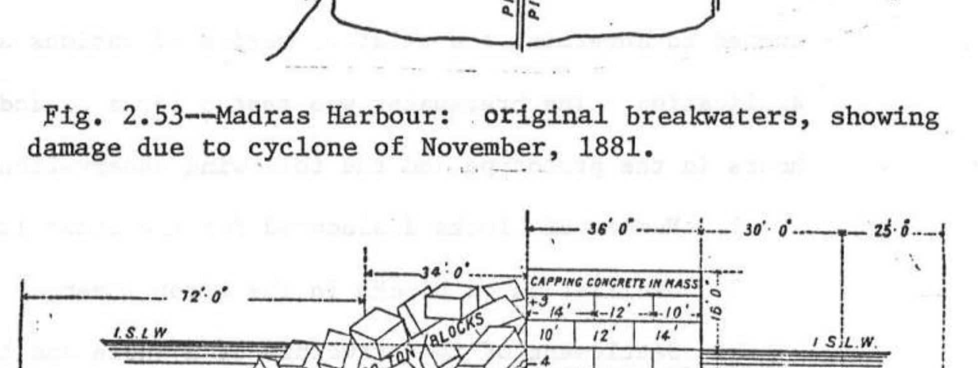


Fig. 2.54--Section of Madras Breakwater as redesigned (1883).

F. Breakwater at Virzagapatam, India

The island breakwater at Vizagapatam on the east coast of India is not a substantial structure, but is rather unique in its construction and its subsequent modification; thus, warranting some mentioning.

The breakwater, consisting of two suttled ships filled with 1/2 cwt to 1 cwt rubble stones, was protected by 2-ton to 6-ton stones on the weather and lee sides.

The weather side protection was damaged heavily by the storms of 1933 and 1940. Because of the progressive removal of stones in the rubble mound, the ships' plates were badly damaged. The rubble was consequently drawn out to the sea leaving hallows at numerous places. Realizing the necessity of strengthening the breakwater, 5-ton blocks were placed in 1959 on top of the rubble mound for preventing further deterioration (Fig. 2.55). These concrete blocks also failed to withstand wave action and assumed flatter slopes. Experiments were conducted to determine the relative merits of various armor units for optimum application. The breakwater was tested for a period equivalent to 12 hours in the pratotype and the following observations were noted:

1. Number of blocks dislocated for the armor layer;
2. Oscillation of blocks in the armor layer;
3. Settlement of the structure as a whole and the final profile of the structure that was attained;
4. Stability of the toe of the breakwater;
5. Any strikingly notable characteristics;
6. Pressures on the ships' plates in order to compare the effectiveness of different protective layers in sustaining and reducing wave attacks.

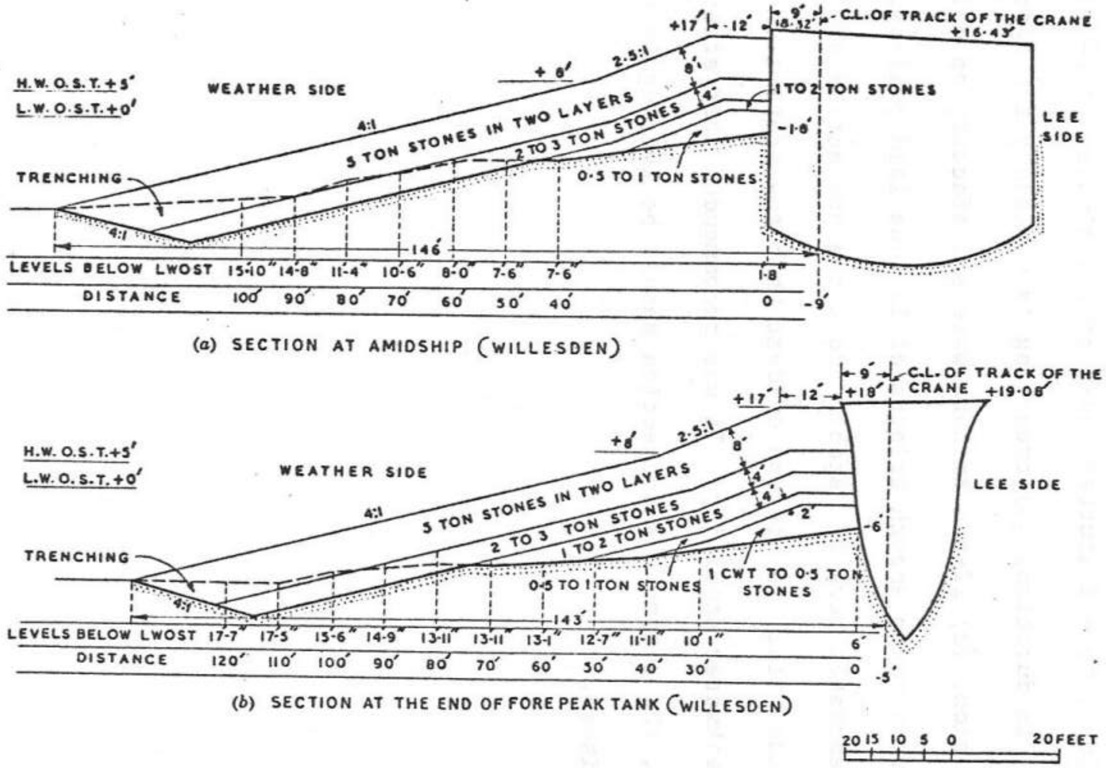
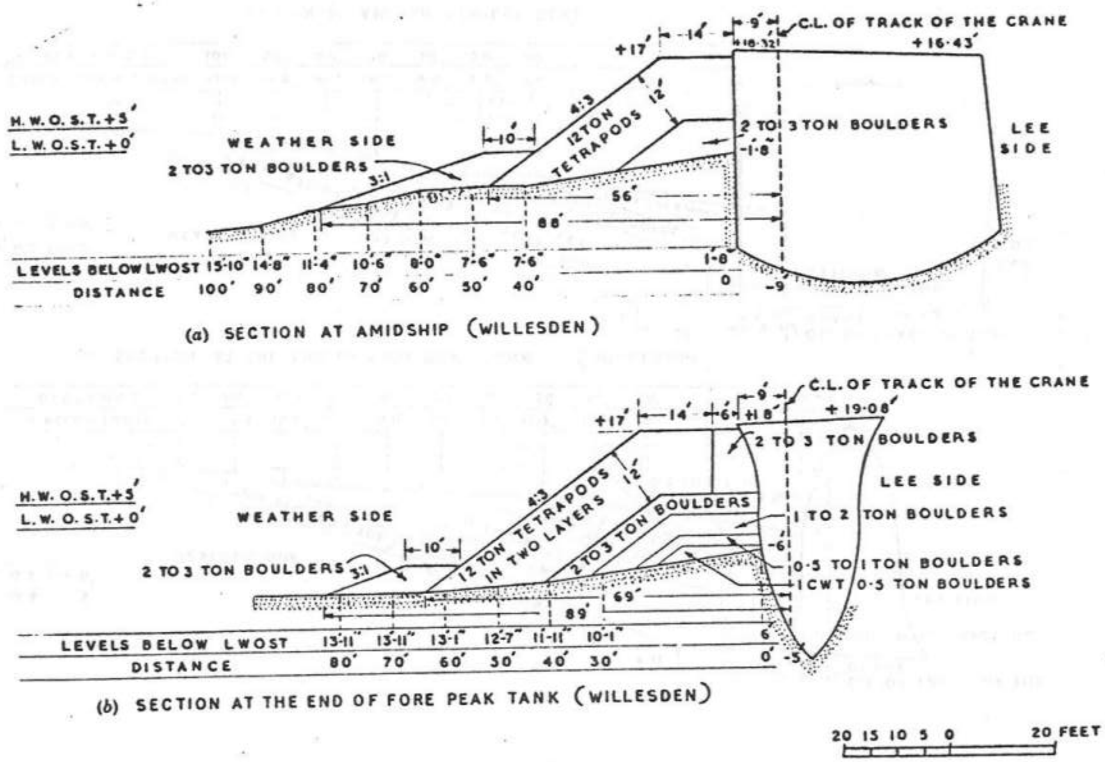


Figure 2.55--Original Breakwater Cross-Sections

The various armor units tested included Concrete blocks of 12-ton, 20-ton, and 25-ton (Fig. 2.56) tetrapods of 12-ton size (Fig. 2.57) and tribars of 12-ton size. (Fig. 2.58). It was found that (a) the stone blocks were completely unstable, (b) the 12-ton tetrapods laid on a 4 to 3 slope, were disturbed under breaking waves of 14 feet high and 14 sec. in duration, (c) trenching is necessary for tetrapods to prevent dislocation, (d) 12-ton tribars were satisfactory on a 1.5 to 1 slope, however, strange enough tribars of 12 tons laid pell-mell at the toe of the breakwater were not stable and a toe protection of 5-ton stones was found essential, (e) it was evident that toe protection is crucial to the breakwater stability. It was recommended that after each storm season, the entire toe protection should be thoroughly examined and replenished.



(a) SECTION AT AMIDSHIP (WILLESDEN)

(b) SECTION AT THE END OF FORE PEAK TANK (WILLESDEN)

Figure 2.56--Protection with 12-ton Tetrapods on 4 to 3 Slope

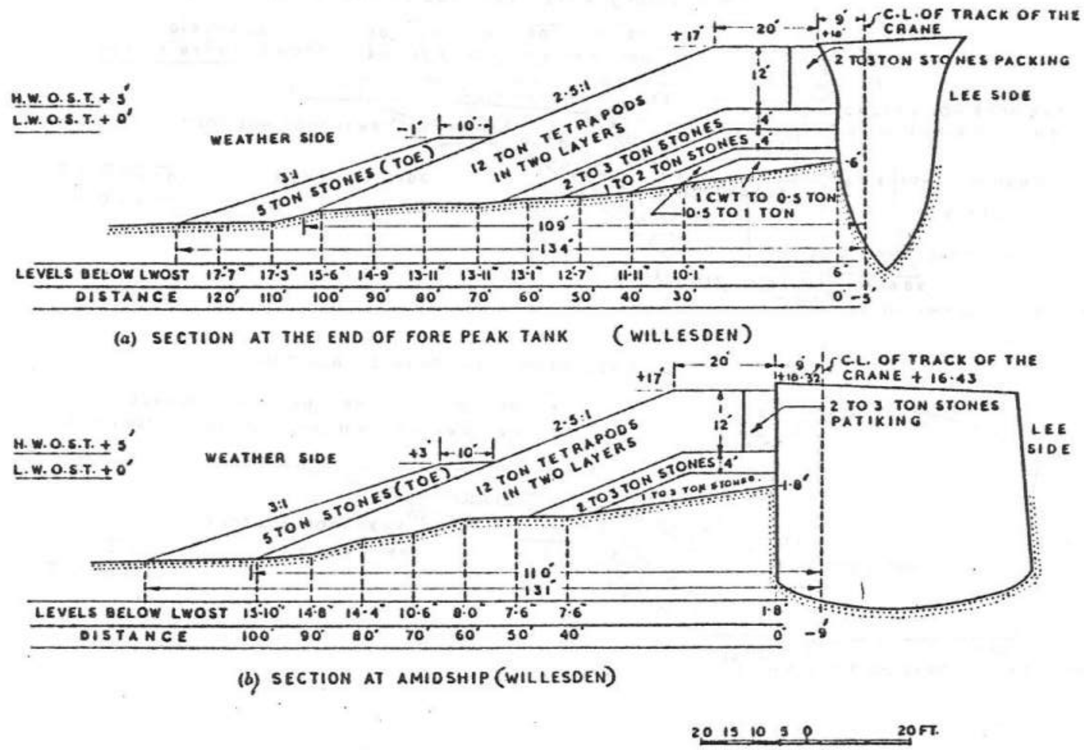


Figure 2.57--Protection with 12-ton Tetrapods on 2.5 to 1 Slope

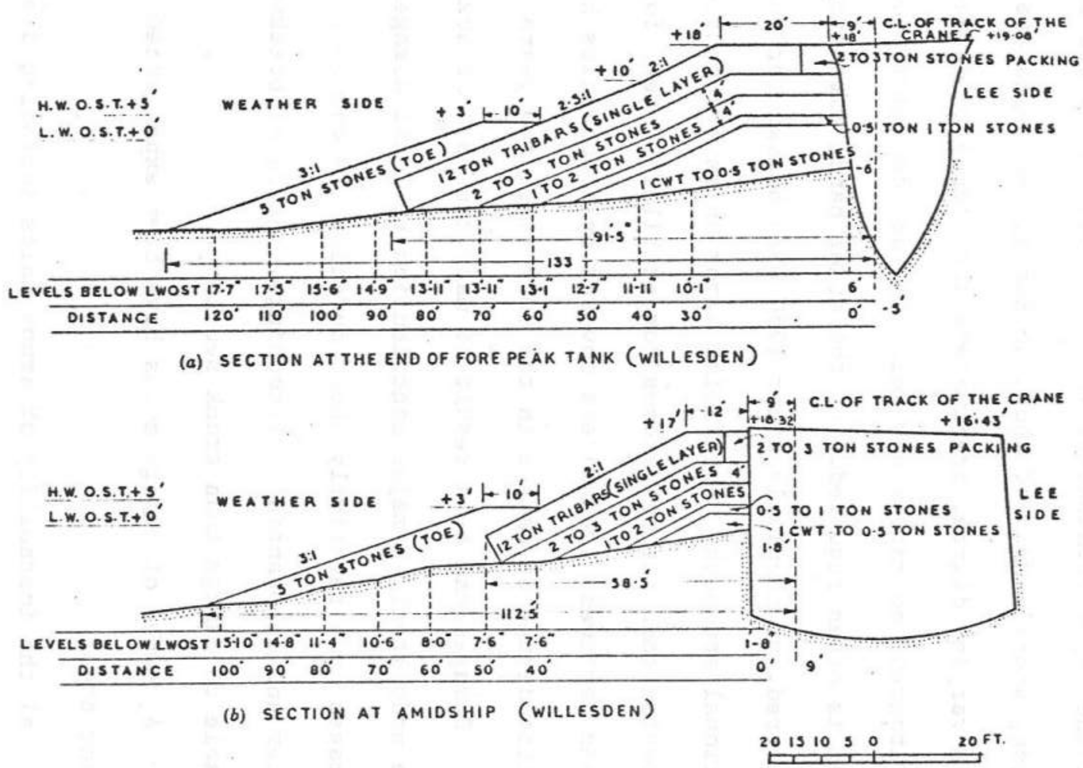


Figure 2.58--Protection with 12-ton Tribars

2.4 Summary

Major breakwaters in the world largely falls into one of the two categories; that is, rubble-mound type and composite type. Almost all the major breakwaters in the United States are rubble mound structures, whereas most of those in Europe and Japan are composite type. To a varying degree, breakwaters are liable to damage due to hostile environment no matter how well they are designed. Constant replenshment is often required. On the other hand, catastrophic failures seldom occurred, particularly after 1950 when breakwater design took a more rational approach. Artificial armor units are generally considered superior than natural stones for stability. Yet, for almost all the cases reviewed, breakwaters covered by armor units have sustained a finite degree of damage in the very first few years of emplacement.

Damages can be a result of many causes. Attacks of breaking waves are undoubtedly a major contribution to local damages. However, major damages in a relatively short duration are commonly coincided with high water and substantial overtoppings. Heads of breakwaters are more liable to damage than trunk sections.

A number of damage modes have been experienced in the past instances, among them:

a) the instability of armor units including dislocation, breakage, settlement and consolidation (most of the rubble mound breakwaters).

b) breakwater sections sliding off (Catania, Genoa, Madras).

c) breakwater collapsed as a whole (Algiers).

d) settlement of foundation (Marseille).

e) structure breakage due to falling water (Alderney).

f) the insufficient stability of sea-side slope and the resulting flattening (Praia de Vitoria).

g) section bleached due to falling water (Ymuidem).

h) breakwater section sliding out of place (Hachinoke, Onahama).

i) scouring of foundation (Kashima,Madras)

j) breakwater caisson overturned due to difference of sea level between sea side and Harbor side caused by trunami (Hachinohe).

Hydraulic model testing proved to be an important and useful tool in breakwater design. There were good evidences that model studies can reveal the potential damage areas. The model studies were less accurate in predicting the extent of damage. Unfortunately, the model predictions tend to be less conservative.

3. CURRENT DESIGN PRACTICES

3.1 Existing Codes, Standards, and Specifications

In the United States there are no existing design codes and/or specifications for breakwater design. Commonly referenced design standards are:

a. Shore Protection, Planning and Design Technical Report No. 4, 3rd ed., U. S. Army Coastal Engineering Research Center, 1966.

b. Design Manual - Soil Mechanics, Foundations, and Earth Structures NAVFAC DM-7 U. S. Naval Facilities Engineering Command, 1971.

c. Design Manual - Waterfront Operational Facilities NAVFAC DM-25, U. S. Naval Facilities Engineering Command, 1971.

d. Design Manual - Harbor and Coastal Facilities NAVFAC DM-26, U. S. Naval Facilities Engineering Command, 1968.

In Japan, there are no design codes and/or specifications either. The commonly referenced publications are:

a. Design Criteria of Port and Harbor Structures Japan Association of Port and Harbors.

b. Design Handbook of Shore Structures Japanese Society of Civil Engineering.

c. Handbook of Hydraulics Formula Japanese Society of Civil Engineering.

In England, there are no design codes and/or specifications. The commonly referenced publication is:

Dock and Harbour Engineering (4 Vols.) Charles Griffin & Co., LTD. London, 1969.

In the Netherlands there are no design codes and/or specifications. Important literature concerning the design, construction and stability aspects are a series of confidential reports M856 (14 parts), M1105 (3 parts), M1117 and M1225 (all in Dutch).

The Director of the Rijkswaterstaat also published a number of notes concerning design procedures for rubble mound breakwater that have been used during the past decade in the Netherlands.

In summary, from all the countries with which I corresponded, there are no design codes and/or specifications that are enforced by public agencies.

3.2 Commonly Accepted Design Practice

3.2.1 Environmental Factors

Oceanographic data rarely appears orderly but random in nature. Therefore, statistical method is often employed to analyze the data and to set criteria to determine proper values for design purposes.

In the offshore structure design, two types of statistical information are generally sought. The first type is the so-called long term statistics which provide design parameters such as 50-year design wave, 100-year extreme winds, etc. Such parameters when incorporated with designated design structure life enables one to determine the loadings. The second type of information involves the statistical representation of certain oceanic phenomena which do not have simple time or space variations. Typical examples of this type can be found by using the spectrum to represent sea surface variations or earthquake input. Such information is of paramont importance when one considers the dynamic response of offshore structures.

3.2.1.1 Wind

Wind is an important but indirect factor in breakwater design. Direct wind loading is generally neglected. However, storm waves, storm tides, wind-driven currents and, sometimes, water sprout are all closely related to the design wind condition. Two kinds of information are only commonly employed - long term statistics of extreme values and fluid mechanics models of storms.

Strictly speaking, the wind statistics should consist of three aspects: the speed, the direction, and the duration.

In the present practice, however, long term statistics are sought in terms of wind strength versus return period. A number of methods have been developed in the past to estimate long term statistics of extreme values; among them, Gumbel's first asymptotic distribution (sometimes referred to as a Fisher-Tippett Type 1 distribution (Ref. 651, Ref. 648, Ref. 661, Ref. 663) and Frechet extreme value distributions (sometimes referred to as a Fisher-Tippett Type II distribution, Thom. 1973 a, 1973 b) have been extensively used. The Gumbel's first asymptotic distribution is not bounded. This means that the expected maximum value will tend to grow without limit as the interval of prediction approaches infinity. Thus, if this distribution has any bias, it is in the direction of overestimation. The Frechet distribution is related to the first type by an exponential transformation. Figures 3.1 and 3.2 illustrate, respectively, the two types of distributions when applied to wind predictions. The method of Gumbel's distribution is introduced here. The method of applying Frechet distribution is similar in principal and the readers are referred to references cited above.

Gumbel's first asymptotic distribution function has two basic features: first, if the initial distribution is of exponential type such as normal distribution, then the trend of logarithmic increase is a straight line; secondly, the asymptotic distribution function of this type, i.e., when number of trail becomes large, is a universal one and has the form

$$F(x) = e^{-e^{-y}} \tag{3.1}$$

where

$$y = \alpha(x-u) \tag{3.2}$$

is the reduced variate of largest value, and x is the actual largest value. The parameter u is the mean value, and $1/\alpha$ is the measure of dispersion; both of them have to be determined by experiment. The return period according to the asymptotic distribution is

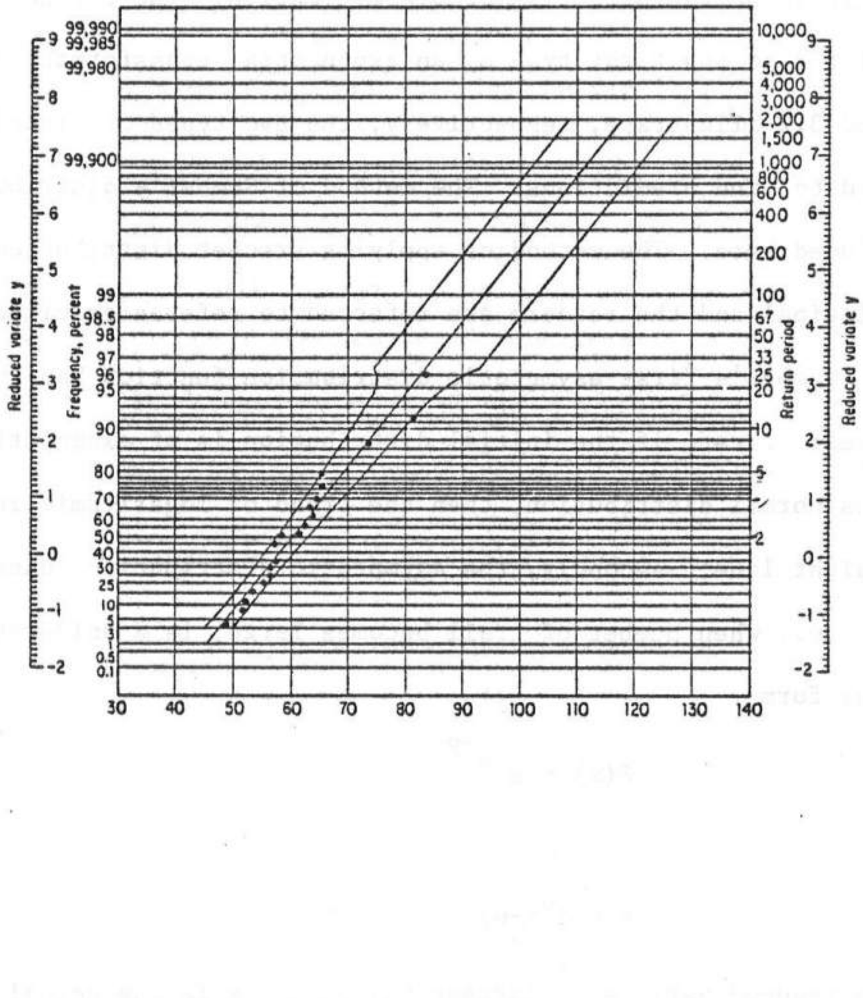


Fig. 3.1 Prediction of Extreme Wind by Gumbel's Distribution
(Ref. 661)

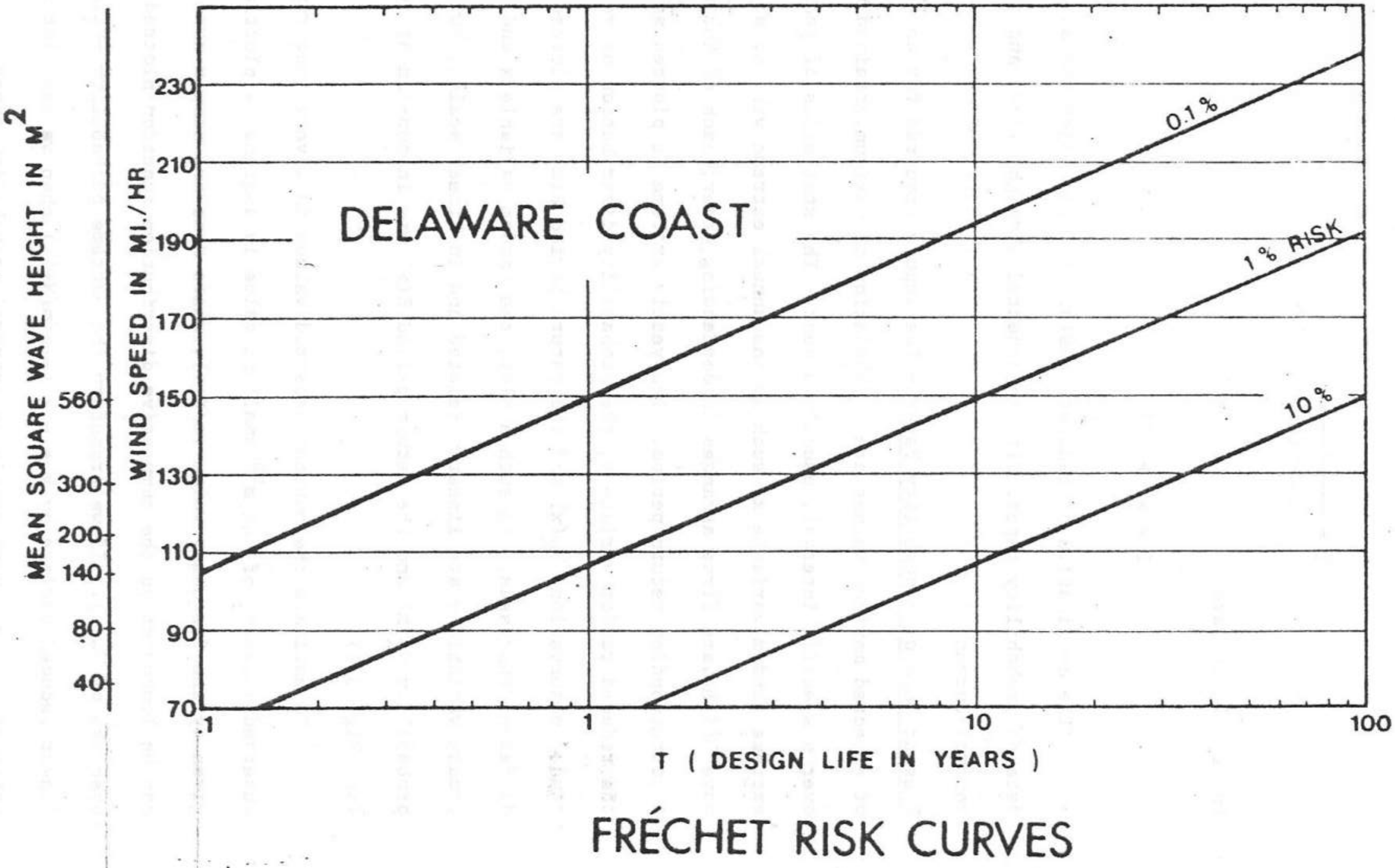


Fig. 3.2 Prediction of Extreme Wind by Fisher-Tippett Distribution

$$T = \frac{1}{1-\mathrm{Exp}(e^{-y})} \tag{3.3}$$

For T > 7, we have

$$T = e^{y} + 1/2 \tag{3.4}$$

The application procedures consist of three major steps: present data on probability paper, fit the expected straight line, and construct control curves.

Present Data on Probability Paper – The inputs required for an estimate of expected maximum values are a tabulation of maximum attained values over a specific interval, usually a year. The statistics of yearly extreme random variable x, such as the annual extreme wind or significant wave height,are first arranged in descending order; each of the data has a corresponding return period. The yearly extreme is plotted as the ordinate, the reduced random variate y, the probability distribution of yearly extreme random observations $W_X(x)$ and the return period R(x) are plotted as three different abscissas. In such a plot, the random variable x and the reduced random variable y are linearly related and in linear scales, but the probability $W_X(x)$ and the return period R(x) are in non-linear scales (see Fig. 3.1).

To estimate the maximum expected values in a very long run, each observed value x_m of the m^{th} smallest value is assigned a plotting position given by the frequency value (m/N+1), where N is the sample size. m/N+1 can be looked at as the cumulative distribution function plotted on the abscissa of $W_X(x)$. If we transform the extreme probability $-V_X(x)$ to linear reduced variate by $y = -\lg(-\lg(m/N+1))$ then we can plot every point of (y_m, x_m) very easily on extremal probability paper.

The Expected Straight Line - The observed points plotted on the probability paper cannot be used as a prediction tool. We hope that the theory applies to the observations and consequently there exists a linear relation between yearly extreme random observations x and reduced variate y. To obtain such a relation, two parameters α and u in Eq. (3.2) are to be estimated. These values are calculated from

$$\frac{1}{\alpha} = S_x/\sigma(N) \tag{3.5}$$

and

$$u = \overline{x}_N - \overline{y(N)}/\alpha \tag{3.6}$$

where $\overline{x}_N$ is the sample mean such that

$$\overline{x}_N = \frac{1}{N} \sum_{i=1}^{N} x_i \tag{3.7}$$

S_x is the sample standard deviation,

$$S_x = \sqrt{\overline{x}_N^{\,2} - (\overline{x}_N)^2} \tag{3.8}$$

$\sigma(N)$ and $\overline{y}(N)$ are the theoretical standard deviation and mean of the extreme value distributions both of which depend on the sample size. These values are presented in Table 3.1.

Substituting these values into the equation of the return period, the relation between return period and the extreme variate can be determined. It represents a straight line on the semi-log plot. This straight line represents the expected value for a certain return period, or can be interpreted as the line of 50% confidence of non-exceedance.

Table 3.1

(N) and $\overline{Y}$(N) as Functions of Sample Size For Extreme Value Statistics

Means and Standard Deviations of Reduced Extremes

N	$\overline{y(N)}$	$\sigma(y)$	N	$\overline{y(N)}$	$\sigma(y)$
10	.4952	.9497	35	.5403	1.1285
11	.4996	.9697	36	.5410	1.1313
12	.5035	.9833	37	.5418	1.1339
13	.5070	.9972	38	.5424	1.1363
14	.5100	1.0095	39	.5430	1.1388
15	.5128	1.0206	40	.5436	1.1413
16	.5157	1.0316	41	.5442	1.1436
17	.5187	1.0411	42	.5448	1.1458
18	.5202	1.0493	43	.5453	1.1480
19	.5220	1.0566	44	.5458	1.1499
20	.5235	1.0628	45	.5463	1.1519
21	.5252	1.0696	46	.5468	1.1538
22	.5268	1.0754	47	.5473	1.1557
23	.5283	1.0811	48	.5477	1.1574
24	.5296	1.0864	49	.5481	1.1590
25	.5308	1.0915	50	.5485	1.1607
26	.5320	1.0961	51	.5489	1.1623
27	.5332	1.1004	52	.5493	1.1638
28	.5343	1.1047	53	.5497	1.1653
29	.5353	1.1086	54	.5501	1.1667
30	.5362	1.1124	55	.5504	1.1681
31	.5371	1.1159	56	.5508	1.1696
32	.5380	1.1193	57	.5511	1.1708
33	.5388	1.1226	58	.5515	1.1721
34	.5396	1.1255	59	.5518	1.1739

Table 3.1 (Cont'd)

Means and Standard Deviations of Reduced Extremes

N	$\overline{y(N)}$	$\sigma(y)$	N	$\overline{y(N)}$	$\sigma(y)$
60	.5521	1.1747	88	.5583	1.1994
62	.5527	1.1770	90	.5586	1.2007
64	.5533	1.1793	92	.5589	1.2020
66	.5538	1.1814	94	.5592	1.2032
68	.5543	1.1834	96	.5595	1.2044
70	.5548	1.1854	98	.5598	1.2055
72	.5552	1.1873	100	.5600	1.2065
74	.5557	1.1890	150	.5646	1.2253
76	.5561	1.1906	200	.5672	1.2360
78	.5565	1.1923	250	.5688	1.2429
80	.5569	1.1938	300	.5699	1.2479
82	.5572	1.1953	400	.5714	1.2545
84	.5576	1.1967	500	.5724	1.2588
86	.5580	1.1980	1000	.5745	1.2685

Where $y_u = -\lg(-\lg \frac{u}{N+1})$

$$\overline{y(n)} = \frac{1}{N} \sum_{u=1}^{N} y_u$$

$$\sigma^2 (y) = \overline{y^2(N)} - \overline{y(N)}^2$$

If confidence levels other than 50% are desirable, control bands can be located on either side of the extremes.

Control Curves – The purpose of using control curves is very similar to the use of significant levels in a normal distribution. For example, in a normal distribution with zero mean for 90% confidence, the observation will be within the band between $x = -1.645$ to $x = 1.645$. The real observations within this range is allowable under .9 probability. In a similar way, two control curves can be drawn in the upper and lower part of an estimated straight line. Different probabilities will have different control curves. The wider band the control curves, the larger probability it will be. The control curves is a test of the theoretical fitness of the past observations. It also can tell the deviation of future observations from the estimated straight line with certain probability.

It is difficult to find asymptotic errors of order statistics directly, but indirectly to construct control curve is possible through the standard errors of normal distribution.

The construction of control curves should be separated into three parts:

a) The first set of control curves related to the body of the data is located on either side of the x-ordinate. When the total number of the sample is large, the asymptotic errors of order statistics can be obtained from the standard errors under the condition of normal distribution. Gumbel has proven that this convergence will be fast if the initial distribution $F_x(X)$ is within the range of .15 to .85. There are different standard errors corresponding to different probabilities. Table 3.2 presents these standard errors. When N is known and α is estimated, the standard errors

Table 3.2

Standard Errors for Gumbel's Asymptotic Distribution Function

Probability $W_X(\chi)$	Reduced Standard Error $N\,\sigma\,(y_m)$	Reduced Variate y
.15	1.2548	-.64
.20	1.2427	-.48
.25	1.2494	-.33
.30	1.2685	-.19
.35	1.2981	-.05
.40	1.3386	.08
.45	1.3845	.23
.50	1.4427	.37
.55	1.5130	.51
.60	1.5984	.67
.65	1.7034	.84
.70	1.8355	1.03
.75	2.0069	1.25
.80	2.2403	1.50
.85	2.5849	1.82

Table 3.3

Values of $f_i(N)$ As a Function of Rank

	N	
i	1	2
1	1.140	3.07
2	0.754	1.78
3	0.589	1.35
4	0.538	1.17

$\sigma(X_m)$ will be known. Add and subtract these values from corresponding points on the expected straight line, the fifteen points can be obtained on each side. Connecting them with a smooth curve, the first part of control curve is constructed.

b) The second part of the control curves is located on either side of the top extremes of X at the distance

$$x_i = f_i(N)/\alpha$$

Where the function $f_i(N)$ depends on the index of rank and on the number of standard deviations of the dispersion, Table 3.3 applies. In practice only the values corresponding to $i = 1$ and 2 are of interest.

c) Since there is no theoretical basis which can be followed when the cumulative probability is beyond N/N + 1, the prediction is therefore less reliable and must be used with caution. The current practice suggested by Gumbel is to draw two straight lines parallel to the theoretical straight line for that range.

Through these three steps, connecting all points, the control curves can be fully constructed.

From the above method of data analysis, one establishes the extreme value versus return period with certain degrees of confidence level (i.e., one standard deviation yields 68.3% confidence of non-exceedance, two standard deviation yield 94.5% confidence of non-exceedance, and so on). These extreme values and return periods, together with desired lifetime of the structure, enable one to establish the risk factor. The following equations apply:

$$T_e = -T_d \ln(1 - R) \tag{3.9}$$

where T_e is the desired structure life and R is the risk factor in percentage.

For example, one desires to have a structure life time of 50 years, at a 3% risk, then the design return period should have

$$T_d = - \frac{T_e}{\ln (1 - R)} = 1{,}500 \text{ years}$$

In analyzing wind loadings on structures, gust factors are often considered. However, for breakwater design, the wind information is usually used to estimate wave conditions and storm surges at the site, short-duration peak wind can generally be neglected.

As to the fluid mechanics models of storms, one usually is interested in high intensity events such as the representation of a toronado. A toronado may result in unusually high-intensity wind of short duration and abnormal high water in limited regions due to pressure drop. A common theoretical representation of a toronado is the so-called Rankin Vortex model (Fig. 3.3). The velocity and pressure distributions can be estimated by the following equations:

$$V_r = 0$$

$$V_o = Kr \qquad r \leq r_o \tag{3.10}$$

and

$$V_o = K\frac{r_o^2}{r} \qquad r \geq r_o$$

where K is a constant and r_o is the radius of rotational core.

$$p = p_o - \frac{p}{1 + \frac{r}{r_o}} \tag{3.11}$$

where p_o atmospheric pressure at the outer-most circular isobaric line of the storm.

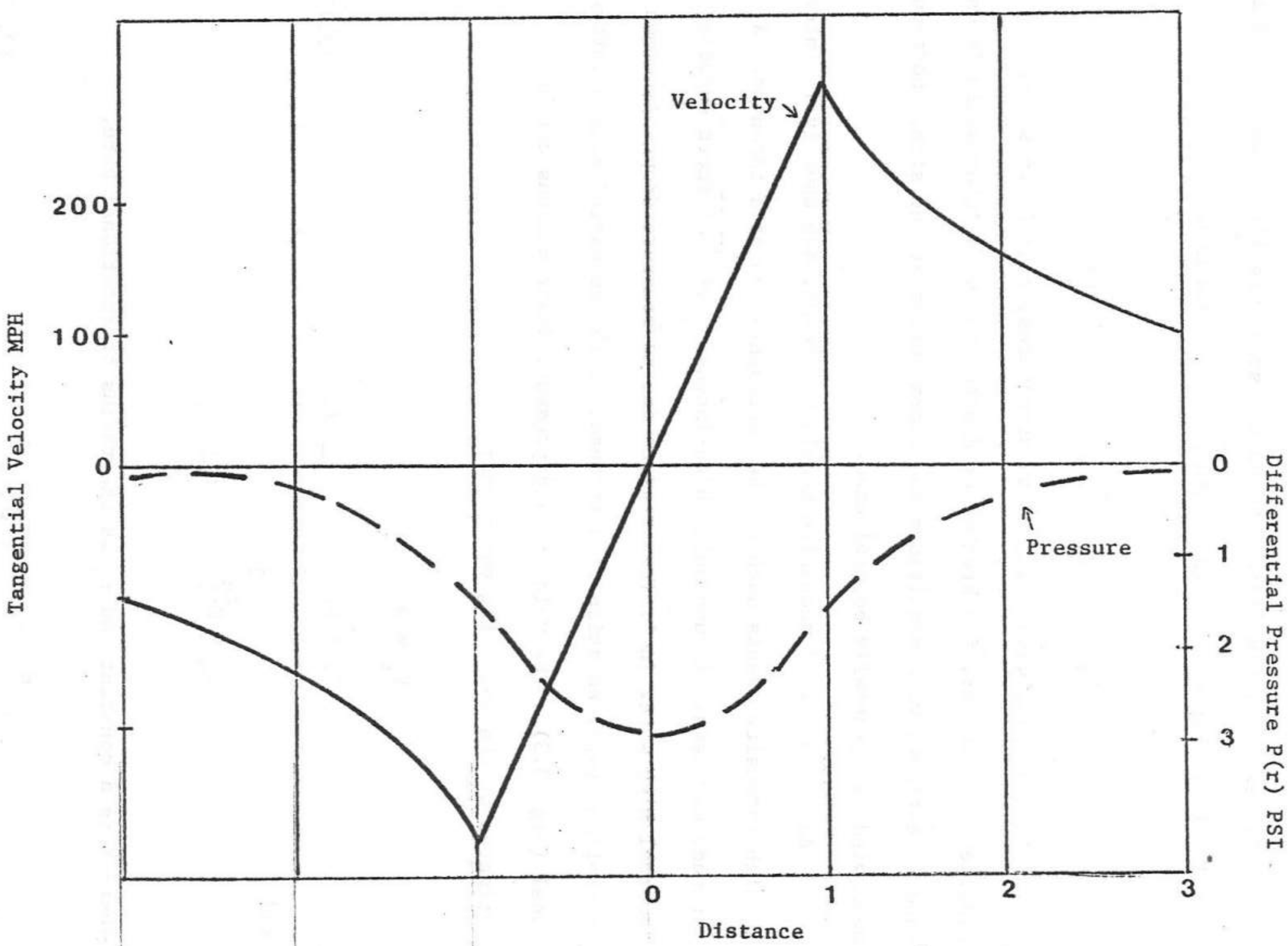


Fig. 3.3 Rankin Vortex Model of Toronado

3.2.1.2 Waves

At present, waves are probably the single most important factor being considered in breakwater design. Effects of waves on the breakwaters have also received most of the attention from researchers and designers. Factors pertaining to waves are their height, periods, duration, and direction. Basically, waves are expressed either as monochromatic trains of regular shape or as random phenomena of irregular shape.

The wave height is the primary factor concerning the stability, damage, or runup and overtopping incurring on the breakwater. Many existing stability criteria are expressed strictly in terms of wave height (Refs. 217 and 304, Ref. 431, Ref. 347).

Until recently, the effects of wave periods have been generally neglected in the structural design of breakwater although it has been recognized that wave length (sometimes expressed in terms of wave steepness) is an important factor for wave runup and overtopping. As previously discussed, one of the primary damaging mechanisms on the lee side of the breakwater is the overtopping waves. Therefore, neglecting wave periods in breakwater design is not totally justified. In addition, wave periods are closely associated with wave breaking; breakers of various forms will result in different loadings on structures (Figs. 3.4 and 3.5). Also, it has been reported recently (Ref. 477) that the effects of wave periods appear to be more significant in the later stage of damage on breakwaters.

The storm duration is another factor which is lightly treated in breakwater design at the present time. This factor is of considerable importance in the advanced stage of damage (see, for example, the breakwater

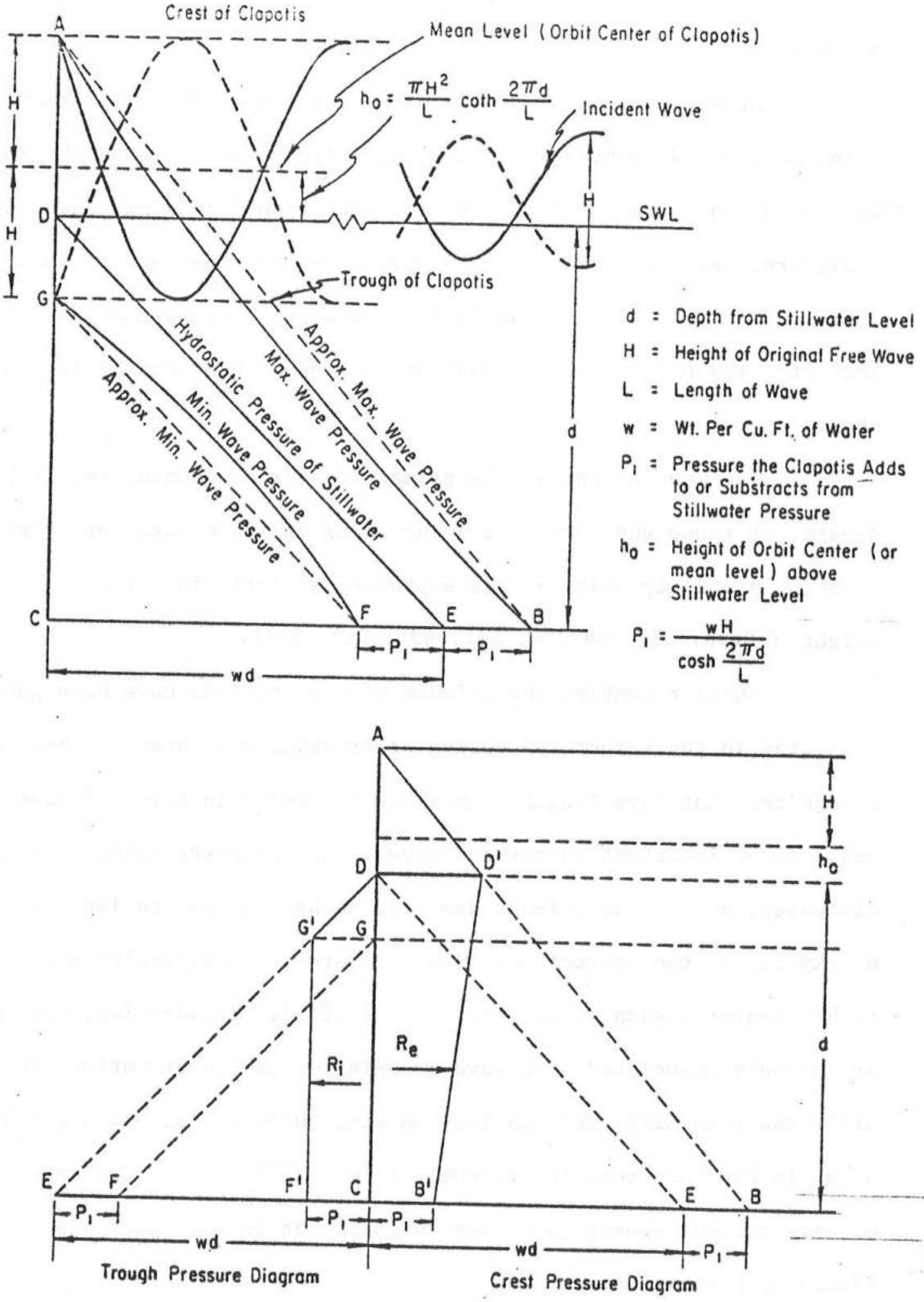


Fig. 3.4 Loading on Marine Structure Due to Non-Breaking Waves

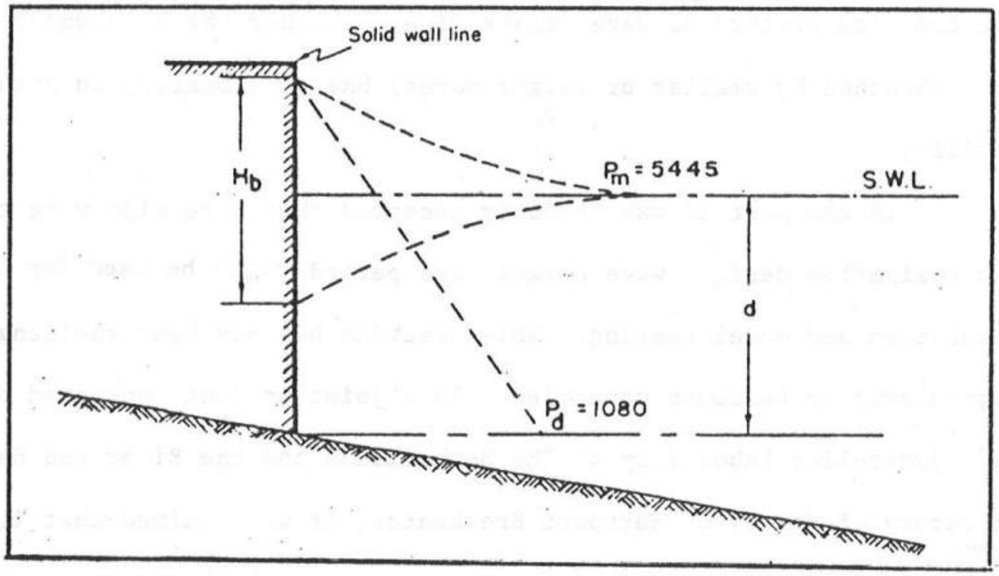


Fig. 3.5 Loading on Marine Structure Due to Breaking Waves

at Praaia da Vitória, Portugal). Above a certain level of wave height, damage of the structures will increase if sufficient duration is allowed. Below this level the damage to the structure stops at a certain limit where a new equilibrium profile is obtained. Therefore, the design duration should be established to ascertain that the damage would not endanger the integrity of the structure. There is also recent speculation that the time history of wave attack (i.e., whether the breakwater is first attached by smaller or larger waves) has some bearing on structure stability.

In the past it was commonly accepted that a regular wave train with designated design wave height and period could be used for both design calculation and model testing. This practice has now been challenged, particularly in Eurpoean countries. In a joint project conducted by the Delft Hydraulics Laboratory of The Netherlands and the River and Harbor Laboratory of Norway on Europort Breakwater, it was claimed that irregular waves seemed to represent a more severe wave attack than regular waves with heights equal to the significant wave heights of the irregular waves. Since facilities that are capable of generating irregular waves are rather limited and the test results were also limited, the investigators cautiously qualified their conclusions to the particular breakwater. It is also not clear whether the severer attack of irregular waves is due to the portion of higher waves (rather than the significant waves) or due to the random nature of the incident waves.

To determine the design wave condition, methods similar to those described in the previous section dealing with extreme values can be applied provided wave data is available. If such information is not available an indirect method;such as that based on the wind wave spectrum, might be used. The design wave may then be chosen using either the significant wave height (average wave height of the highest one-third of the waves in the total wave spectrum) or the extreme wave height (average wave height of the highest one-tenth of the waves in the total wave spectrum).

In the event that statistical properties of storm waves or random waves are employed as the design input, one-dimensional wave spectrum (wave energy spectrum in the frequency or period domain) is presently used. There are a number of variations in presenting the wave spectrum. For deep water cases, the S-M-B method (Sverdrup-Munk-Bretschneider) and the P-N-J method (Pierson-Newmann-James) are among the popular ones.

Just as the S-M-B method is based on empirical wave data, so it is with the P-N-J method. However, the wave data utilized were not the same in the two methods, and the method of analysis of the data was different. The method of analysis of the data for the S-M-B method consisted in determining the significant wave height and period, which in turn were related to windspeed, fetch length, and wind duration. Consequently, the P-N-J method can be used to predict the spectrum of waves, from which one may obtain the significant wave height as well as the statistical distribution of the waves. The S-M-B method on the other hand is used to predict the significant wave height, from which it is possible to obtain the wave spectrum and the statistical distribution of the waves. Both methods utilize the distribution function derived theoretically by Longuet-Higgins (Ref.790). This distribution function is in very close agreement with the empirical relationships given by Putz (Ref. 793) based on the analysis of 25 ocean wave records. Consequently, when both S-M-B and P-N-J methods predict exactly the same significant wave height, then the two methods result in exactly the same distribution of waves.

Details of wave spectrum presentation, estimation and forecasting can be found in Pierson et. al. (Ref. 646)Bretschneider (Ref. 640, 643) For wave spectrum estimation in shallow water, the reader is referred to Bretschneider (1954); Sakamoto and Izima (1964).

In breakwater design, the evaluation of water particle velocity and water particle acceleration associated with the design waves is sometimes required. This exercise entails the selection of wave theories. Figure 3.6 summarizes the state of the art of the applicabilities of various wave theories for different conditions. The various expressions for various wave theories are too numerous to list here. A good source of wave theories is the book <u>Oceanographical Engineering</u> by Wiegel (Ref. 708).

Another important consideration in breakwater design is the determination of the concentration of wave energy. There are numerous examples of breakwater damages that are localized at places where the wave energy is concentrated. For instance, Goda (Ref.291)has cited many cases of damage of this type which he termed "meandering damage" (see Section 2.4).

The phenomenon of wave energy concentration is basically due to wave refraction which is a process by which the direction of a wave moving in shallow water at an angle to the contours is changed. The part of the wave advancing in shallower water moves more slowly than that part still advancing in deeper water, causing the wave crest to bend toward alignment with the underwater contours. Procedures to construct refraction diagrams are well developed. Two methods -

the Wave-Orthogonal method and the Wave-Front method - are commonly used. Graphical methods were presented in Shore Protection, Planning and Design Manual (1974); Design Standard for Port and Harbor Structures (1968); and in a number of articles (Ref. 796; Ref. 787). Computer programs were also developed for three-dimensional cases (Ref. 789; Orr, 1969; Ref. 794).

3.2.1.3 Tides, Storm Surges and Water Depth

In breakwater design, both water depth and water depth variation are important factors. The water depth affects the breakwater height, runup and overtopping, scouring, and the characteristics of waves impinging on the structure. The water surface variation determines the zone of damage , scouring, and critical loading conditions. At certain levels of water the forces may be due to unbroken waves, whereas at other water levels the full effect of breaking waves must be resisted.

The major factors that govern the water depth and water depth variations are the bottom contour, tidal variation, and storm surge. The range of tidal variation is not only important to loadings on the structure but also important to the selection of the types of breakwaters to be constructed. For instance, for locations where both the tidal range and the water depth are modest, rubble mound breakwater may prove to be a good choice. On the other hand, when the tidal variation is large, composite breakwater or vertical-wall breakwater may be more desirable. Because of its slowly varying nature, the factor of tides in breakwater design is almost entirely related to its effect on water surface elevation, and the dynamic action of the tide could be completely ignored unless in areas where the tidal currents are unusually strong, such as in the Cook inlet

Alaska. Astronomical tides are repetitive phenomena, thus predictive from past occurrences. Determination of tidal variation does not usually present a serious design problem. On the other hand, the meteorological tide, often referred to as storm surge, is highly irregular in magnitude and frequency of occurrence in severe weather. Therefore, determination of design storm surge often represents an important task in breakwater design.

Two different methods are currently employed to analyze storm surge problems. One of these methods is again utilizing statistical analysis of extreme values as discussed earlier. To use such a method one must, of course, have available historical data. The other method is to predict the storm surge through meteorological information.

Factors that influence the maximum storm surge are wind stress, Corialis force, atmospheric pressure anomalies, resurgence, resonance, impact of rainfall, wave setup, among others. There is also the problem of convergence, divergence and bottom configuration changes. Some of these factors interact with each other such as wind stress and Corialis force and wave setup and bottom configuration. Others contribute independently. Since storm surge is a complicated but important problem, there have been a great number of contributions on the subject. There are at least over 400 contributions on wind tide and storm surge and related subjects (Ref. 646). These contributions covered topics of actual observations, field measurements, analytical studies and numerical computations.

Although the contributions on the subject are voluminous, there is no single universally acclaimed one that is widely accepted by designers. For instance, among the various computer programs for numerical calculation of storm surge, the Hansen numerical method, known as one of the H-N (hydrodynamical-numerical) methods, was used for North Sea Storm surge; Masamoci Miyazaki(Ref.791)developed a numerical program for the Gulf of Mexico; Jelesnianski (Ref.653) developed a numerical program for the case of no bottom stress; the numerical model developed by Pearce (Ref. 654) was for Hurricane Camille; the Reif and Baoline's (Ref.658) model was for the Bay area.

For the reason that the problem of storm surge is different for different cases,Bretschneider(Ref.788)made an attempt to classify the design problem so as to aid in selection of appropriate formulas and techniques most suitable for the problem. The classification is shown in the following Table:

CLASSIFICATION OF DESIGN PROBLEM

A. Enclosed Lakes and Reservoirs

1. Rectangular channel, constant depth
2. Regular in shape
3. Somewhat irregular in shape
4. Very irregular in shape

B. Off Coast or on Continental Shelf

1. Bottom of constant depth
2. Bottom of constant slope
3. Slightly irregular bottom profile
4. Irregular bottom profile

C. Coastline

1. Smooth coastline
2. Coastline somewhat irregular
3. Jagged coastline

D. Behind Coastline

1. Low natural barriers
2. Medium-high natural barriers
3. High natural barriers

E. Open Bays and Estuaries

1. Entrance backed by long estuaries and with tidal flow moving freely past entrance
2. Entrance backed by short estuary and with tidal flow moving freely past entrance
3. Entrance constructed sufficiently to prevent free movement of tidal flow past entrance

Discussions and suggested methods of analysis were also presented by Bretschneider (Ref. 788).

3.2.1.4 Current

Current is an indirect factor; that is, in nearshore area where the breakwater is usually constructed the current is induced by waves, tide and wind. Large scale oceanic currents are of no particular concern in breakwater design.

Currents affect breakwaters in a number of ways. They may cause scouring of breakwater foundation by carrying away bottom material. They exert drag force on the structure as a whole and on individual units. The interaction of waves and currents may aid in wave breaking and runup and overtopping. The interaction of currents and breakwaters may cause shoreline modification. When a breakwater is partially damaged, currents are the major agency of eroding the core away.

At present a number of methods can be employed to estimate the three major components of currents - the tidal current, the wave-induced current, and the wind stress current. However, in design practice, one often relies on field measurement. Such a practice is indicative to the fact that nearshore current is a complicated phenomenon and that although the effects of currents on breakwater structures are always mentioned in design, quantitative prediction is often lacking.

3.2.1.5 Ice

Only a handful of literature can be found that deals with the general effects of ice on coastal structures. Most of the literature (Ref. 784; Ref. 783; Ref. 785; Ref. 786) are descriptive

There is practically no information on the effects of ice on breakwater. Even in countries like Norway and Japan where some of their ports are located at high latitudes, no design information has been documented on ice effects.

3.2.1.6 Geological Information

Geological information required in breakwater design includes detailed sounding of bottom topography and the soil condition. The bottom topography should not only include the construction site but also extend to the near field so that wave propagation and refraction can be determined.

To determine soil characteristics, core samples are usually taken. Since most of the breakwaters are massive earth structures, it is not uncommon to require deep cores 150 to 200 ft. The following soil characteristics are commonly determined through laboratory testing:

Physical properties -

- unit weight
- grain size analysis
- permeability
- cohesion
- porosity

Mechanical properties -

- consolidation
- shear strength
- pore pressure
- penetration resistance

Dynamic Properties -

resonant column test

dynamic strain-controlled cyclic triaxial tests

cyclic static triaxial tests

All these tests are required to determine whether the foundation is adequate for the structure or any improvement may be required. All the above tests are quite standard and are explained in most standard soil mechanics books.

One of the characteristics unique to sandy soil material is the property of liquefaction. The exact nature of liquefaction is not well know and the term is loosely referred to the phenomenon of excessive settlement of foundation and weakened soil strength under dynamic(in particular, repetitive) loadings which cause the soil to acquire certain degrees of fluid-flow type motion.

If saturated sand is subjected to ground vibrations, it tends to compact and decrease in volume; if drainage is unable to occur, the tendency to decrease in volume results in an increase in pore pressure, and if the pore pressure builds to the point at which it is equal to the overburden pressure, the effective stress becomes zero, the sand loses its strength completely, and it develops into a liquefied state.

To assess the susceptibility of liquefaction and its associated damage potential to superstructures, one needs to answer at least the following questions:

1. What is the condition that will result in liquefaction?
2. What is the state of pressure distribution during and following liquefaction?

3. What is the expected duration of liquefaction?

Unfortunately, none of these questions can be answered adequately within the present state of the art. Prediction of liquefaction potential is a rather recent engineering endeavor. This is evident from the list of references provided. In the United States the analysis of liquefaction mainly follows the method developed at the Earthquake Engineering Research Center, University of California at Berkeley (see list of references).

The pertinent soil information required to perform evaluation of soil liquefaction potential includes:

mean grain size (D_{50}) and size distribution

critical valid ratio

triaxial cyclic test

shear test

confinement pressure

standard penetration test

3.2.1.7 Earthquake and Tsunamis

At present, there is practically no information on using earthquake as impact for breakwater design. In Japan, one of the most earthquake-prone countries, it is generally assumed that if a breakwater is so designed as to withstand severe wave action, it can also survive earthquakes with no significant damage.

Recently, Japan has experienced three severe earthquakes:

a. Niigata earthquake on June 16, 1964 (M=7.5)

b. Tokachi Oki earthquake on May 16, 1968 (M=7.8)

c. Nemuro Hanto Oki earthquake on June 17, 1973 (M=7.4)

Post earthquake surveys showed that despite some severe damages suffered by port structures, mainly quay walls, the breakwaters sustained very little damage due to direct earthquake loading. Typical example of breakwater damage in slight sinkage of caisson on the rubble mound. The quantity of sinkage was of the order of 10-30 cm.

If an earthquake is generated in far field at offshore area, Tsunami may be crested. Tsumanis, in the ferm of long-period and low amplitude waves is imperceptive in the open ocean. However, when it propagates into shallower water and encounters obstructions such as breakwaters, water tends to pile up and results in great water level difference on the seaside and leeside of the structure. This great water level difference is often a source of concern. An example of failure due to Tsunami is found in the Port of Hachinohe in Japan during the 1968 Tokachi Oki earthquake. Caissons of 4.5 m wide, stable at the time of the earthquake, were overturned due to water level difference that reached in excess of 4 m. A total of 336 m caisson has been breached.

3.2.2 Structure Factors

3.2.2.1 Classification

Functionally, there are two major classes of breakwaters. The first class is a structure that depends only on its mass to break up waves. The second class of breakwaters utilizes shape and materials to give it an "ability" or "characteristic" that tends to break up waves. Most major breakwaters are of the second class. In addition to these two classes of breakwaters which are structures, there are breakwaters that cannot be quite classified as structures, such as pneumatic breakwaters or air-bubble breakwaters. Only breakwaters that are structures are dealt with in this section.

As far as shape is concerned, there are also two general classes of breakwaters. Breakwaters with vertical faces constitute one class. These depend on their reflecting characteristics to be effective. Sloping breakwaters are the second class and they cause the waves to break or be partially reflected.

One type of sloping breakwater is the rubble mound breakwater (see Fig. 3.6). This is the most common breakwater found in the United States. Almost all the major breakwaters in this country that combat heavy seas are of rubble mound type (Kawaikae, Kahulue, Crescent City, Humboldt, etc.). Only a few of the European major breakwaters fall in this category (Europort, Hook of Holland). Rubble mound breakwaters are best suited for areas where stone is plentiful and there is a low tidal range. They can be built on almost any bottom condition, even thick soft layers, but they are usually restricted to a depth of less than sixty feet. Because of the manner in which the rubble mound breakwaters are constructed (basically through piling up stones of various sizes) they seldom result in catastrophic failures.

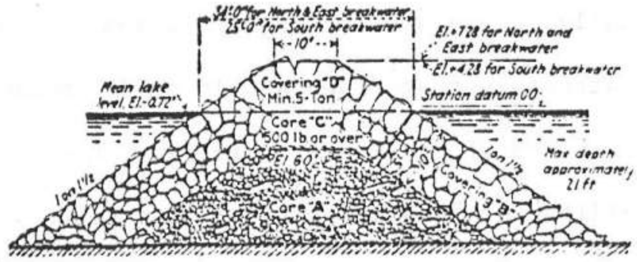


Fig. 3.6 Rubble Mound Breakwater

Since rubble breakwaters are porous, there is the possibility of undesirable water transmission. This is dependent on the thickness of the cover layer and the core, as well as the size of the stone.

A second type of breakwater that is similar to a rubble mound breakwater is a composite breakwater (Fig. 3. 7). A composite breakwater has a rubble mound base, but the part that endures the wave forces is a solid superstructure. Composite breakwaters are especially good for deep water, where the depth makes a rubble breakwater economically unfeasible. They are also effective in areas of high tidal variations. Major breakwaters in Japan and European countries largely fall into this category (see Chapter 2).

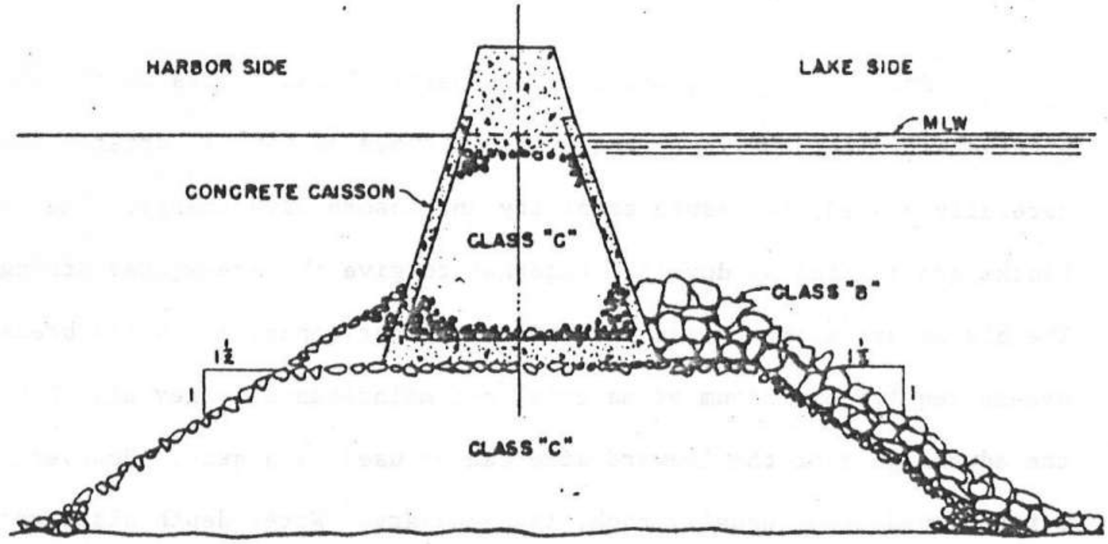


Fig. 3.7 Composite Breakwater

In composite breakwaters, the super structure is the most critical part, simply due to the wave action it must endure. The base of the super structure can be either at mean low water or well below mean low water. In the former case, the base is more subject to scouring action by the waves. In either case, failure can occur by overturning or sliding or differential settling. Unlike rubble mound structures, major catastrophic failures of composite breakwaters occurred a number of times in the past. Damage repairing is not a simple case of replenishing rubble but often represents a major task.

A composite breakwater can have a variety of super structure forms. The superstructure can be a solid monolithic wall, a caisson structure, or various types of blockwork. It can be sloping, straight, or stepped. Composite breakwaters can also have a solid cap on top for added stability.

One type of breakwater in the vertical wall class is a solid breakwater (Fig. 3. 8) It uses various types of blocks, orderly and carefully placed, to insure stability and absorb wave energy. The blocks are jointed or dowelled together to give the breakwater strength. The blocks are either stone blocked or precast concrete. Solid breakwaters require a minimum of material and maintenance. They also have the advantage that the leeward side can be used as a quay. However, a firm foundation, usually rock, is necessary. Water depth also imposes some constraint. Failure to a solid breakwater is serious since it is usually due to overturning or sliding. Owing to the above reasons, the usage of vertical-wall type breakwaters is rather limited.

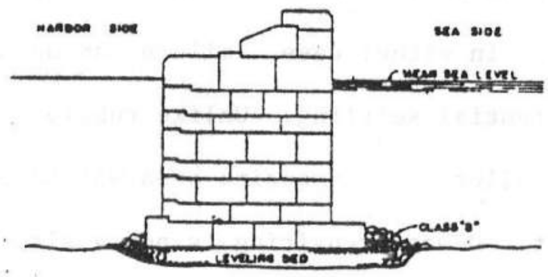


Fig. 3.8 Vertical-Wall Breakwater

Another type of breakwater is the caisson type (Fig. 3.9). Caissons are hollow reinforced cells or circular pipes that are embedded to a trench or into a rubble base. They are filled with either sand, gravel, or rocks and capped with a superstructure. The superstructure serves to break up the waves and can be either sloped, vertical, or stepped. The caisson can be sloped or vertical also. Sloping faces are the most desirable as they improve stability and help prevent overturning.

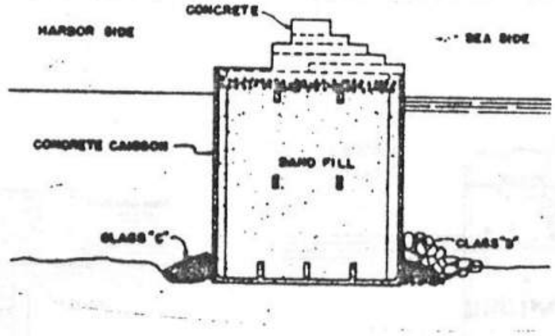


Fig. 3.9 Caisson-Type Breakwater

Caisson breakwaters are principally used in lakes at depths of less than forty-five feet. They can be constructed quickly on a soft bottom and at a low expense. Their biggest drawback is their inability to withstand breaking waves. Breaking waves apply a shock pressure much greater than the static pressure of a non-breaking wave. This dynamic pressure can batter and damage the caisson as well as cause increased pore water pressure in the foundation soil.

A variation of solid caisson breakwaters is the perforated wall type patented by the Canadian government. The installation at Baie Comeau, Quebec, Canada, utilizes this caisson at the seaward side to absorb and dissipate wave energy. There are current plans to utilize perforated breakwaters to combat the heavy North Sea waves.

Similar to the caisson breakwater, is the sheet pile breakwater (Fig. 3. 10). It is usually used in fresh water lakes where there is only moderate wave action. Big hollow steel pipes are driven into the bo-tom and filled with sand. They are usually capped with large stone.

Another similar breakwater is the pile crib (Fig. 3. 11). It is two parallel rows of pile driven into the bottom with rock fill in between. A concrete cap can be on top and usually is. Pile crib breakwaters are very economical, but suitable only for small waves at shallow depths, usually in lakes.

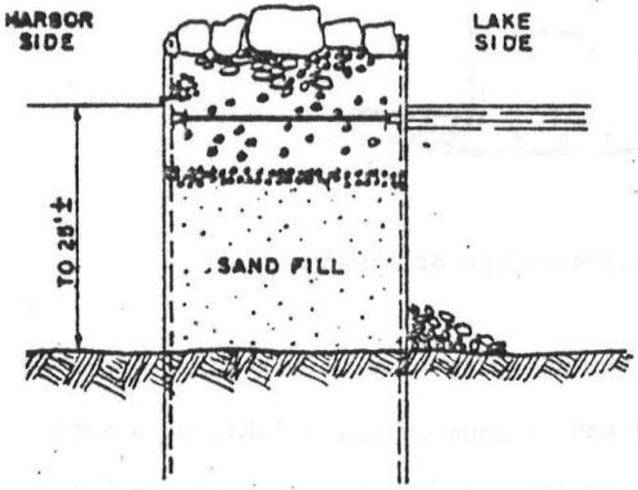


Fig. 3.10 Sheet Pile Breakwater

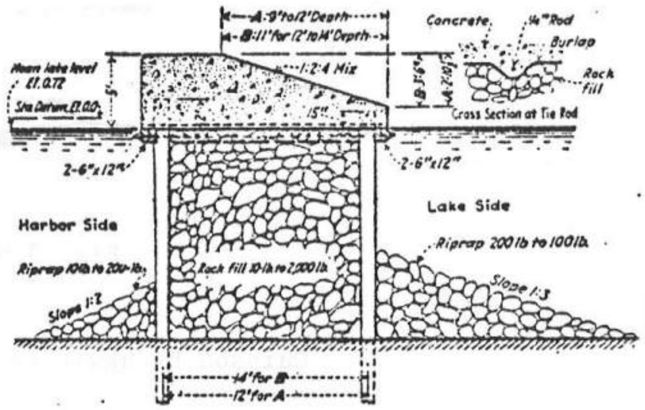


Fig. 3.11 Pile-Crib Breakwater

A-frame breakwaters are also very economical and limited to shallow waters. An A-frame breakwater is a series of concrete reinforced A-frames that stack side-by-side, similar to books on a shelf. Slots in the walls catch water and let it run off as the wave receeds, while protrusions break up and dissipate wave energy. Overtopping in small and reflected waves have less height energy than a vertical wall would produce. Some advantages of A-frame breakwaters are rapid construction, continually renewed harbor water, harbor side than can be a quay, light bearing load on the foundation, easy repair, and the unfinished parts are not damaged by storms.

Submerged breakwaters offer a potentially cheap solution to some problems. Generally they are rubble mound breakwaters below mean low water. These breakwaters can cause up to fifty percent incident wave energy loss. Thirty to sixty percent of the wave energy that is transmitted is transferred to a higher frequency than that of the incident wave.

Timber breakwaters are found on lakes and usually have sloping faces. They are not very durable or stable compared to stone or rock breakwaters.

Although many types of breakwaters are presented here, the major breakwaters actually fall into the first two categories - the rubble mound type and the composite type. In the following sections only these two types of breakwaters are discussed.

3.2.2.2 Rubble Mound Breakwaters

General Description

The rubble mound breakwater, as explained, is more or less a heterogeneous assemblage of natural stones of different sizes and shapes either dumped at random or placed in courses.

The cross-sectional shape of a rubble mound breakwater usually resembles a trapezoid. It has a core of relatively small stones surrounded on the exposed part by much larger stones, called armor blocks. The cover layer of armor blocks determines the stability of the structure; therefore, it is common to have armor blocks weighing thousands of pounds each. It is evident that the cover layer is the most important part of the cross section. Once the armor blocks are moved, the underlying core is exposed and quickly damaged by the waves.

The typical breakwater sections recommended by the U. S. Corps of Engineers (Ref. 1) are shown in Figs. 3.12 and 3.13 .

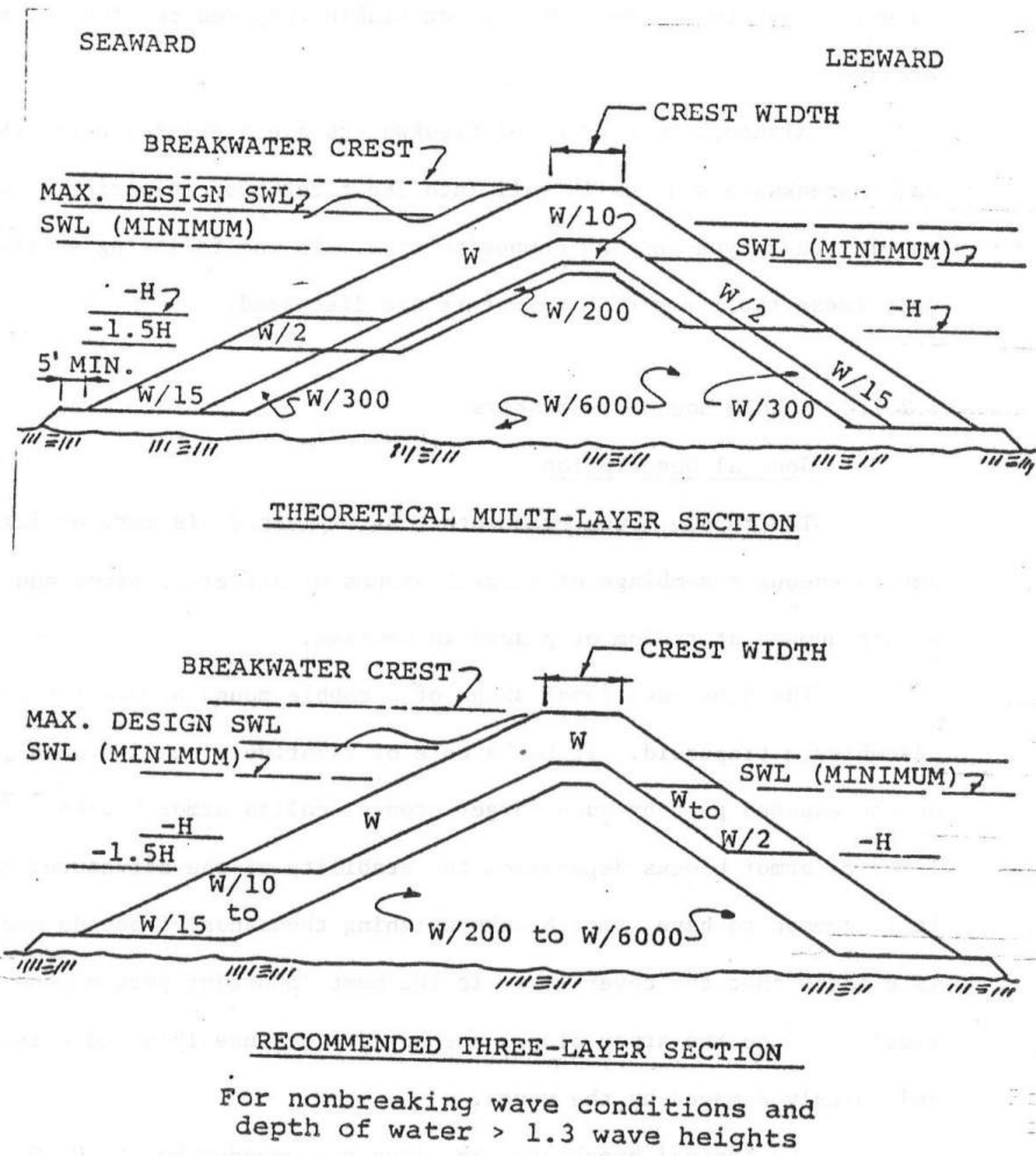


Fig. 3.12 Typical Deepwater Rubble-Mound Breakwater Section

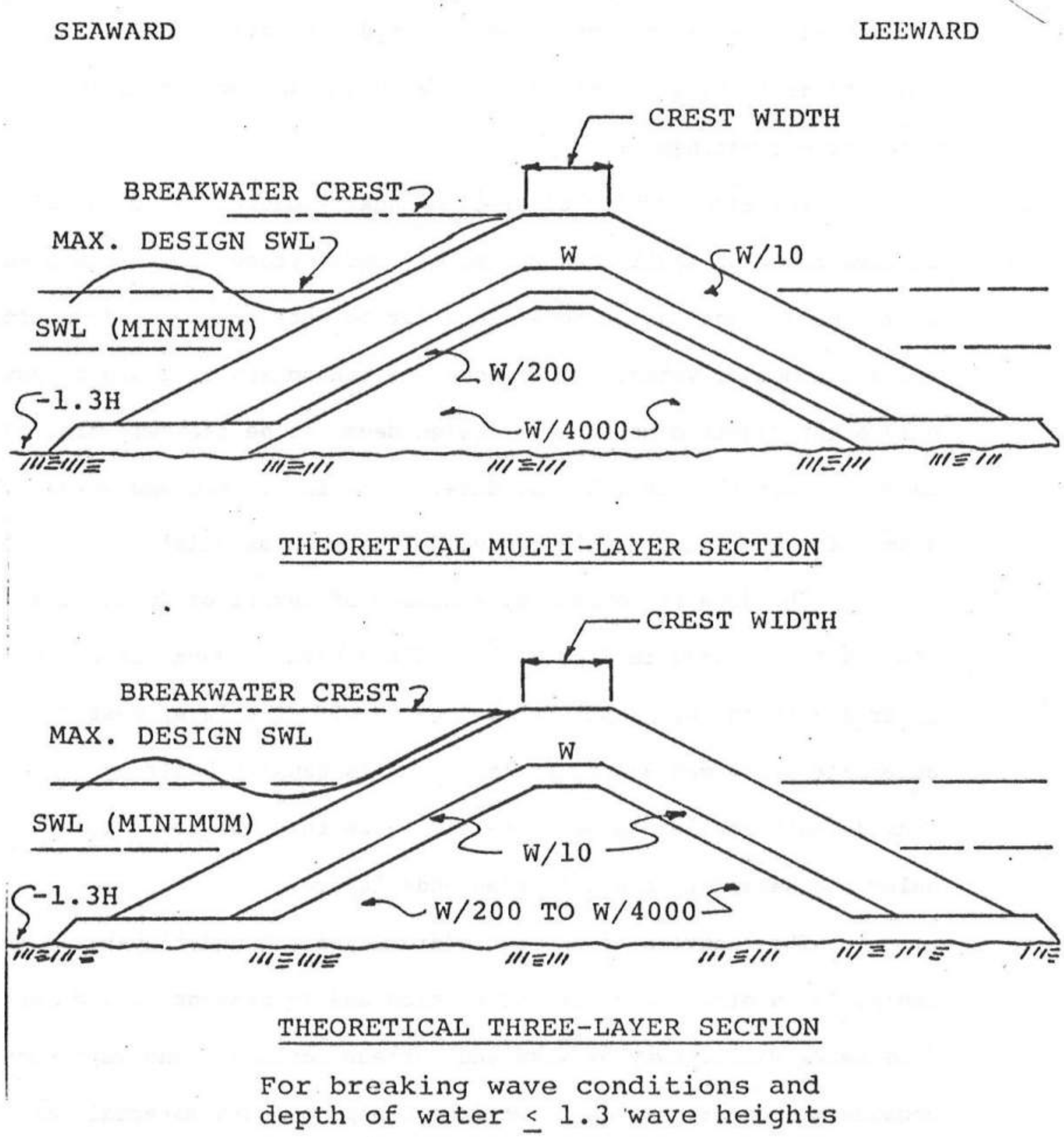


Fig. 3.13 Typical Shallow Rubble-Mound Breakwater Section

Core Cover Layers and Armor Layers

Rubble mound breakwaters have been, and still are, constructed in a great number of cross-sections dictated to some extent by site conditions but the general principal remains essentially the same. This consists of forming a core to the breakwater and providing it with protective coverings.

The core usually consists of quarry materials of various sizes, in some cases of spalls and refuse of random stone from a few pounds in weight to hundreds or more, in order to obtain maximum compactness and a minimum of voids. Other than the randomness of stone sizes, the only other requirement of core design deems to be the selection of material such that it will not deteriorate in air and sea water. Limestone, for instance, should be avoided as core material.

The core is covered by a number of layers of increasing stone size as illustrated in Fig. 3. 12 . The outermost cover is called armor layer and is the principal protective layer. The layer next to the outermost is termed secondary layer and is usually built by stones of considerably smaller (about one-half) size than the armor layer. Layers below the secondary are all named underlayers.

The function of the secondary layer and underlayer, as the names imply, is to offer secondary protection and to prevent core material from being wasted away by wave and current actions. The major design considerations are to ensure minimum escape of core material and at the same time to provide sufficient porosity to avoid high back pressure in the armor layer. Under current design practice, these layers are almost exclusively built by natural stones. The stones selected for construction should be structurally sound, durable, and hard. It should be free from laminations, weak cleavages, and undesirable weathering, and it should be of such character that it not disintegrate from the action of air, sea water, or handling and placing. In general, stones with high specific gravity are desirable. As far as size distribution is concerned, both uniform stone and graded stone are being used. It is recommended by U. S. Army Corps of Engineers that all stones should conform to the following test designations: apparent specific gravity, ASTM Designation: C127-59; and abrasion, ASTM Designation C131-55.

As to the selection of stone sizes for the secondary and under layers, U. S. Army Corps of Engineers (Ref. 1) provided some guidelines. These guidelines are pretty much followed by Japan and European countries. Figures 3. 12 and 3. 13 illustrate the recommendations by the Corps of Engineers. The classification of stone sizes, weight and dimensions are shown in the following table.

Wt. Tons	Avg. Cu. Dimen. (Feet)	Wt. Lbs.	Avg. Cu. Dimen. (Feet)	Wt. Lbs.	Avg. Cu. Dimen. (Inches)	Wt. Lbs.	Avg. Cu. Dimen. (Inches)	Wt. Lbs.	Equiv. Sphere (Inches)
1	2.3	100	0.85	5	3.75			0.0125	
2	2.9	200	1.07	10	4.75	0.50	2.00	0.025	0.8
3	3.3	300	1.22	15					
4	3.7	400	1.34	20	6.00	1.00	2.25	0.50	1.0
5	3.9	500	1.45	25					
6	4.2	600	1.54	30	6.75	1.50	2.50	0.075	1.1
7	4.4	700	1.62	35					
8	4.6	800	1.69	40	7.50	2.00	2.75	0.100	1.3
9	4.8	900	1.76	45					
10	5.0	1000	1.82	50	8.00	2.50	3.00	0.125	1.4
11	5.1	1100	1.89	55					
12	5.3	1200	1.94	60	8.50	3.00	3.13	0.150	1.4
13	5.4	1300	1.99	65					
14	5.5	1400	2.04	70	9.00	3.50	3.38	0.175	1.5
15	5.7	1500	2.09	75					
16	5.8	1600	2.14	80	9.50	4.00	3.50	0.200	1.6
17	5.9	1700	2.18	85					
18	6.0	1800	2.22	90	9.75	4.50	3.65	0.225	1.6
19	6.1	1900	2.26	95	10.00				
20	6.2	2000	2.30	100	10.25	5.00	3.75	0.250	1.7

NOTE: Dimensions based on stone weight of 165 lbs/ft^3.

Table 3.4 - Stone Weights and Dimensions

It is the general contention among the breakwater designers, and probably rightly so, that the armor layer is the most important structure element in rubble mound. Some believe that the success or failure of a mound breakwater depends almost wholly upon this layer - that is to say upon its composition, selection of material, size, layer-thickness, slope, levels, and its extension to underwater and underwater slopes.

Considerable research has been carried out in the past and is still going on at present concerning the design of armor layer. These include the search for new shapes and materials for armor units and the stability criteria of the armor layer. Both mass and shape of the armor blocks directly affect stability. By increasing the mass of an armor block, more force is required to lift it. The shape of the unit can reduce the drag force and bond the units together. Each armor unit can be assigned a coefficient of bonding which is a function of its shape and material. This coefficient reflects the ability of the unit to remain attached to surrounding blocks. It is clear that by altering either the shape or the mass of the armor unit, its tendency to be moved by waves changes.

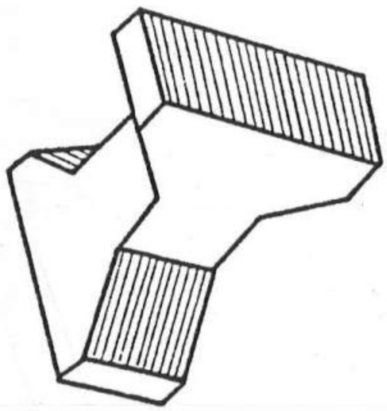
In addition to natural stones, many different artificial or prefabricated armor units are not available. Smooth and rough quarry stones are the two main kinds of natural armor blocks. Because they have no definite shape, they are placed in random or pell-mell fashion. Artificial or prefabricated units can achieve maximum bonding stability when placed uniformly. The tendency is the increasing utilization of prefabricated units, particularly, recently designed major breakwaters, because of their superior interlocking ability. Shapes of prefabricated armor blocks include cubes, modified cubes, tetrapods, quadripods, hexapods, tribars, Svee-blocks, Dolos, parallel pipes, etc. Figure 3.14 illustrates the various shapes of the artificial units. All these artificial armor units are made of precast concrete with or without reinforcement. There is no definite criterion, at present, as to whether reinforcement is essential or even is beneficial. A detailed discussion of using concrete armor units for breakwater structure was documented in a recent report (Ref. 434).

In most cases, the armor layer is the outermost layer of a mound breakwater. In a few isolated cases, these armor blocks are protected by gabions or a cover of stone asphalt.

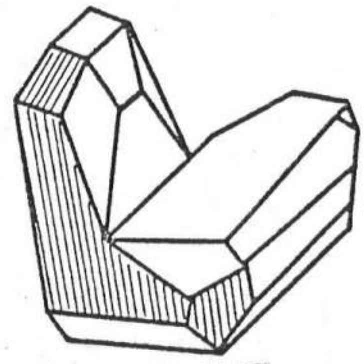
Shapes and Slopes

The cross-sectional shape of a mound breakwater is commonly known to be trapezoidal. Actually, there is quite a range of variations in shapes and slopes among different breakwaters. It is not uncommon to have breakwaters of composite slopes or composite slopes with underwater berm sections.

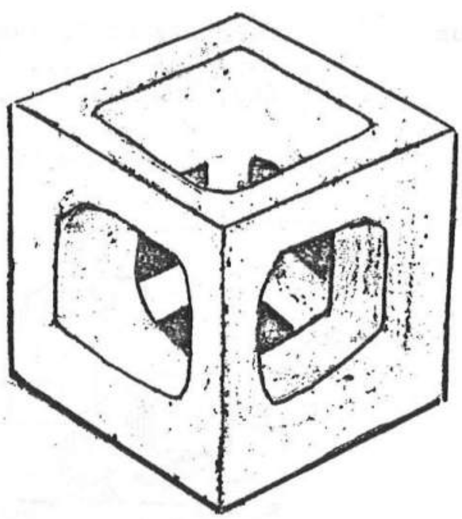
The underwater slope of the cover layer and its point of break are critical to wave breaking characteristics and to the location and extent of surge and backwash. The slope above the mean water level is important to wave runup and overtopping which in turn affect the stability of the breakwater. Breakwaters are usually designed with relatively steep slopes both in the seaward face and leeward face to save material (it is common to have $1\frac{1}{2} \sim 3.0$ horizontal to 1 vertical for seaward side; $1 \sim 1\frac{1}{2}$ horizontal to 1 vertical for the leeward side). Among some of the older breakwaters one occasionally finds cases of mild slopes



(From Paape and Walther, 1963)
Akmon

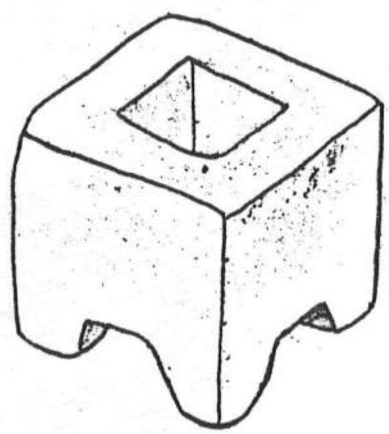


(From Paape and Walther, 1963)
Bipod



(Courtesy of Coode and Partners, Consulting Engineers, 2 Victoria St., London, S.W. 1)

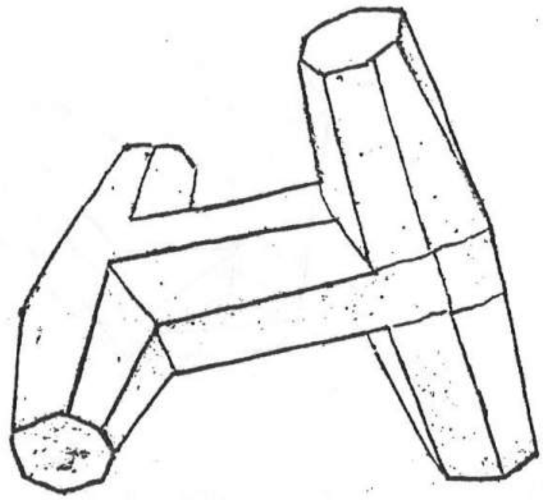
Cob



(After Jackson, 1968)

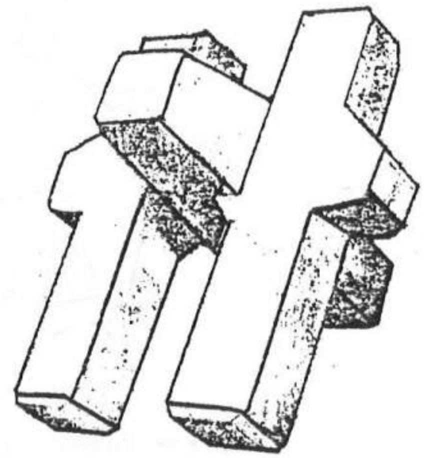
Cube (Modified)

Fig. 3.14 Various Shapes of Artificial Units



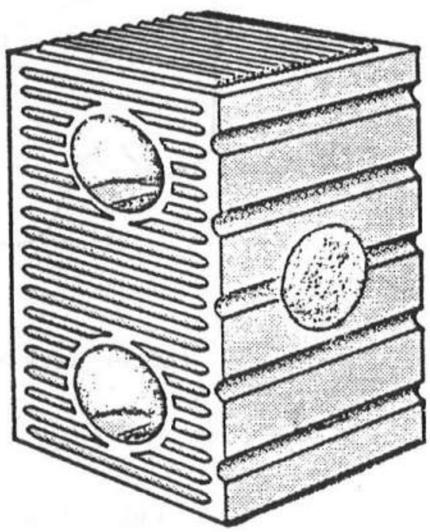
(Courtesy of E. M. Merrifield, Harbor Engineer, Port of East London, Republic of South Africa)

Dolos



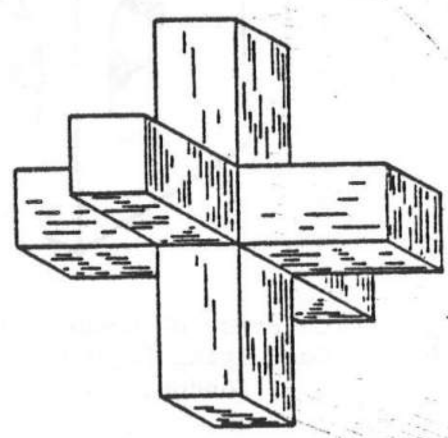
(Courtesy of S. Nagai, Osaka City University, Sugimoto-Cho, Sumihoshi-Ku, Osaka, Japan)

Gassho block



(Courtesy of P. Grobbelaar, 1971)

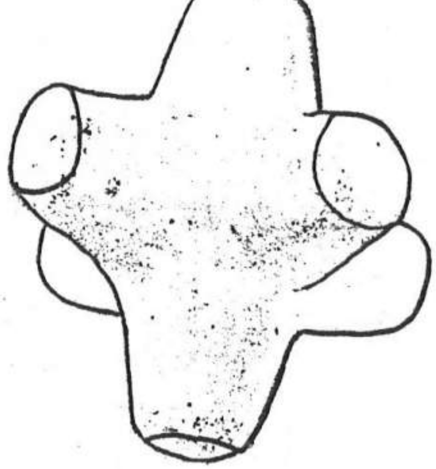
Grobbelaar block



(From Hexaleg Block Works)

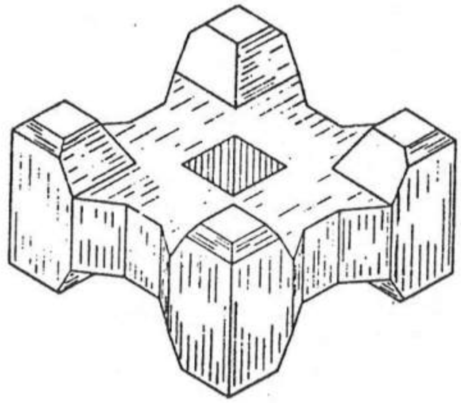
Hexalog block

Fig. 3.14 Various Shapes of Artificial Units (Cont'd)



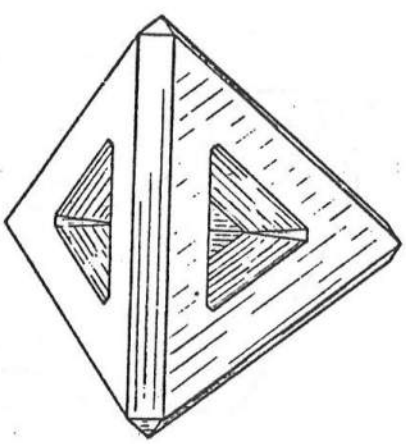
(After Jackson, 1968)

Hexapod



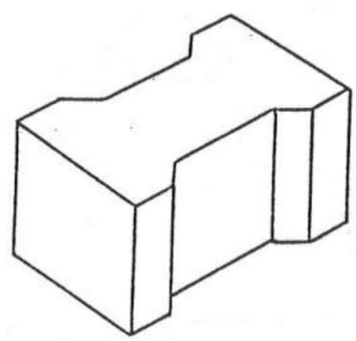
(After Nagai, 1962)

Hollow Square



(After Nagai, 1961)

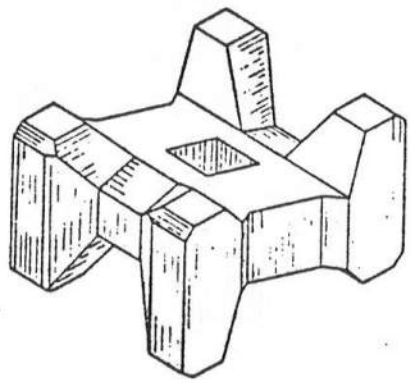
Hollow Tetrahedron



(Courtesy of U. S. Army Engineer District, Galveston, 1972)

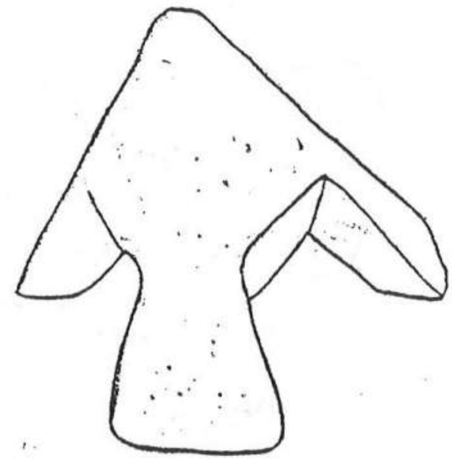
Interlocking H-block

Fig. 3.14 Various Shapes of Artificial Units (Cont'd)



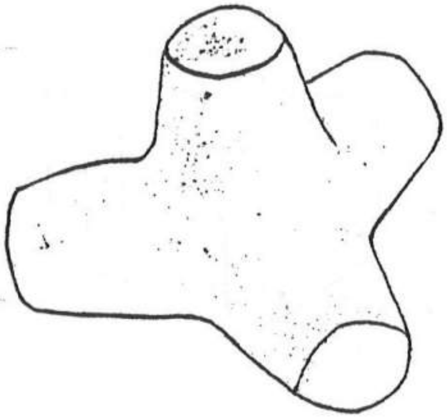
(After Nagai, 1962)

N-shaped block



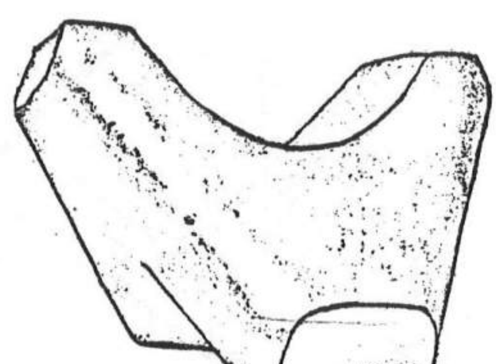
(After Jackson, 1961)

Pelican stool



(After Jackson, 1968)

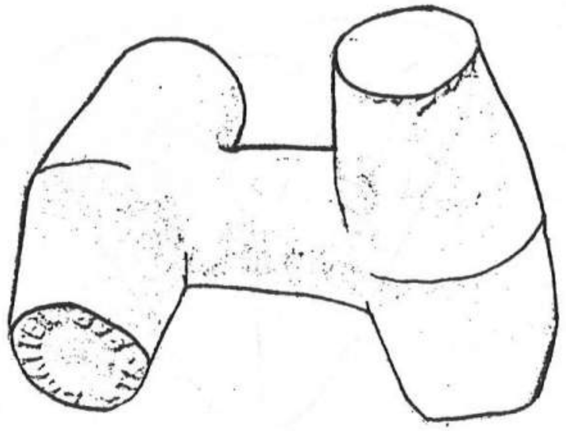
Quadripod



(Courtesy of Stabits Ltd., Sardinia House, 52 Lincon's Inn Fields, London, W.C. 2)

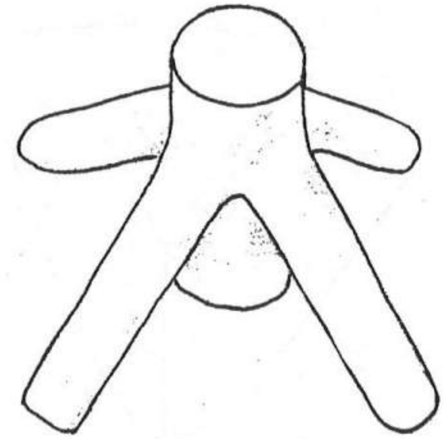
Stabit

Fig. 3.14 Various Shapes of Artificial Units (Cont'd)



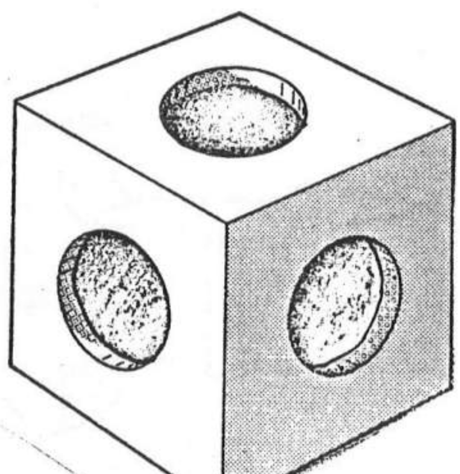
(Courtesy of R. J. O'Neill, Marine Modules, Inc., 475 Tuckahoe Road, Yonkers, N. Y. 10710)

Sta-Bar



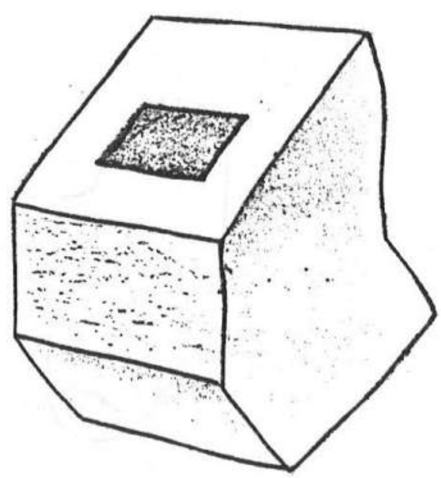
(Courtesy of R. J. O'Neill, Marine Modules, Inc., 475 Tuckahoe Road, Yonkers, N. Y. 10710)

Sta-Pod



(Courtesy of B. Hakkeling, Ing, Merellaan 269, Maassluis, Netherlands)

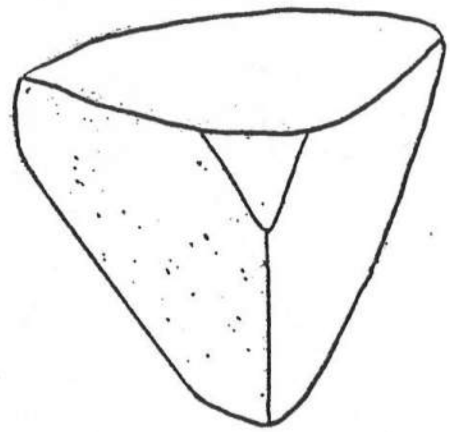
Stolk Cube



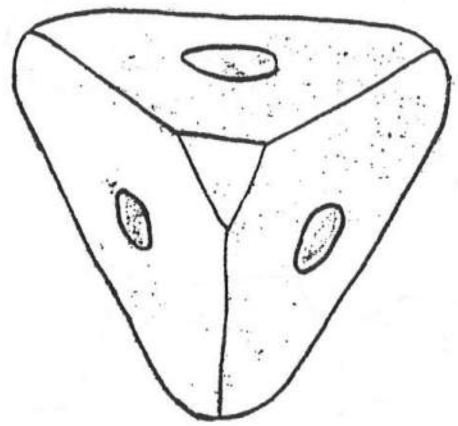
(Courtesy of Noreno, Cort Adlers Gate 16, Oslo, Norway)

Svee Block

Fig. 3.14 Various Shapes of Artificial Units (Cont'd)



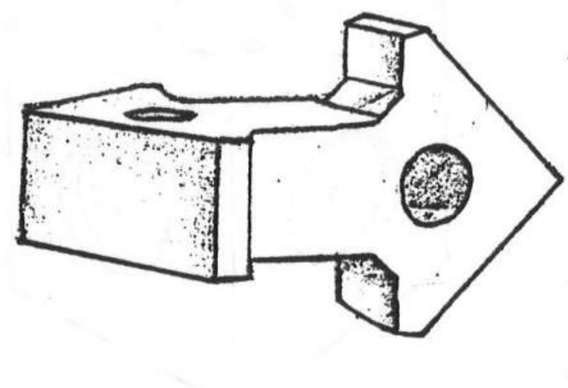
(After Jackson, 1968)
Tetrahedron (Solid)



(After Jackson, 1968)
Tetrahedron (Perforated)

(After Tetrapods Technical Note and Applications)

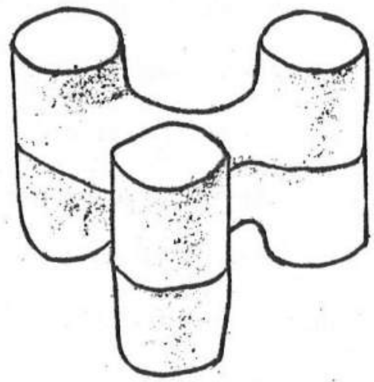
Tetrapod



(Courtesy of P. Grobbelaar, 1971)

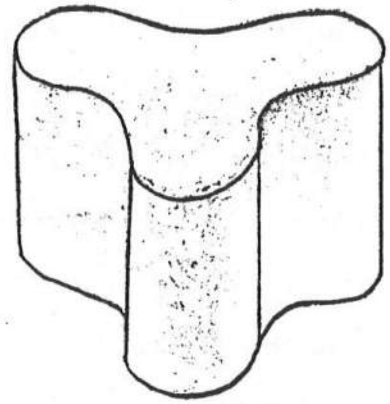
Toskane

Fig. 3.14 Various Shapes of Artificial Units (Cont'd)



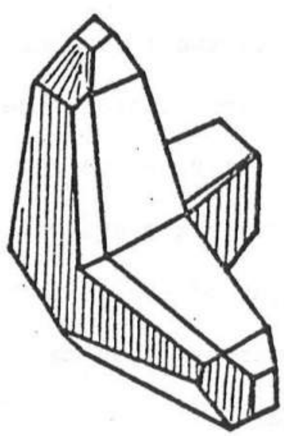
(After Jackson, 1968)

Tribar



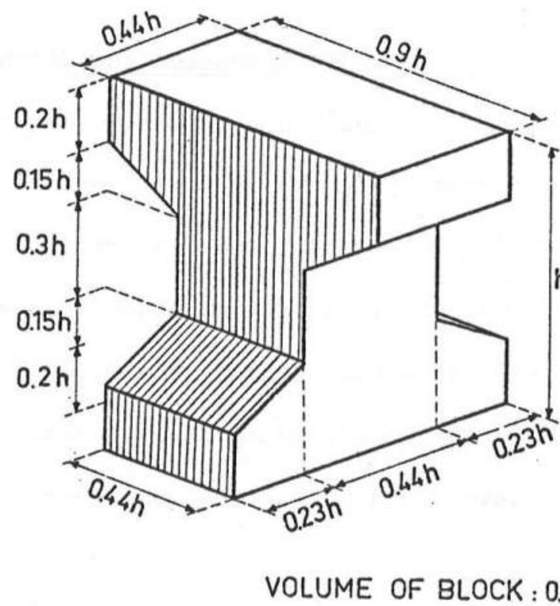
(After Davidson, 1971)

Tri-Long



(From Paape and Walther, 1963)

Tripod



(From Paape and Walther, 1963)

Details of Akmon Armor Unit

Fig. 3.14 Various Shapes of Artificial Units (Cont'd)

(For instance, Holyhead breakwater and Alderney breakwater have about 7H to 1V slope). As a rule of thumb, steep-sloped structures are more acceptable to wave overtopping but less to wave breaking whereas the reverse is true for mild-sloped ones. Because of the advent of precast armor units that have higher interlocking ability, the trend of current breakwater design seems to converge to single-steep-slope type structures. Most breakwaters when partially damaged by wave attack assumed a curved-shaped profile as shown. This shape seemed to maintain its stability for quite a long time, therefore, new research is being initiated in various parts of the world to investigate the feasibility of designing cover layers of non-uniform slopes. However, the difficulty of this new concept is that different wave conditions will result in different stable profiles.

Crest Elevation and Width

The crest elevation and crest width are two important structural factors influencing wave overtopping which in turn controls the stability of the leeward slope. Overtopping is usually tolerated to the extent that the area protected does not suffer excessive wave agitation and that the structure is not endangered due to bleaching of the leeward section. During prolonged storms, bleaching of the leeward section may lead to severe and accelerated damage of the breakwater.

The crest elevation is usually governed by the design storm tide and wave runup which are discussed in the following section on structural-environmental interaction. The question of the crest width of a rubble structure when overtopping is allowed is difficult to answer due to the lack of data and the large variations of design. So far there is no commonly accepted design standard. The U. S. Corps of Engineers recommended that, for overtopping and non-overtopping conditions, the minimum recommended width correspond to the combined widths of three capstones or to the width required for operation of construction and maintenance equipment, whichever is greater. In Europe, however, some designers felt that the crest width could be as narrow as two capstones. Because of the lack of criterion, caution should be exercised when the breakwater is expected to experience overtopping.

Structure Trunk and Structure Head

Structure trunk is the main body of the breakwater that has continuous lateral support of adjacent trunk sections. The wave attack in the trunk section is usually on the front surface and is, therefore, two dimensional. Much of the laboratory studies on breakwater stability are for the structure trunk section. This is not only because the structure trunk represents the major portion of the breakwater, but also because of the limitation of most of the test facilities.

The structure head is the termination section of the breakwater. There seems to be much more variation in the stability of a given head section than is apparent for a trunk section. For one thing, the head section is exposed on three sides instead of one to withstand powerful forces. Owing to the absence of uninterrupted lateral support, the structure head must be self-sustained and independent. That is, it must be treated as a perfectly detached and isolated structure capable of resisting all external influences, for in its downfall is involved the possible destruction of the breakwater section adjoining it.

In designing the structure head, the form, in particular its interferences with waves, the degree of exposure, the cross current, the nature of the sea-bed, etc. are all significant factors. Unlike the trunk section where the lee side of the structure is protected from direct wave attack, the leeward section of the structure head is the most vulnerable place because of constant impact due to falling water mass. In the present design practice, the breakwater head is always designed as a heavier structure. However, even with this practice, there are more instances of structure head damage than structure trunks.

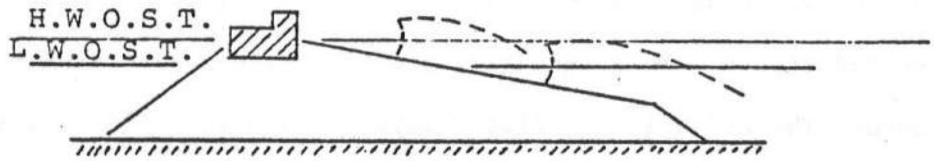
Filter Layer

A filter layer is used for preventing the material in the breakwater from washing out and aiding in toe protection of the breakwater. In most of the existing breakwaters, the filter system is composed of layers of graded stones. More recently, plastic filter systems have also been used. The criteria for filter layer design are: (1) the stones should be so graded that the core material will not be washed away; (2) adequate voids are provided to avoid high hydrostatic pressure being built up in the breakwater; (3) the filter layer should extend far enough beyond the scouring depth to minimize erosion potential.

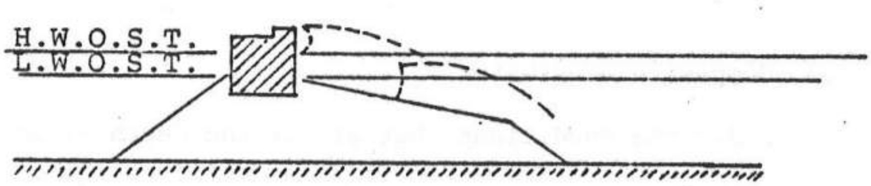
3.2.2.3 Composite Breakwater

Under the conditions that either the depth of water is great, or there is a wide tidal range, composite breakwaters of a combination of a rubble mound and a vertical wall are often selected. As mentioned earlier, breakwaters in Japan and many of the earlier breakwaters in Europe are of this type.

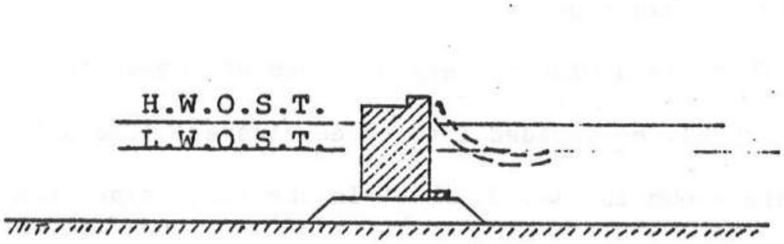
There is a wide variety of types of composite breakwaters. They can roughly be divided into three classes. The salient features of them are shown in Fig. 3. 15 . In the first type, the rubble mound is the real breakwater - the superstructure serves the purpose of preventing the overtopping. It also serves as a cap to hold the crest stones. The rubble slope extends to high water levels, thus forcing all waves to break before reaching the wall. The second type has a rubble mound at low water levels. The quantity of material required is less than that required in the preceeding case, but the superstructure is increased



TYPE A - COMPOSITE BREAKWATER WITH RUBBLE MOUND EXTENDING TO HIGH-WATER LEVEL



TYPE B - COMPOSITE BREAKWATER WITH CREST OF MOUND AT LOW-WATER LEVEL



TYPE C - COMPOSITE BREAKWATER WITH RUBBLE MOUND AS SUB-FOUNDATION TO VERTICAL WALL

Fig. 3.15 Various Features of Composite Type Breakwater in height. The stability conditions are also greatly altered. The broken-wave assaults are now closer to the wall at low tide and on the wall itself at high tide. Thus, the superstructure must be designed to sustain the impact of breaking waves. The third type has a rubble mound terminating at a level below the low-water mark and at a depth of water no less than one maximum wave height. Under the circumstances, most of the assaulting waves will retain their oscillatory characters and will be reflected as clapotis. In this type, the rubble mound is of small dimensions and really serves as a subfoundation to distribute the wall pressures over the sea bed.

In composite breakwater design the main considerations are the stability problem and the caisson structure. For the stability considerations one must assess:

1) The stability of the vertical wall including sliding, overturning and bearing stress of the mound foundation;

2) The stability of the rubble mound including sliding and armor unit stability;

3) The stability of the breakwater as a whole including circular sliding, settlement and meandering damage.

For the caisson structure, in addition to the common caisson design practice, one must also consider the problems of caisson stability at floating and the forces and stability during launching, towing and installation. References and detailed design procedures for this type of breakwater are offered in Refs. 4 and 5 .

3.2.3 Structure - Environmental Interactions

Prior to the 1930's the design of breakwater was based largely on experience. The designers relied heavily on the performance of previously constructed breakwaters. After that period, the design of breakwaters has been gradually on a more rational basis. Unfortunately, the problem of breakwater-environmental interactions is such a complicated one that many design factors still cannot be defined adequately. The present state of the art of how these design variables are handled is briefly reviewed here.

3.2.3.1 Loadings on Structures

In the current design practice, the external forces commonly considered are the wave forces, buoyancy and dead weight of the structure. Forces due to wind and current are generally neglected because of their relatively small magnitude. Furthermore, detailed force analysis is only performed in composite breakwater design and caisson design. For rubble mound breakwaters, external forces mentioned above are implicit design factors as will be seen later.

Wave forces exerted on structure can be distinguished as due to non-breaking waves, breaking waves and broken waves. Whether a structure is subject to either or a combination of those forces depends on the wave characteristics, the water depth at the toe of the structure and the foreshore slope and configuration.

To determine wave breaking characteristics one is usually required to carry the deepwater waves in the structure vicinity through the adjustment of refraction and shoaling. It is not an easy task to determine the characteristics of breakers nor the location of breaking. Rather, a zone on the structure is properly identified as susceptible to breaking wave attack.

The force due to non-breaking waves is basically hydrostatic. The Sainflou method or simplified Sainflou method (CERC, 1974, JPHA 1968,etc.) are commonly used to evaluate the non-breaking wave force and pressure distribution. There is very little doubt among designers in the validity of this method. A number of other variations have been proposed by various investigators (Ref. 233; Ref. 197). Their results, either the total force or the pressure distribution, differ very little from Sainflou's.

Figure 3.4 shows the wave pressure distribution according to Sainflow's method. ABED is the pressure diagram of the sur-pressures due to wave action, DEC is the still-water pressure diagram, p_1 is the value of the pressure due to wave action at the sea-bed, h_o is the use of the mean level of the clapotis formed due to reflecting wave. Sainflow's formula for peak pressure involves hyperbolic trigonometrical functions. For approximation, the pressure distribution can be treated as straight lines as shown. In this case, the only quantities which must be evaluated before the diagram can be drawn are the values of p_1 and h_o. These values can be obtained by:

$$p_1 = \frac{WH}{\cosh \frac{2\pi d}{L}}$$

and

$$h_o = \frac{\pi H^2}{L} \coth \frac{2\pi d}{L}$$

where H = wave height

L = wave length

d = water depth

W = specific weight of sea water.

For a unit length of wall, with h_o as the mean level of the clapotis above the stillwater level and P_1 the common length of the segments EB and EF, the resultant R_i and the moment about the base M_i are given respectively, for the maximum crest level (subscript e) and the minimum trough level (subscript i) of the clapotis by the formulas:

$$R_e = \frac{(d + H + h_o)\ (wd + P_1)}{2} - \frac{wd^2}{2}$$

$$M_e = \frac{(d + h_o + H)^2\ (wd + P_1)}{6} - \frac{wd^3}{6}$$

$$R_i = \frac{wd^2}{2} - \frac{(d + h_o - H)\ (wd - P_1)}{2}$$

$$M_i = \frac{wd^3}{6} - \frac{(d + h_o - H)^2\ (wd - P_1)}{6}$$

The calculation of breaking wave force is quite a different story. So far the mechanism of wave breaking is still not fully understood,let alone the prediction of breaking wave force on a barrier. A number of theories on breaking wave force has been advanced in the past. Among them, the Bagnold's experiments (Ref. 5 & 7) and Minikin's formula (Ref.233) was known to most of the designers. Bagnold's equation was based on his experimental work conducted in England. His equation involves a time expression which is difficult to determine for prototype structure. In order to overcome the difficulty of fixing this time scale, Minikin proposed a formula that did away with the time scale and the pressure distribution is expressed in terms of wave characteristics only. However, his work was based on extrapolating Bagnold's result to a limited full-scale experiment conducted by Rouviele and Petry (Ref.5) and Cagli (Ref. 197). The reliability of his formula is not all convincing. However, in spite of this, the Minikin formula has been widely used for lack of an improved method of analysis. In modern breakwater design practice, model tests are still of paramont importance to determine the loadings.

The Minikin equation assumes a peak pressure to be at still-water level and to diminish rapidly to zero at the crest of the wave at a height H/2 above peak pressure. Below the peak pressure there is also a rapid diminishing of pressure to you at H/2 below still-water level. Therefore, in accordance with Minikin's formula, the pressure distribution assumes a triangular shape (Fig. 3.5). The magnitude of the peak pressure, according to Minikin, should be

$$p_{max} = \frac{2\pi d}{LD} \ WH \ \{\frac{D + d}{2}\}$$

where H = deep water wave height

D = depth of water at toe of mound

d = depth of water at vertical face of wall.

In Japan, Hiroi's formula (Ref. 8) is often used. In his formula, it is assumed that the pressure intensity acts on the wall uniformly from 1.25 H above the still water level to the bottom of the structure with a magnitude equal to 1.5 times the head of the wave height

(Fig. 3.16). One of the shortcomings of his formula, of course, is that it could only be applied to shallow water cases. Also, it really only provides estimation of the total magnitudes of the breaking-wave pressure and does not yield information for local areas.

A formula which takes into account partial breaking waves has also been proposed by some authors (Ref. 5 and 293).

Goda's pressure distribution is trapezoidal as shown in Fig. 3.17.

The pressure intensities, p_1, p_2, and p_3 are calculated by:

$$p_1 = w_o H_{max}(\alpha_1 + \alpha_2 \cos^2\beta)$$

$$p_2 = \frac{p_1}{\cosh 2\pi h/L}$$

$$p_3 = \alpha_3 p_1$$

where:

$$\alpha_1 = 0.6 + \frac{1}{2}\left(\frac{4\pi h/L}{\sinh 4\pi h/L}\right)^2$$

$$\alpha_2 = \min\left\{\frac{h_b - d}{3h_b}\left(\frac{H_{max}}{d}\right)^2, \frac{2d}{H_{max}}\right\}$$

$$\alpha_3 = 1 - \frac{h'}{h}\left(1 - \frac{1}{\cosh 2\pi h/L}\right)$$

L: wavelength, β: angle of wave approach

In addition to the wave loadings, which is the prime consideration, the hydrostatic water pressure, buoyancy and lift, and dead weight of the breakwater are also considered as external loadings.

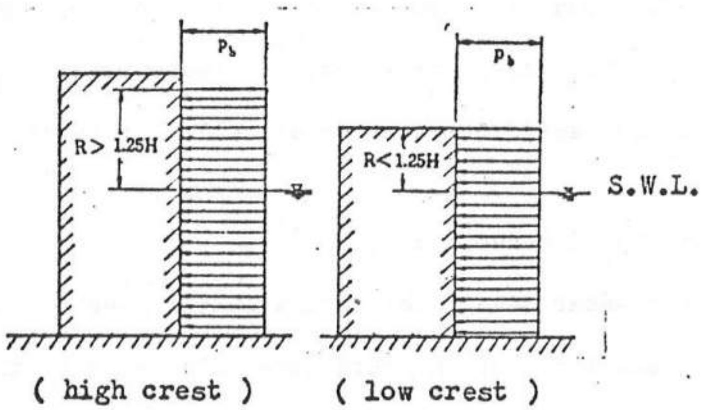


Fig. 3.16 Pressure Diagram Due to Breaking Waves According to Hiroi

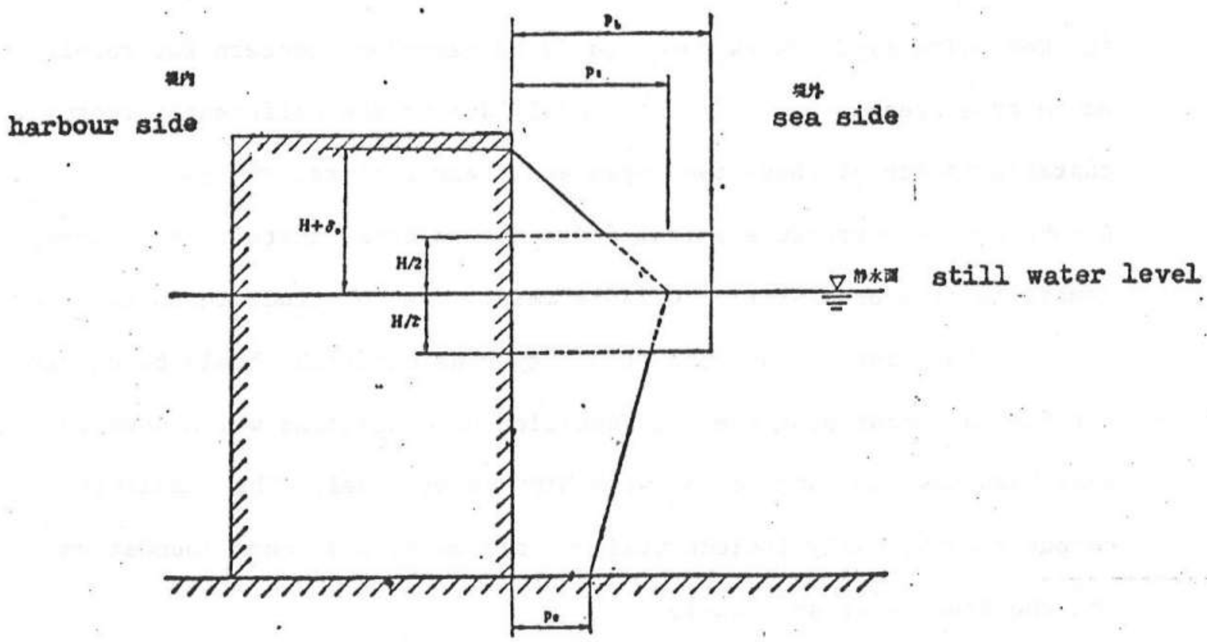


Fig. 3.17 Pressure Diagram Due to Partially Breaking Wave According to Goda

With this loading information in hand, the stress on breakwater can be analyzed. Usually, stress is only analyzed for vertical walls, caisson type of structures, and composite breakwaters. Rubble mound types of breakwaters are designed mainly on stability criteria.

3.2.3.2 Stability of Structure

In breakwater design the term stability bears two different meanings: the stability of the structure as a whole or the stability of armor units. The latter applies specifically to rubble mound type breakwaters. In this section, only the structure stability as a whole is discussed. The stability of armor units will be presented in the subsequent section.

The stability of the structure as a whole is of major concern for composite type breakwaters and is of secondary concern for rubble mound type breakwaters. This is mainly due to the different structural characteristics of these two types and their failure modes. A rubble mound structure seldom fails due to total instability whereas composite type breakwaters collapse mainly due to structure instability.

Needless to say, the stability consideration should be carried out for the worst possible sea condition or conditions which usually mean high wave in conjunction with high water level. The stability computations usually include wall or caisson, rubble mound foundation, and the breakwater as a whole.

For the stability of the wall or caisson, the problems of sliding, overturning and bearing stress on the foundation should be examined. Standard procedures for static analysis on earth structures are followed in these calculations. For random wave action, Ito (Ref. 353) what he called the probable sliding distance to evaluate the ultimate stability of caisson walls. This probable sliding distance is defined as the expected total displacement of a caisson due to irregular waves of certain magnitude and duration.

For the mound foundation, sliding along various possible sliding planes should be analyzed. Relations similar to the following are applied:

$$F = \frac{f\ (W\cos\theta - P\sin\theta)}{W\sin\theta + P\cos\theta}$$

where

- F = factor of safety
- W = total weight of the wall and the rubble mound above the supposed sliding plane subtracted by buoyancy and uplift
- θ = angle between the horizontal plane and the sliding plane
- f = coefficient of friction
- P = total horizontal wave force

The accepted safety factor should actually be more than 1.2. (Ref. 8) As for the structure as a whole, sinkage, differential settling and circular sliding are the common causes of instability and should be looked into.

3.2.3.3 Stability of Armor Units and Slope of Cover Layer

As mentioned earlier, for rubble mound breakwater design, the armor layer is the primary concern. The armor layer is generally designed in accordance with so-called stability criteria.

While it was appreciated that the slope and sizes of stones and of attacking waves were interrelated, it was not until 1933 that a first rational method has been proposed by deCastro. In 1938, Iribarren first published his work regarding armor unit design. His formula is still used today by European designers. The expression is (Ref.349)

$$W = \frac{KH^3\gamma_n}{(f\cos\alpha - \sin\alpha)^3(\gamma_n - \gamma_f)^3}$$

where

W = weight of the stone

H = wave height

f = coefficient of friction

α = angle of the slope

γ_n = specific weight of the stone

γ_f = specific weight of the water

K = stability coefficient combining all unevaluated variables.

This equation is used to determine the size of the stone for designated design wave height and structure slope. The coefficient of friction was taken equal to 1 in his original paper. However, the author later concluded through an extensive series of tests that this coefficient varies greatly with the number of blocks as counted along one line from bottom to top of the slope. For natural stones, the coefficient approaches 1.0 for large numbers of blocks, while with the number as low as six, the coefficient was found to be 2.38. The stability coefficient K is another experimentally determined value. Table 3.5 summarizes the values of K and f for different kinds of blocks for "no-damage" condition.

	K	f
Natural Stone	0.43	1.0 to 3.64
Concrete Parallel Pipe	0.43	2.84
Tetrapods	0.656	3.47

Table 3.5 Values of K and f for Iribarrin Formula for No Damage Condition (Ref. 349)

A major drawback of Iribarrin's formula is that the coefficient K is not dimensionless and this makes it difficult to extend experimental results to prototype application. His formula has been modified by many investigators, but the basis of the formula remains unchanged.

In Europe, Hedar of Sweeden (Ref. 431) and Svee from Norway (Ref. 452) have proposed variations of Iribarrin's formula. Perhaps the most widely acclaimed modification came from Hudson of the United States (Ref. 478) Hudson's formula differs from Irribarrin's in that the stability coefficient is dimensionless. This modification enables more rational interpretation of small-scale experimental results for prototype design usage. His formula takes the following form:

$$W = \frac{H^3 \gamma_n}{K_D (S_n - 1)^3 \cot\alpha}$$

in which

S_n is the specific gravity of armor unit

K_D is the dimensional stability coefficient that combines all unevaluated variables.

One of the reasons that Hudson's formula becomes so popular is that many experiments have been carried out in the United States, mainly by U. S. Army Corp of Engineers, to determine the values of K_D for various armor units, both natural and prefabricated. Thus, the Hudson formula has a wide base for practical application. Many experimental results of values K_D are summarized in a report edited by Hudson (Ref. 434) Some of the tabulated results for different kinds of armor units are reproduced here in Tables 3.6 to Tables 3. 8. Table 3. 9 summarizes the type of molded armor units that have been developed. Tables 3.6 and 3.7 provide recommended values of K_D for structure trunk and structure head respectively for no damage condition. In here no damage condition allows minor displacement of armor units to the extent that the stability of the armor-unit section is not affected. The recommended values of K_D for structure trunk when some damage to the structure can be allowed were given in Table 3.8.

In Russia, a formula similar to that of Iribarrin also exists. It takes the form:

$$W = \frac{\mu \gamma_2 H^2 L}{\sqrt{1 + \cot^3\alpha}\ (\gamma_n/\gamma_f - 1)^3}$$

where

μ is the friction coefficient

L is the wave length

Thus, the Russian formula differs from the rest in that the wave length is explicitly apparent in the equation.

Brandtzaeg (Ref.193) in a summary paper presented in XXIst International Navigation Congress provided an illustrative example comparing stability calculations by using different formulae. His example is reproduced in

Table 3.6

Recommended* Values of K_D for Design of Structure Trunk

Breaking and Nonbreaking Waves, No-Damage and No-Overtopping Criteria

Unit	n	Placing Technique	K_D Breaking Waves	K_D Nonbreaking Waves
Smooth quarrystone	2	Random	2.1	2.4
Rough quarrystone	2	Random	3.5	4.0
Tetrapod	2	Random	7.2	8.3
Quadripod	2	Random	7.2	8.3
Tribar	2	Random	9.0	10.4
Tribar	1	Uniform	12.0	15.0
Dolos	2	Random	22.0**	25.0**

* Breaking-wave data are tentative and subject to change after more comprehensive ES 815 tests are completed.

** Tentative and subject to change after comprehensive ES 815 tests are completed. A few preliminary ES 815 tests, conducted in 1971, indicated that K_D for dolosse on steep slopes may be limited by slope failure rather than damage to the armor-unit cover layer. Therefore, a sea-side slope steeper than $\cot \alpha = 2.0$ is not recommended at this time.

Table 3.7

Recommended* Values of K_D for Design of Structure Head

n = 2, Random Placing Technique, No-Damage and No-Overtopping Criteria

Unit**	$\cot \alpha$	K_D Breaking Waves	K_D Nonbreaking Waves
Smooth quarrystone	1.5-3.0	1.7	1.9
Rough quarrystone	1.5	2.9	3.2
Rough quarrystone	2.0	2.5	2.8
Rough quarrystone	3.0	2.0	2.3
Tetrapod and quadripod	1.5	5.9	6.6
Tetrapod and quadripod	2.0	5.5	6.1
Tetrapod and quadripod	3.0	4.0	4.4
Tribar	1.5	8.3	9.0
Tribar	2.0	7.8	8.5
Tribar	3.0	7.0	7.7
Dolos	2.0	15.0	16.5
Dolos	3.0	13.5	15.0

* Tentative and subject to change after comprehensive ES 815 tests are completed.

** No data presently available for other armor units.

Recommended Values of K_D for Design of Structure Trunk When Some Damage to Structure Can Be Allowed; n = 2, Random Placing Technique, Nonbreaking Waves*

Unit	D, Percent 0-5	5-10	10-20	20-30	30-40	40-50
Smooth quarrystone						
H/H_{DW}	1.00	1.08	1.19	1.29	1.41	1.54
K_D	2.4	3.0	4.0	5.1	6.7	8.7
Rough quarrystone						
H/H_{DW}	1.00	1.08	1.23	1.37	1.47	
K_D	4.0	4.9	7.3	10.0	12.4	
Quadripod and tetrapod						
H/H_{DW}	1.00	1.09	1.21	1.32	1.41	1.50
K_D	8.3	10.8	14.5	19.2	23.4	27.8
Tribar						
H/H_{DW}	1.00	1.11	1.30	1.50	1.59	
K_D	10.4	14.2	22.8	35.2	41.8	
Dolos	(No data presently available)					

* See paragraph 14.

TABLE 3.8

TABLE 3.9

Types of Concrete Armor Units

Name of Unit	Development of Unit: Country	Development of Unit: Year	U. S. Patent Number
Akmon	Netherlands	1962	None
Bipod	Netherlands	1962	None
Cob	England	1969	None
Cube*	--	--	None
Cube (modified)	USA	1959	None
Dolos	Rep. So. Africa	1963	None
Dom	Mexico	1970	(?)
Gassho block	Japan	1967	None
Grobbelaar block	Rep. So. Africa	1957	None
Hexaleg block	Japan	(?)	None
Hexapod	USA	1959	None
Hollow square	Japan	1960	3,176,468
Hollow tetrahedron	Japan	1959	None
Interlocking H-block	USA	1958	None
N-shaped block	Japan	1960	3,176,468
Pelican stool	USA	1960	None
Quadripod	USA	1959	None**
Rectangular block*	--		None
Stabit	England	1961	None
Stabilopod	Rumania	1965	None
Sta-Bar	USA	1966	3,636,713
Sta-Pod	USA	1966	3,399,535
Stolk cube	Netherlands	1965	3,548,600
Svee block	Norway	1961	3,210,944
Tetrahedron (solid)	USA	1942	None
Tetrahedron (perforated)	USA	1959	None
Tetrapod	France	1950	2,766,592
Toskane	Rep. So. Africa	1966	None
Tribar	USA	1958	2,909,037†
Trigon	USA	1962	(?)
Tri-long	USA	1968	None
Tripod	Netherlands	1962	None

* Cubes and rectangular blocks are known to have been used in masonry type breakwaters since early Roman times, and in rubble-mound breakwaters during the last two centuries. The cube was tested at WES as early as 1943.

**Patent for tetrapods applies also to quadripods.
†Royalty free to agencies of U. S. Government.
The underscored units have been tested, some extensively, at WES.

Table 3. 10 , in which it can be seen that all these formulae yield compatible results for steep-sloped breakwaters whereas the deviations become greater for milder slopes.

The similarity and simplicity of the above formulae may create the impression that the calculation of rubble mound breakwater stability is a straightforward and routine matter. Actually, it is quite to the contrary. The simple mathematical expression indicates that the analysis can only go so far. All the unanalyzed variables that influence the stability of armor units are lumped into one coefficient. This coefficient varies, as one would expect, over a wide range of various situations. Most breakwater designers would advise the use of these formulae as guidelines, and the performance of actual design through the assistance of laboratory tests. Therefore, it is in order here to discuss briefly the limitations of the aforementioned formulae and the problems which should be watched for in actual design.

All the formulae presented above apply to relatively steep-sloped structures (roughly 1V to 3H as the limitation). For structures with shallower slopes, the mechanism of armor unit movement may be different from what the formulae are based upon. Hedar (Ref. 431) for instance, suggested that with slopes flatter than a certain limit, failure occurs through blocks being moved up the slope instead of down. Strictly speaking, these formulae are for non-overtopped breakwaters. Furthermore, these equations ignore the angle of approach which could be significant in exerting sideway forces to the armor units. All these formulae are intended for zones adjacent to the water level. For underwater armor units, the mechanism could be different. Kaplan(Ref.222) has proposed a formula for underwater

Table 3.10

Necessary Weight of Armor Blocks, with Wave Height H = 6.0 meters, According to Five Formulae

a = angle of slope
γr = specific weight of rocks = 2.60
γf = specific weight of fluid = 1.00
L = wave length in metres

Formulae	Coefficients	Q in tons, for cotg α =			
		1.25	1.50	2.00	2.50
Spain: (Iribarren) $Q = \frac{\gamma r\, N\, H^3}{(f\cos\alpha - \sin\alpha)^3 (\gamma r/\gamma f - 1)^3}$	N = 0.430 f = 2.38	31.2	20.3	12.4	9.5
Norway: (Svee) $Q = \frac{K\, \gamma r\, H^3}{\cos^3 \alpha (\gamma r/\gamma f - 1)^3}$	K = 0.12	34.5	28.4	22.9	20.5
Sweden: (Hedar) $Q = \frac{K_1\, Kdown^3\, \gamma r\, H^3}{(tg\phi \cos\alpha - \sin\ )^3 (\gamma r/\gamma f - 1)^3}$	Kdown =	3.2	4.2	5.7	6.4
	$K_1 = 0{,}1113.10^{-3}$ $tg\phi = 1.11$	35.6	22.5	17.3	14.0
U.S.A.: (Hudson) $Q = \frac{\gamma r\, H^3}{K\Delta\, cotg\alpha (\gamma r/\gamma f - 1)^3}$	$K\Delta = 3.2$	34.2	28.5	21.4	17.1
U.S.S.R.: $Q = \frac{\mu\, \gamma r\, H^2\, L}{\sqrt{1 + cotg^3 \alpha}\, (\gamma r/\gamma f - 1)^3}$	$\mu = 0.025$ L = H/0.05	39.8	32.7	22.8	16.8

armor unit by taking into account current effects. For lack of experimental verification, his formula is not widely used. Iribarrin also has modified the formula for underwater application by simply adjusting the input wave height.

The various values of the stability coefficient K_D account for all variables other than structure slope, wave height, and unit weight of armor units and the fluid in which they are placed. These additional variables that could influence the armor unit stability are:

1. wave length
2. depth of water
3. duration of storm
4. position of armor unit with respect to SWL
5. degree of overtopping
6. shape of armor unit
7. manner of placing the armor unit
8. portion of breakwater
9. number of layers
10. damaging history
11. coefficient of friction among units
12. porosity and voids
13. randomness of incident waves
14. effects of current.

Most of the available information on K_D was obtained from laboratory results. They are subjected to the following limitations:

a. They are tested under uniform wave height and wave period with wave impinging at right angle.

b. Placement of units is unlikely to duplicate prototype.

c. Scale effects are not completely determined.

d. Structural (material) behavior of the units is not modeled.

It is also clear that the variables involved are too numerous to permit exhaustive investigation. Such an investigation is, perhaps, not warranted anyway.

At present, the variables usually addressed upon are: a) the shape of armor unit; b) manner of placing (random or uniform); c) portion of breakwater (trunk or head); d) wave characteristics (breaking or non-breaking). As to the rest of the listed variables, the present knowledge varies from scattered information to non-existent. It certainly doesn't mean that these effects can be ignored but that for each design case, these effects have to be examined individually.

The duration of the storm, for instance, has long been recognized as a factor not to be ignored. However, most experiments were conducted with a certain number of waves of a definite size and shape without actually simulating the storm duration. Many investigators have expressed opinions that the damage of breakwaters is an accelerated process. That is, the damage will progress at an ever increasing pace once an initial damage occurred. Font (Ref.207) reported his experimental findings and concluded that for the initial movement of rocks or tetrapods it seems that the duration is not important. The **duration** becomes relevant for advanced damage. An example of his results is shown in Fig. 3. 18 . In there, $H_{1\%}$ is defined as the wave height that causes 1% damage.

Another factor that has drawn increasing attention is the effect of irregular waves that one actually experiences as opposed to

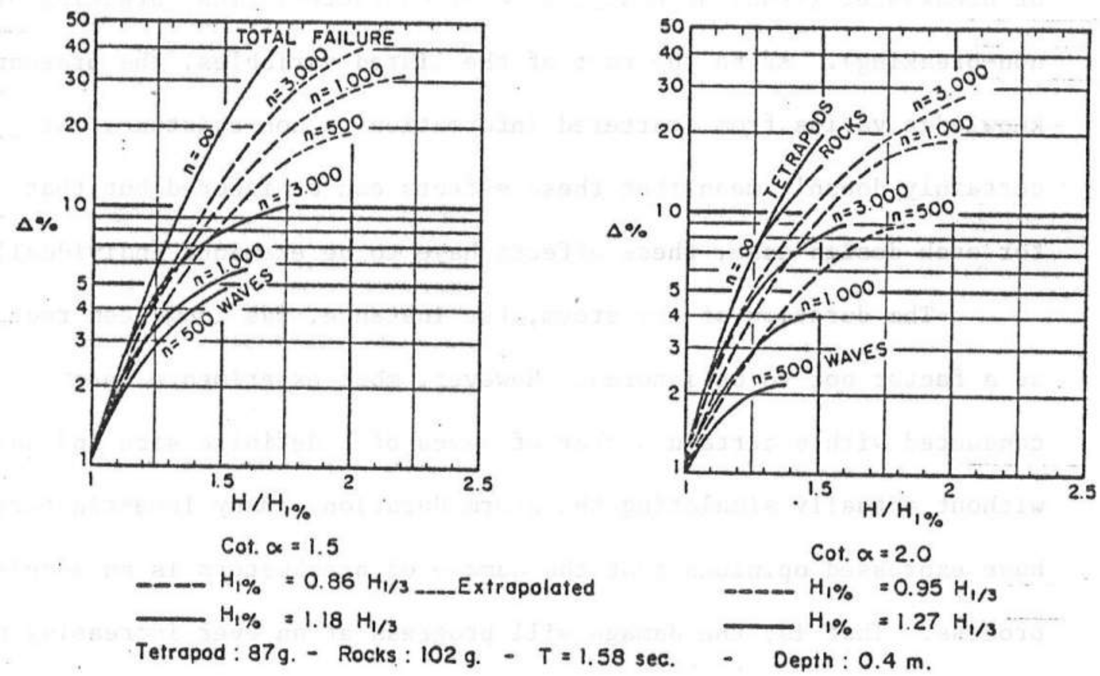


Fig. 3.18 Effects of Storm Duration of Armor Unit Stability (Format)

the regular wave tests conducted in the laboratory. Two recent related investigations on Europort Breakwaters, one by the River and Harbor Research Laboratory in Norway (Ref. 26) and the other by Delft Hydraulics Laboratory in the Netherlands (Ref. 794) , seemed to indicate that the fact that the wave in nature is irregular and this factor cannot be ignored. The results showed higher damage due to irregular waves than regular waves (with significant wave height of irregular waves equal to the height of regular waves). Samples of their results are shown in Figs. 3.19 and 3.20 . The result shown in Fig. 3.20 is also tied in with the effect of foreshore geometry to breakwater stability.

The effect of wave length on breakwater stability is still unresolved. According to some investigators, including Hudson, the wave length has a minimum effect except when it becomes a crucial factor in determining wave breaking. Therefore, the values of K_D presented by Hudson are different for breaking and non-breaking waves. There are other investigators who argued that the water motion is closely related to wave length (or wave period); therefore, the force exerted in the armor unit must be different for different wave lengths. Russia's formula is one of the examples. Research results in England (Ref. 477) also concluded that the effect of wave length (period) may be important in the advanced stage of damage.

It is generally agreed upon that an increase in porosity will enhance the stability of armor units; the exact extent is not known. It probably is safe to say that the effect of porosity is insignificant

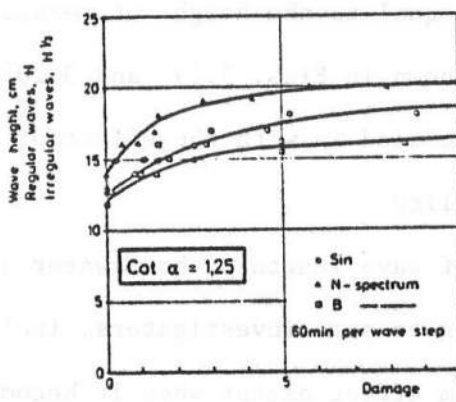
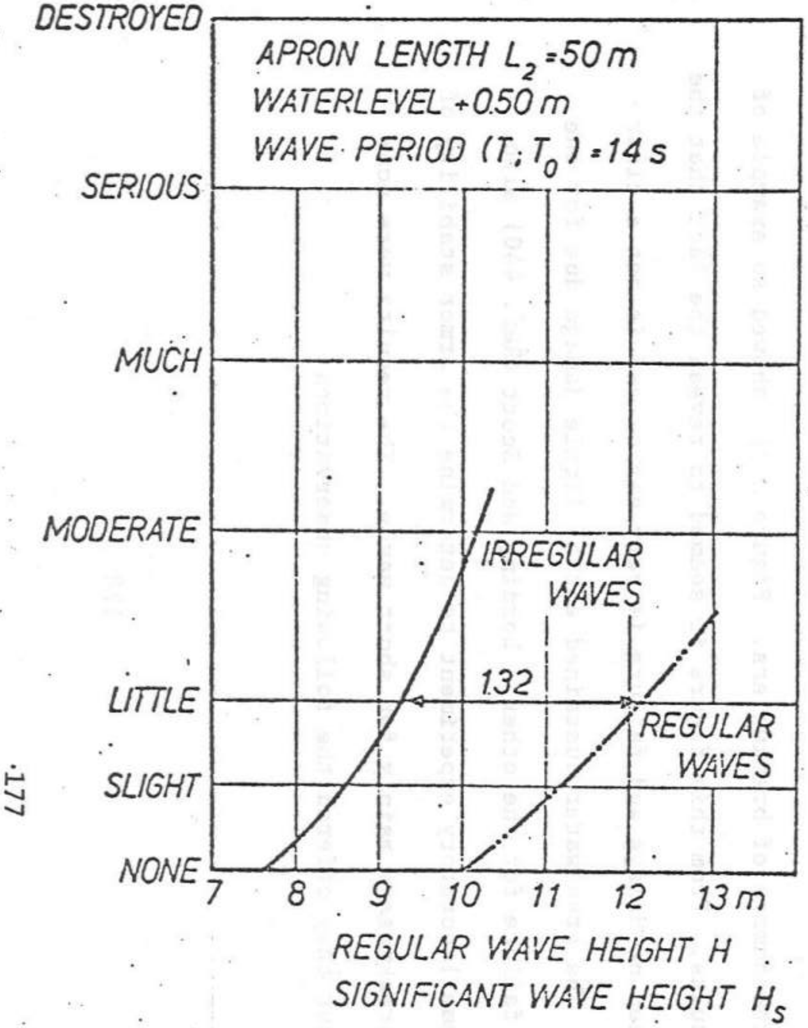
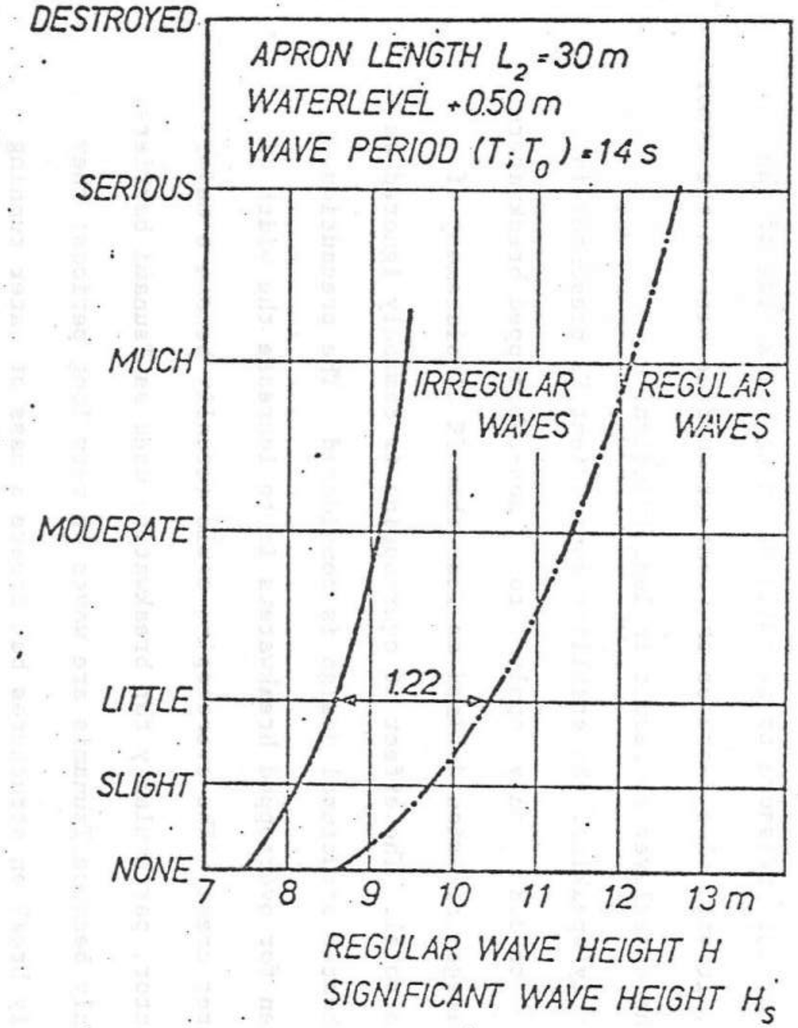


Fig. 3.19 Damage Comparisons Due to Regular Waves and Irregular Waves



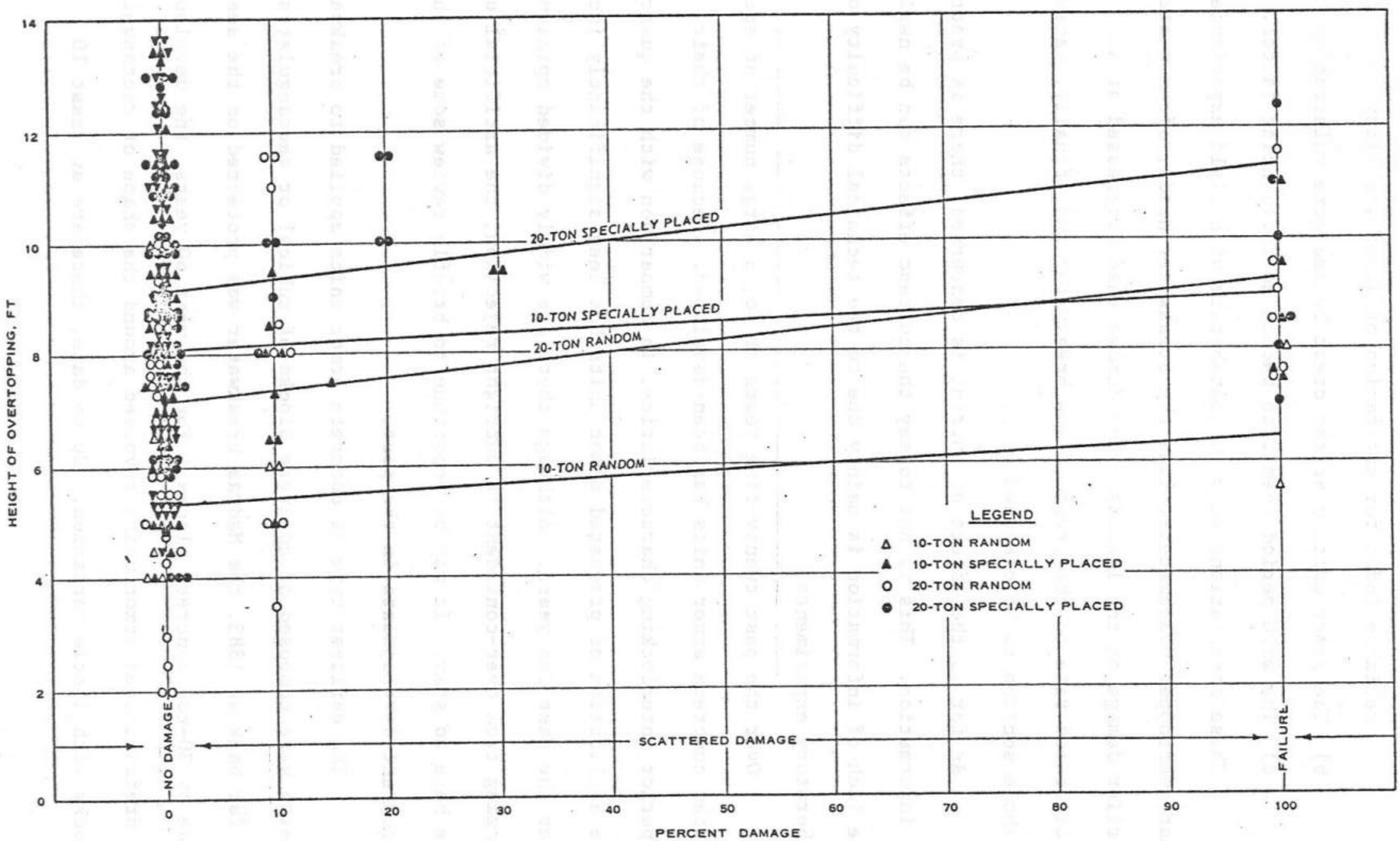
COMPARISON DAMAGE FUNCTIONS FOR REGULAR AND IRREGULAR WAVES (PROFILE II)



COMPARISON DAMAGE FUNCTIONS FOR REGULA AND IRREGULAR WAVES (PROFILE II)

Fig. 3.20 Damage Comparisons Due to Regular Waves and Irregular Waves (Ref. 794) and that although an increase in porosity will decrease the potential of high hydrostatic pressure to be built up on the back side of the armor layer, it will also decrease the contact surface between the armor layer and the underlayer to result in less friction.

Strictly speaking, the stability coefficient K_D presented in Tables 3.6 to 3.8 only apply to non-overtopped breakwaters with a no-damage criterion defined as less than 5% displacement of units of the total. The effect of overtopping is commonly ignored as far as breakwater structural design is considered. The precaution usually taken for overtopped breakwaters is to increase the width of the breakwater crest. The overtopping could actually become a major damaging factor, particularly for breakwaters used as Tsunami barriers. This is mainly because Tsunamis are waves of very long periods; they don't usually break on structures but create a mass of water running up and finally overtopping the barriers. This mass of water, when it falls, may slam hard on the leeside and cause excessive damage. Tests performed for the rehabilitation of the Tsunami barrier at Hilo Harbor, Hawaii (Corps of Engineers, Rep. 1968), indicated the effects of overtopping on the damage of breakwaters. Figure 3.21 showed an example of the test results. From this figure it seemed to reveal the fact that the margin between no damage and failure is very narrow and is not a clear cut matter. The breakwater sustained either little damage due for one instance or failure for the other. Lording and Scott (Ref. 440) also performed some laboratory experiment to determine the armor stability of overtopped breakwaters mainly for short waves. The results were non-conclusive but they offered the following observations:



HEIGHT OF OVERTOPPING VERSUS PERCENT DAMAGE

TEST SERIES 2, TEST SECTION 10

Fig. 3.21 Armor Layer Damage Due to Overtopping

a) For overtopped breakwaters, the lee slope may be a more reliable index for estimation of armor stability.

b) The armor units near the crest became more vulnerable.

c) The wave period seemed to become an influencing factor.

These observations seem to collaborate with field experience that overtopped breakwaters, like the breakwater heads, often sustained earlier damage on the leeside. The damage then progressed at an accelerated rate to the crest of the breakwater and, finally, caused a whole section to be bleached.

As far as the effect of current is concerned, there is practically no information. This is not to say the current effects can be neglected. The lack of information is mainly due to the technical difficulty of laboratory experiments.

Over the past twenty-five years or so, a large number of special-shaped concrete armor units has been developed. Because of their superior interlocking characteristics, in comparison with the quarrystone, the application of precasted armor units has been significantly increased over the past ten years. Although there are widely divided opinions ranging from over-confident to outright rejection, the artificial units are here to stay. It may be opportune to briefly review some of the important developments in the past.

The earliest type of concrete armor units applied to breakwater design were composed of concrete blocks of cubical or rectangular shapes. As far back as 1883, the Madras breakwater was protected on the seaward side by 30-ton concrete blocks. For the next 60 years, the development of prefabricated armor units revolved around the shape of rectangular blocks with little variation. Up to date, there are at least 10 variations of this kind including the more popular ones such as modified cubes, stalk cubes and tetrahedon. In 1950, tetrapod was developed in France, which is quite different in shape to the conventional blocks. It almost dominated the scene of breakwater repairs and reinforcements for the following decade. More than 20 breakwaters used tetrapod for repairing, extension or construction. In the late 1950's it became evident that tetrapod out-performed natural quarrystones in maintaining stability. From this point on, a race was on throughout the world to develop artificial armor units of more radical shapes. It was realized that tetrapod, though superior than quarrystone, has a fundamental disadvantage of high center of gravity. Therefore, it may be easily rolled over on a steep-sloped wall. In the United States, tribar was developed by Palmer (Ref. 792) and was successfully applied to a number of breakwater repairs. Other developments include sta-bar and sta-pod by a private concern, quadripod, a modified version of tetrapod, trilong, etc. In Japan, a number of modifications of existing forms were made such as hollow squares and hollow tetrahedron. Professor Nagai (1962) developed an N-shaped hollow block with four legs and reported a high stability coefficient through laboratory tests. In Europe, tripod, akmon and stolk cubes were the contributions from the Netherlands. Staleit was developed in England and was used in England and New Zealand. Values of stability coefficient ranging from 19 to 25 were reported. The svee block was the production of Norway. Its stability depends heavily on the orderly placing. Adding to this list, dolos, grobbelaar block and toshane were developed in the Republic of South Africa. Among them, dolos (Ref. 775) has received the most attention.

The authors have reported damage coefficients as high as 40 for double-layer random placement and 25 for single layer uniform placement; both for 0% damage case (the 0% damage actually allows for 2% damage). Hudson (Ref. 434) instead tentatively recommended $K_D = 22$ for structural trunk design for breaking waves and 15 for structural head for two layers. According to the results reported by Merrifield and Zwamborn, the margin of the stability coefficient is even larger compared with other shapes when percent damage allowed is increased (Fig. 3.22). Such a characteristic is, of course, attractive from the point of view of safety margin. A list of the types of concrete armor units is provided in Table 3. 9 . Among these units, many are in the experimenting state with no sufficient technical data to substantiate them. Some require special placing arrangements; thus, are limited in their application. Tetrapod, tribar, dolos and tetrahedron are among the popular ones. In addition to the shape characteristic which is the dominant factor in stability, there are a number of other facets concerning the application of precasted concrete armor units that should not be ignored. The specific gravity of concrete used for casting, for instance, is a factor that could influence the stability. According to Hudson's formula, higher specific quantity should be advantageous. Brandtzaeg (Ref.344) proposed a modified formula to include the effects of specific weight when S_n is unusually large and small. His formula takes the form:

$$W = \frac{\gamma_r H^3}{K_D (S_n - \phi)^3 \cot\alpha}$$

The additional variable ϕ in the denominator, according to

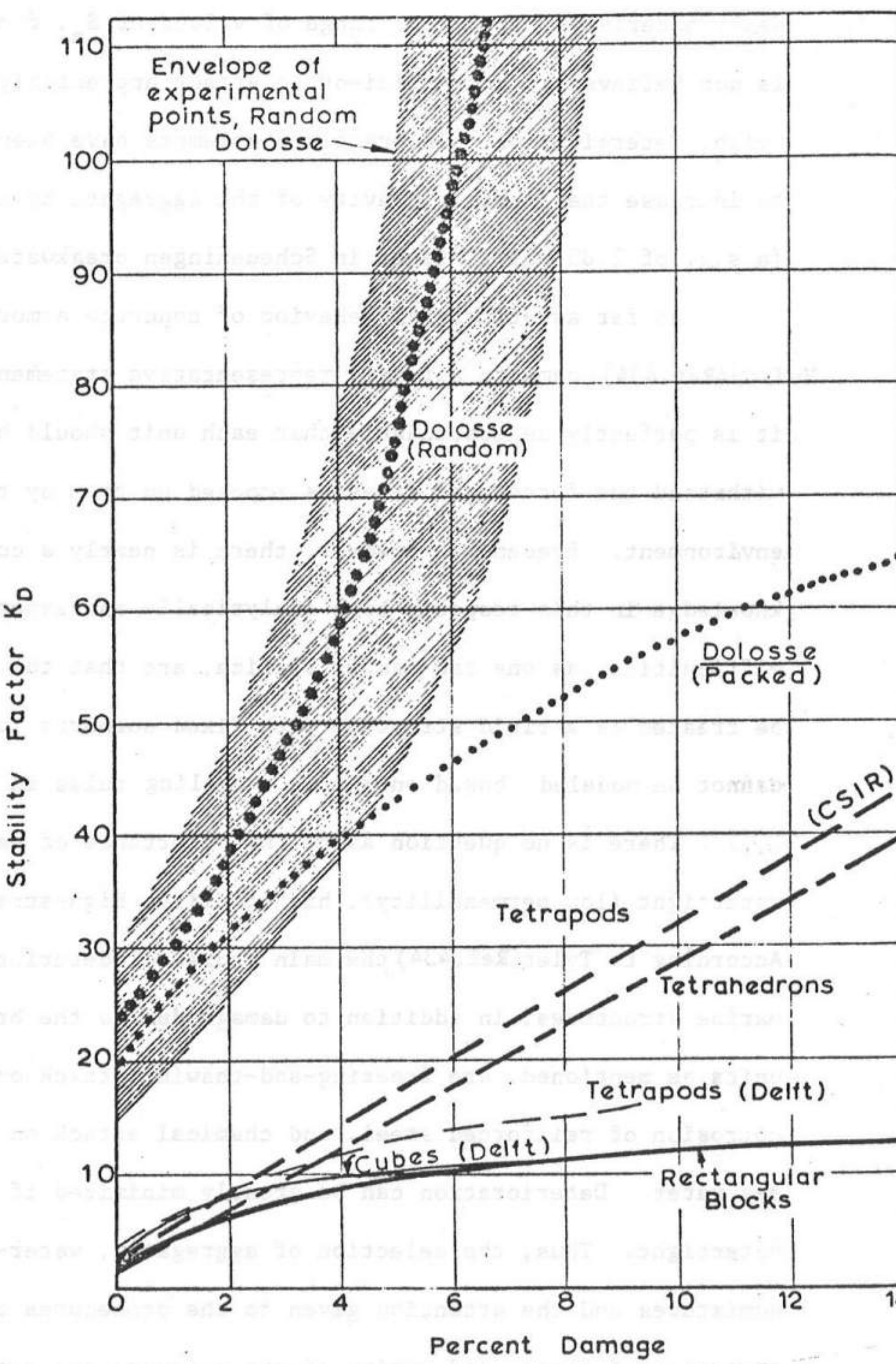


Fig. 3.22 Stability Coefficient vs Percentage Damage

Brandtzaeg could vary from 0.37 to 1.05 for unusually light to unusually heavy materials. The usual range of values of S_n, from 1.88 to 2.76, is not believed to be sufficient to affect appreciably the accuracy of weight determination. In practice, attempts have been made in the past to increase the specific gravity of the aggregate by adding lead slags (a s.g. of 2.85 was achieved in Scheueningen breakwater (Ref. 65)

As far as structural behavior of concrete armor units is concerned, Hudson(Ref.434) came to a fairly representative statement. First of all, it is perfectly understandable that each unit should be designed to withstand the forces and stresses imposed on them by the hostile environment. Presently, however, there is nearly a complete lack of knowledge in this respect, both analytically and experimentally. The difficulties, as one can easily realize, are that the armor unit cannot be treated as a rigid structure with fixed supports in analysis, and they cannot be modeled based on simple modeling rules in experiments.

There is no question as to the importance of using a dense, watertight (low permeability), high-quality, high-strength concrete. According to Tyler(Ref.434), the main causes of deterioration in concrete marine structures, in addition to damage due to the breakage of armor units as mentioned, are freezing-and-thawing attack on the concrete, corrosion of reinforced steel, and chemical attack on the concrete by sea water. Deterioration can be greatly minimized if the concrete is watertight. Thus, the selection of aggregates, water-cement ratio, and admixtures and the attention given to the procedures of mixing, placing, stripping of forms, and curing of the concrete are extremely important. If possible, as suggested by the Report of AD Hoc Committee of Artificial

Armor Units for Coastal Structures, the strength of the concrete should not be less than about 5000 psi at 28 days.

The question whether the concrete should be reinforced remains open. Many contend that reinforcement is unnecessary, but perhaps not harmful. The argument being that the breakage of armor units is mainly due to impact loading and reinforcement does not help in this category. Once they sustained breakage, the reinforced bars may be exposed to seawater causing accelerated corrosion. Systematic drop tests of tetrapods by Danel,et.al.(Ref.469) seemed to find no difference on the fracture characteristic whether the units were reinforced or not. Others firmly believe that armor units should be reinforced. They based their claims on some field observations that the unreinforced units outnumbered reinforced ones in breakage. However, the information is quite limited to be conclusive.

Since the shape factor is expected to play an important role in affecting the fracture behavior, it is important that tests be performed for each specific case to determine whether reinforcement should be used.

After the above discussion of armor units, it may be appropriate to conclude this section with a list summarizing the important engineering points to be exercised in the armor unit selections:

a) The ability of armor units to withstand downrush, uprush and outrush resulting from wave actions. The downrush is considered to be the major damage mechanism under normal conditions on the seaward slope of the breakwater. The uprush is a factor for shallow-sloped structure or when part of the breakwater is flattened by settling or sustained initial damage. The outrush is particularly critical for lee side slope and structure head.

b) The energy absorbing characteristics of the armor layer and the related runup, overtopping and wave reflection characteristics. As explained earlier, the overtopping is one of the important mechanisms to dislodge leeside units. Reflected waves are a major contribution for toe scouring which in turn affects the overall stability of the armor layers.

c) The ability of armor units to withstand impact loading due to breaking waves.

d) The stability of the armor layer after the attack of sustained storm. This includes the aspect of catastrophic structural failure and the aspect of post storm repairability. A number of factors should be considered:

1) The interlocking ability of the armor units when scouring has occurred or core material was washed out from under. Under this condition the type of armor unit selected should be able to sustain its interlocking capability without total collapsing so that repair can be made after the storm is over.

2) The stability of the armor units at advanced stage of damage. Usually the section of armor units is based on their stability at a designated percentage of damage whether it be 0.1 or 2 percent. It is also inevitable that a breakwater will sustain some damage duration storms. The behavior of armor units after certain

damage has occurred, therefore, provides the safety margin of the structure. The values of stability coefficient K_D at various percentages of damage, for instance, serves as a partial indicator. Figure 3. 23 is an example of such a plot. If a structure were designed for 0% damage, it would probably have the same design safety margin whether dolosse , tetrapods or quarry stones were selected as the armor units. However, according to Fig. 3. 22 , should certain damage occur, dolosse would provide better protection for the structure from further deteriorating than the other two.

3) The stability of armor units at waves exceeds the design wave height. This factor is another indication of safety margin in armor unit section. Figure 3.24 illustrates this property for a number of different armor units.

4) The ability of armor units to resist further deteriorating when a section breaking water crest has been bleached. Usually when a section of the crest has been opened up either because of uprush or overtopping, the water began to pour in like a jet. This jet type action could be very damaging to threaten total structure failure unless the armor units can readjust themselves to stabilize deterioration. There is practically no information in this regard.

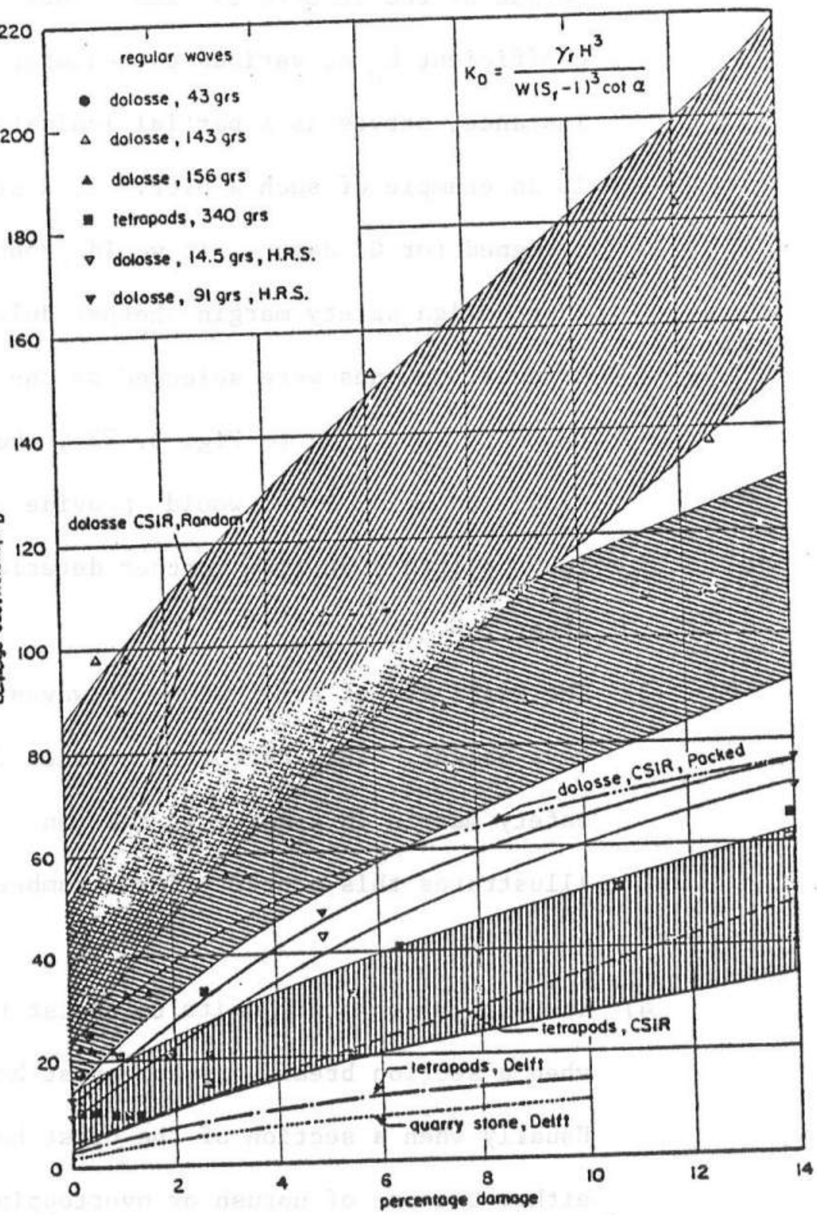


Fig. 3.23 Stability Coefficient vs Percentage Damage

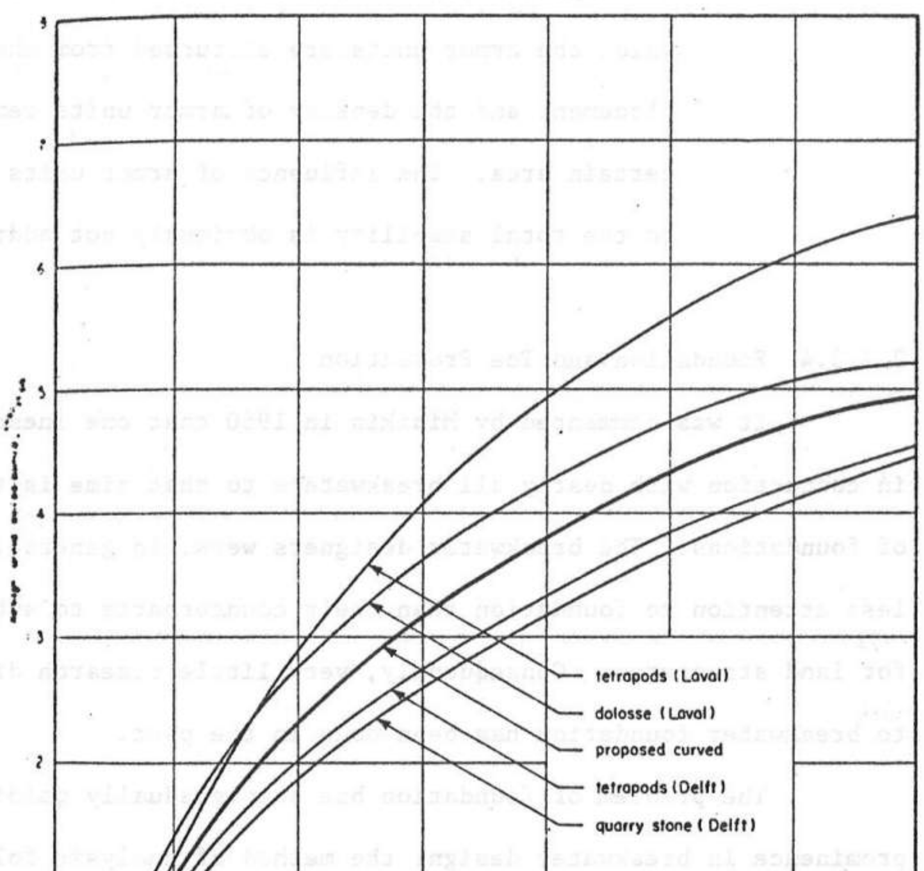


Fig. 3.24 Design Wave Exceedance $H_s/H_{\underline{so}}$

e) The stability of the armor layer when partial breakage of armor units occurs. In the usual manner of armor unit testing, the damage is measured in terms of dislocation or displacement. It serves to indicate the degree to which the armor units are disturbed from their initial placement and the density of armor units remained in a certain area. The influence of armor units integrating to the total stability is obviously not addressed.

3.2.3.4 Foundation and Toe Protection

It was commented by Minikin in 1950 that one inescapable fact in connection with nearly all breakwaters to that time is the weakness of foundations. The breakwater designers were, in general, paying for less attention to foundation than their counterparts to sub-foundation for land structures. Consequently, very little research directly related to breakwater foundation has been done in the past.

The problem of foundation has been gradually gaining prominence in breakwater design; the method of analysis follows quite closely to the earth structure design on land. The Design Manual 7 - Soil Mechanics, Foundations, and Earth Structures - published by the U. S. Naval Facility Engineering Command (1962) and Design Standard for Port and Harbor Structures published by the Japan Port and Harbor Association (1968) are two standard manuals frequently consulted.

The foundation of breakwater is usually treated as a shallow foundation subject to vertical loadings due to the weight of the breakwater and horizontal loadings due to waves. Strictly speaking, the vertical load diagram should have trapezoidal shape. For simplicity, Minikin suggested that a load diagram of triangular shape could be used instead. To compensate for this replacement, the apex height (p) can be considered 25 percent greater than the flat top of the mound (see Fig. 3.25). The horizontal loading results from the dynamic wave pressure and the differential water level on both sides of the structure are due to storm surge, wave setup, etc. In shallow water, the load diagram can be approximated by a triangular shape.

After the load condition has been determined, foundations are analyzed for rotational failure, translational failure, and total and differential settlement. An example of failure lines is shown in Fig. 3. 26 . So far, foundation design practice is carried out for static loading condition. Foundation response to dynamic loading and potential of soil liquifaction , though noted by many designers, were rarely performed. DM-7 recommends a safety factor no less than 1.5 for permanent or sustained loading conditions and no less than 2.0 to limit movements necessary for strength mobilization or local plastic strain at the foundation edge. Detailed requirements for safety factors in bearing capacity analysis can be found in DM-7.

For rubble mound breakwater, the core material is usually compacted sand and gravel. The failure analysis should be extended to include this section. Examples of failure calculations are presented in Figs. 3.27 and 3.28.

Since the location of breakwaters is primarily dictated by the requirements of wave protection the in situ soil condition may not be

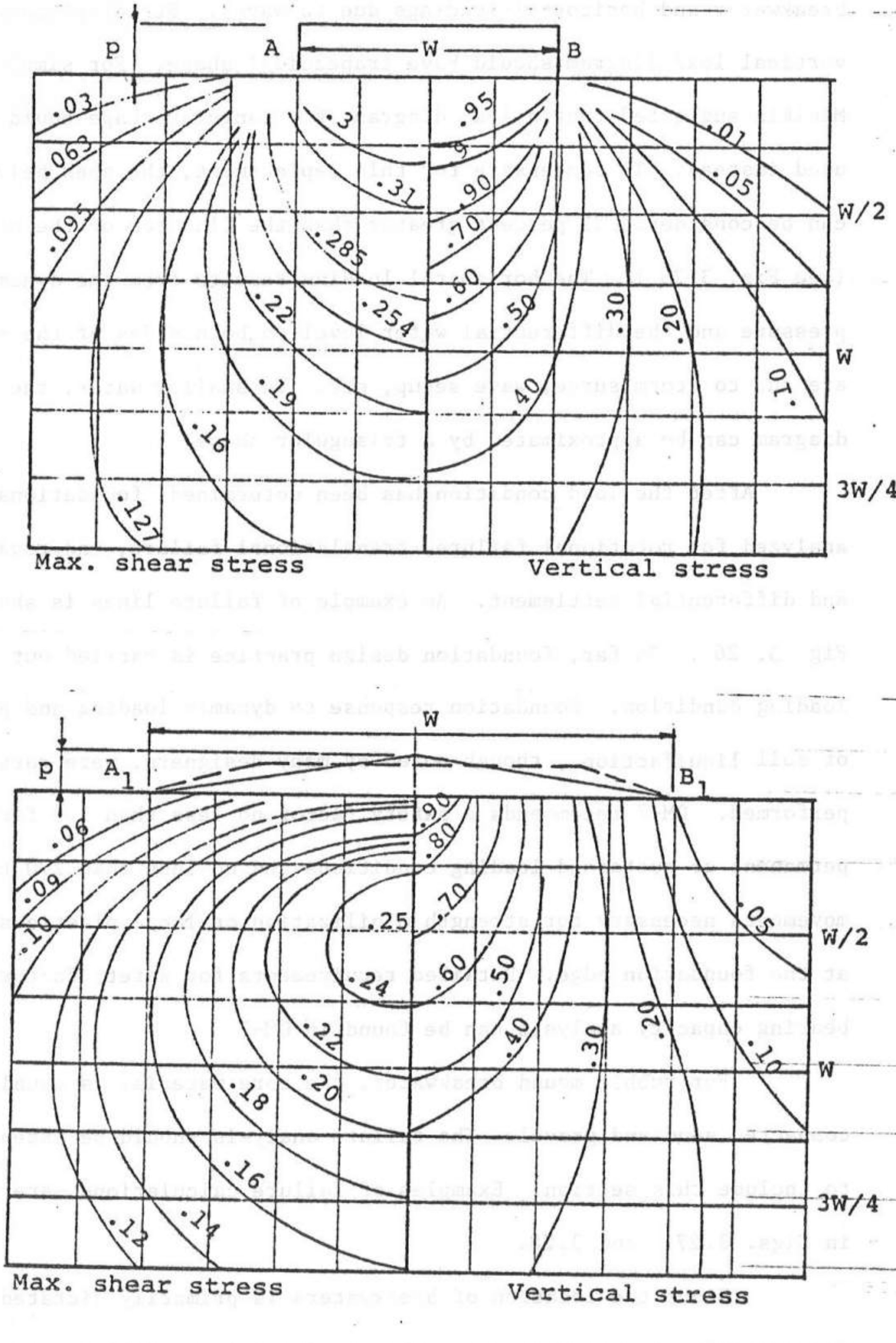


Fig. 3.25 Distribution of Stresses in Sub-Foundation Materials for Uniform and Triangular Loading Conditions

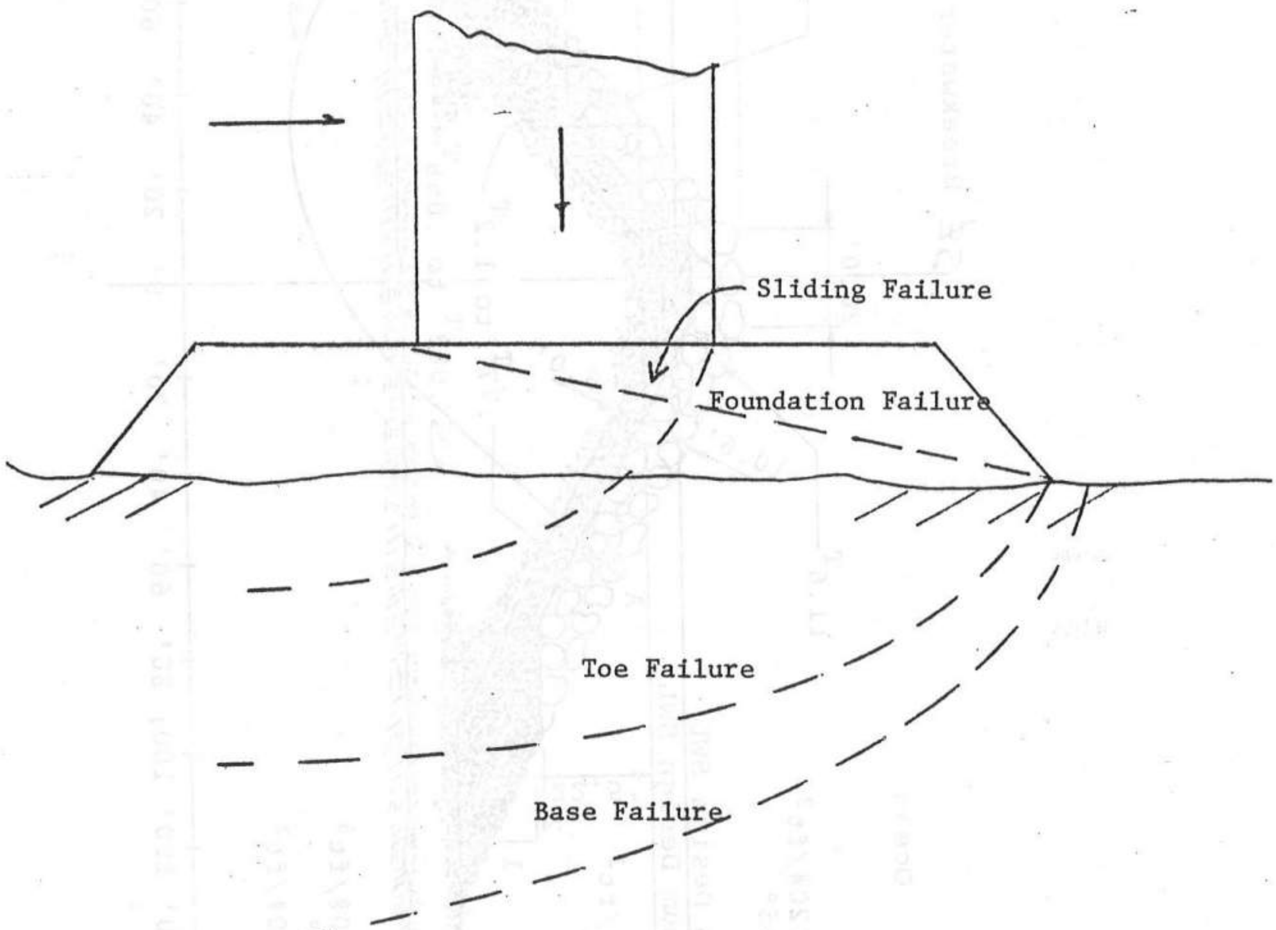


Fig. 3.26 Illustration of Failure Modes in Foundation

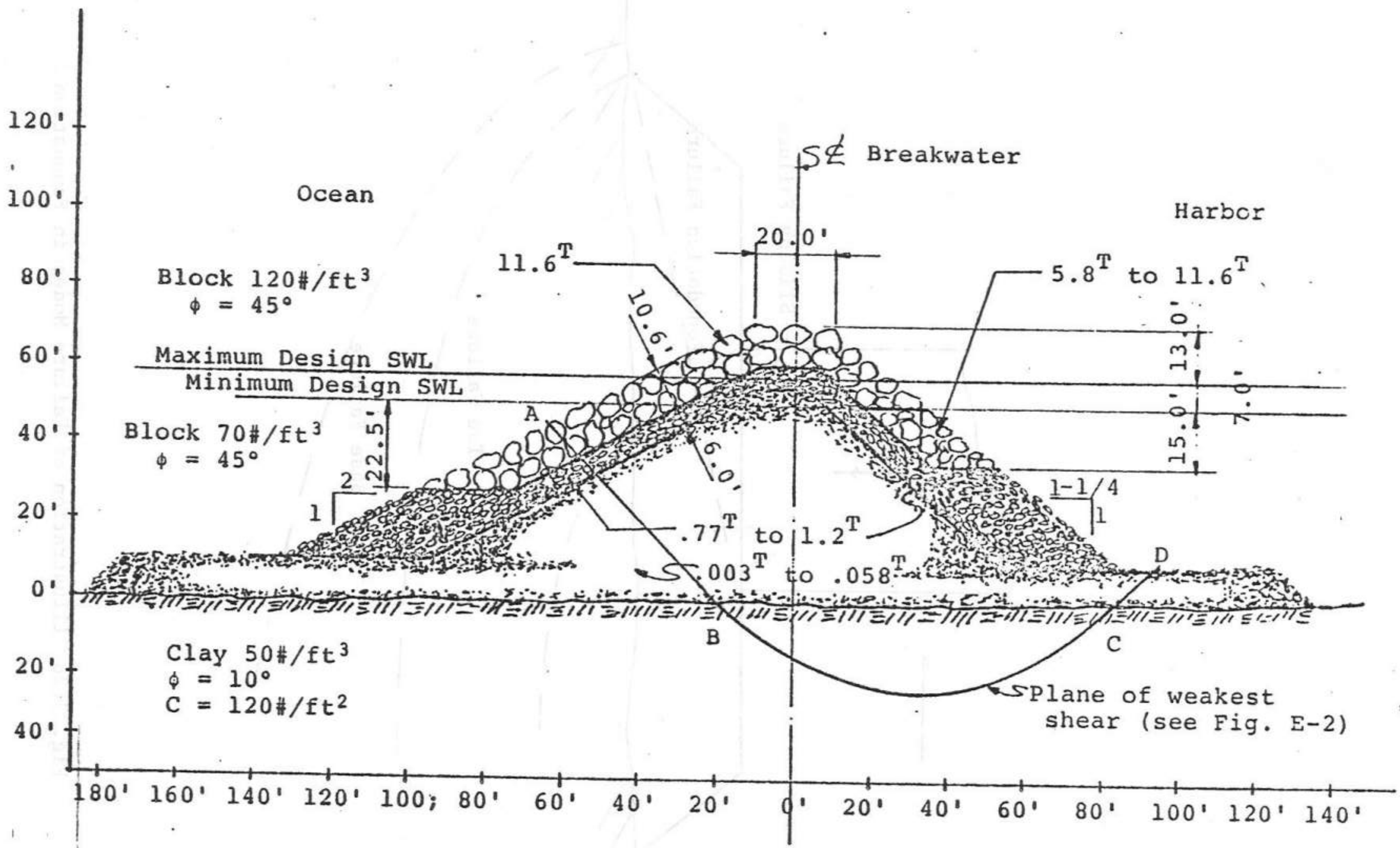


Fig. 3.27 Example of Breakwater Failure Calculation

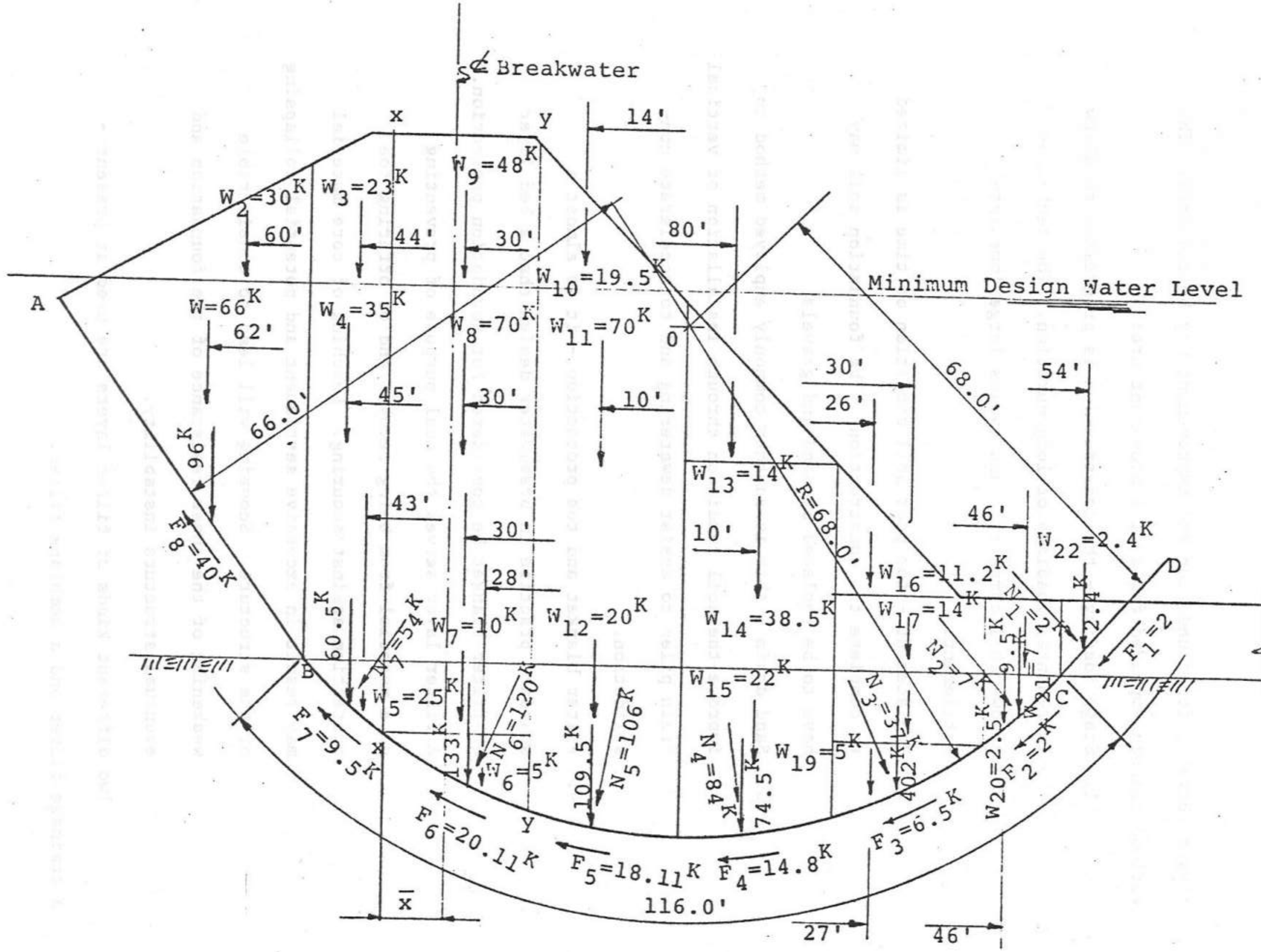


Fig. 3.28 Example of Breakwater Failure Calculations

always suitable for foundation and improvement may be required. The various methods employed for soil improvement are:

1. Stage loading - the construction is programmed to allow incremental loadings of long duration. The technique allows soil compaction and avoids large immediate settlement.
2. Replacement - for poor soil condition or time is limited to complete the construction. The foundation soil may have to be replaced by sand and gravels.
3. Sand drain - this is another commonly employed method to improve the soil condition through installation of vertical drain piles to assist dewatering and to accelerate consolidation.
4. Filter blanket and toe protection - it is almost a standard practice in breakwater design that a bed layer or filter blanket be considered for foundation protection. A filter layer serves the dual purpose of preventing core material from being removed and of offering toe protection against scouring. Leaching of core material may result in excessive settlement and potential collapsing of the structure. Scouring will lead to considerable weakening of the shear resistance of the foundation and eventual structure instability.

Two different kinds of filter layers are used at present - a drainage filter and a membrane filter.

The drainage filter is graded material of fine sands, course sands and gravels such that it provides good drainage to permit quick release of hydrostatic pressure and at the same time to prevent core material from leaching out. The graded material could be in two or more protective layers. The design procedures involved a) make mechanical analysis of the base material; b) estimate sizes of voids in the cover rubble stones; c) design filter by Terzaghi criteria as revised by the U. S. Army Engineers Waterways Experiment Station (Posey, 1961 and 1971). It stipulates that:

$$\frac{D_{15}\ \text{Filter}}{D_{85}\ \text{Base}} < 5$$

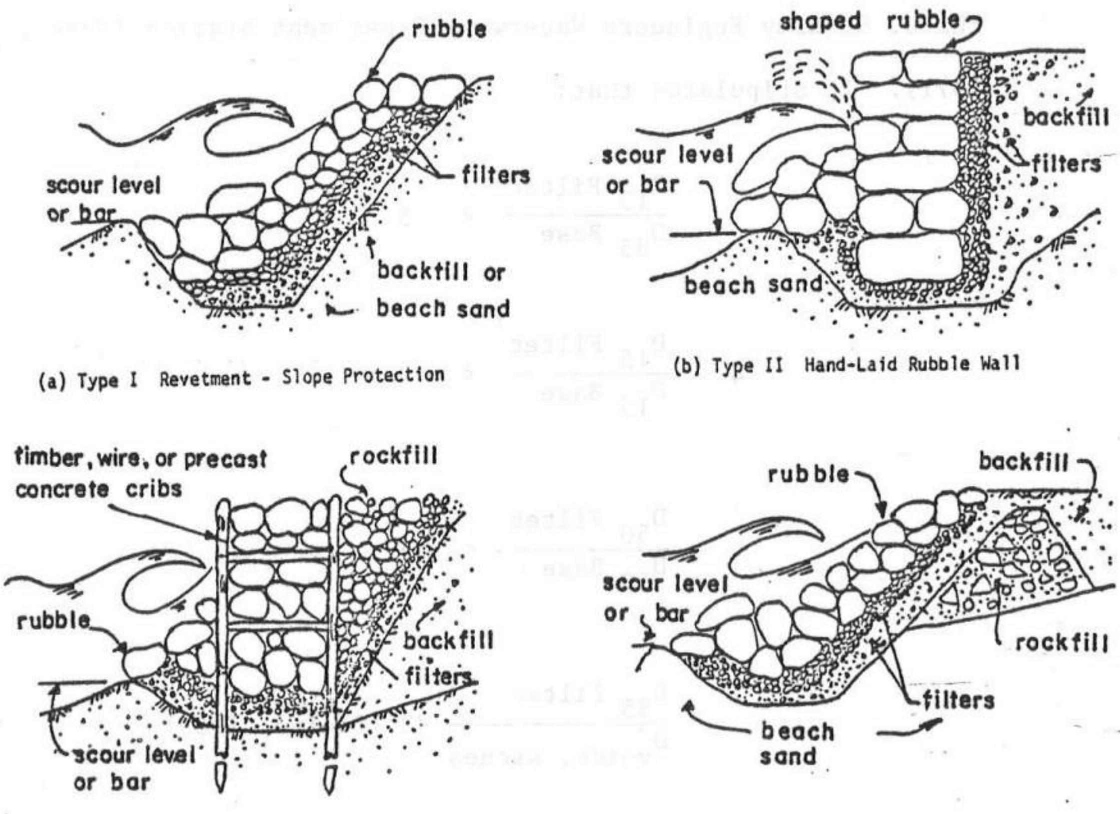
$$4 < \frac{D_{15}\ \text{Filter}}{D_{15}\ \text{Base}} < 20$$

$$\frac{D_{50}\ \text{Filter}}{D_{50}\ \text{Base}} < 25$$

$$\frac{D_{85}\ \text{Filter}}{D_{\text{voids, stones}}} < 2 \quad \text{(Seelye, 1965)}$$

where D = nominal diameter of grain size usually in mm, and for example D_{50} means 50% grain size.

The thickness of the filter mentioned should be adequate for complete coverage of subgrade and base material. The layer should be extended beyond the possible scouring zone. Typical filter layer arrangements are shown in Fig. 3. 29 .



(a) Type I Revetment - Slope Protection

(b) Type II Hand-Laid Rubble Wall

(c) Type III Crib Wall

(d) Type IV Rubble Mound Wall

Fig. 3.29 Typical Filter Layer Arrangement

The membrane filter layer is woven of synthetic fibers such as polyvinylidene chloride resin monofilament yarns and polypropylene monofilament yarns. At present, this type of filter has not been widely used because of lack of experience. The design criteria have been discussed by Lee (Ref.227).He also presented comparisons between the graded filter system and the membrane filter system.

One of the difficulties in filter layer and toe protection design is the problem of estimating scouring forces and the extent of the scouring zone. The information is vital for successful design and yet it is difficult to obtain, and often qualitative. We only know that the scouring is strongly related to wave characteristics, reflection from the structure and current strength and direction. The current design practice relies heavily on experience which, in this case, is unfortunately difficult to transfer from one specific case to the other.

3.2.3.5 Dynamic Analysis of Breakwater - Soil System

The scope of the dynamic analysis of breakwater-soil system encompasses establishing stress-strain relationship when the structure is subjected to earthquake ground motion, and accessing the potential of foundation soil liquefaction. At present, dynamic analysis due to earthquake loading is not a standard practice in breakwater design. Little or no information is available in the United States. In Japan, the earthquake loading is often considered as a surcharge to the structure. One of the reasons that no serious attempt has been made to analyze earthquake loading on breakwaters is due to the fact that the breakwater has not been treated as a structure of serious consequence.

With the recent development of using breakwaters as the protective structures for floating nuclear power plants, the dynamic analysis of breakwaters subjected to earthquake loading could become another important design consideration. Methods based on finite element techniques have been developed recently to perform dynamic analysis of earth structures such as dams (Ref. 797; Ref. 795)

The finite element method is based on the concept that a continuous structure can be replaced by an assemblage of discrete elements interconnected at their model points. The equations of motion of the model points in the finite element system are then established by assigning appropriate stiffness and damping coefficients. These equations are then solved for designated excitation forces, for the model displacements. The stresses in all elements are obtained from the model displacements by a matrix transformation through stress and strain relationships.

It seems logical that such methods could be easily extended to analyze the dynamic response of breakwaters. The difficulty actually arises from:

1) The establishment of the actual values of stiffness and damping coefficients. In particular, about the armor units of various kinds.
2) The effects of water mass and how it can be incorporated into the system.
3) The dynamic effects of water waves in conjunction with the earthquake loading.

4) The definition of structural safety or failure criteria and related safety margins.

Extensive research needs to be conducted before a satisfactory solution can be reached.

The problem of liquefaction potential has been discussed in a previous section. Some breakwaters, particularly rubble mound types, are rather unique structures. One really should examine the problem in two parts: firstly, what is the liquefaction potential of the foundation soil and secondly, should liquefaction occur, what will be the consequence to the structure? Would it result in total structural failure, partial damage, or just settlement? None of these problems can be easily answered within the present knowledge.

3.2.4 Damage Assessment

The modes of damage could be quite different for composite type breakwaters and rubble mound breakwaters. It is more convenient to discuss them separately.

Composite breakwaters - Damage to this type of breakwater is usually more drastic than rubble mound breakwaters. The common damage modes are:

1) Instability of upper structure

a) Structure collapsing as a whole

Possible causes - excessive wave pressure in the horizontal direction, excessive overtopping, foundation failure.

b) Structure sheared off at sections

Possible causes - inadequate bondings, initial stress developed due to differential settling along the structure and foundation settling in the vertical direction, initial stress damaging improper constructions, sudden blows due to breaking waves.

c) Structure sustaining local damages

Possible causes - constant attack of breaking waves, material deterioration, inadequate structure design.

d) Structure receptive under dynamic loadings (no adequate documentation)

e) Structure sliding with respect to foundation

Possible causes - horizontal loadings exceeds friction resistance.

2) Instability of Foundation

a) Excessive scouring

Possible causes - strong current, shallow water, wave reflection

b) Excessive settling

Possible causes - poor foundation material, time inadequate to allow initial settling, unexpected release of pore pressure, liquefaction (no documented case).

c) Foundation sliding along diagonal surface

d) Circular failure at rubble mound foundation

e) Circular failure at toe of rubble mound

f) Circular failure at base material

Rubble Mound Breakwaters - For rubble mound breakwaters, damage is almost exclusively linked to armor layer stability although causes of damage may be quite different for different cases. (See 3.2.3.3 Stability of Armor Units and Slopes of Cover Layers). Because of this reason, damage is seldomly described in terms of damage mode as in the case of composite breakwaters; rather it is commonly defined in terms of degree or extent sustained by the structure.

At present the description of damage remains to be a very confused issue. Not only does a unified standard not exist, but the terminology is also rather ambiguous.

In the United States, the U.S. Corps of Engineers' practice is commonly followed. The damage in laboratory tests is defined in terms of percentage of armor units being displaced out of the total tested. This certainly is a rather loose standard, as one would expect that at least the location of damage and the concentration of dislocated units would have some effect on the structure stability as a whole. To confuse the issue further, the no damage criterion actually allows certain degrees of damage; sometimes it allows 1 to 2% and at other times up to 5% dislocation of armor units is allowed. Judgement from individual investigators plays an important role in determining whether the structure is endangered or not.

In the Netherlands, the new revised damage criteria are defined in terms of percentage with the help of damage description. Through communication of Mr. Vinje, Head Laboratory De Voost, the damage code is provided as follows:

Code	Description of the extent and nature of the damage
"none"	0-1% of the total number of units in the cover-layer.
"slight"	2% of the total number of units in the cover-layer; only small holes in the structure with sizes of 1-2 units.
"little"	3% of the total number of units in the cover-layer; only a few holes in the structure with maximum sizes of 3 units.
"moderate"	larger holes in the cover-layer, one hole of about 10 units and some smaller ones.
"much"	a larger number of large holes of more than 15 units each, in the cover layer.
"serious"	core of the structure visible and/or cover-layer almost completely removed.
"destroyed"	smaller material in the core of the structure is no longer protected, damage is progressing fastly with time.

Other European countries such as Norway and England, use very similar damage criteria as described above with possible slight variations. In Japan, it is unclear exactly what damage criterion has been used as rubble-mound breakwaters are not really of major concern.

Many investigators have the opinion that the existing damage criteria are inadequate both in definition and uniformity. Many propositions have been put forth. Some felt that the armor unit dislocation is not really a good gauge to measure the extent of damage; rather, the extent of underlayers uncovered is a better criterion. Others expressed concern of using the total armor units as a base of measurement and suggested that the percentage damage should be expressed in terms of unit area or unit length of designated zones. Still, there are thoughts that the ability of maintaining the structure slope should serve an indication of structure integrity. All those concerns are certainly justified. One must realize that breakwater construction is really an outgrowth of ancient art. Until very recently, breakwaters have been regarded as the type of structures whose failure would only create secondary consequences. The general attitude has been that rubble mound breakwaters will sustain damage no matter how carefully designed, and that they can always be repaired afterwards. This attitude may have to be changed if breakwaters are to be used to protect more vulnerable activities such as nuclear power plants, or offshore oil terminals.

Therefore, the general trend seems inclined to demanding more precise damage description. It is the author's opinion that the damage of rubble mound breakwaters should at least be described in terms of damage location and the extent of damage incurred. The following serves an example of such a proposition:

Damage Zones

1) Water level zone - mean water level ± design wave height
2) High water zone - above water level zone, below structure crest
3) Low water zone (or zones) - below water level zone, above berm
4) Zone of berms and toe
5) Crest zone
6) Leeward zone
7) Structural head zone

Damage Modes

1) Armor units dislocation
2) Armor units breakage
3) Exposure of under layer
4) Leaching of core material and undermining
5) Scouring and erosion
6) Bleaching sections
7) Meandering damage
8) Slope instability

Damage Degrees (in terms of percentage)

1) None
2) Slight
3) Little
4) Moderate
5) Much
6) Serious
7) Failure

3.2.5 Model Tests

3.2.5.1 Current Practice

Hydraulic model tests are becoming indispensable tools in breakwater design. In today's practice, breakwaters are either designed or checked through hydraulic models. Skills and model theories are highly developed and the practice is quite standardized throughout the world. The United States, Japan, Netherlands, Norway, France, England and the Republic of South Africa are all fairly well equipped to conduct large scale breakwater model tests. Cases of major test programs conducted in the past can be found in listed references.

All hydraulic model tests should preserve geometrical similarity; that is, the ratios of all homologous dimensions on prototype and models are equal. For testing of breakwaters, dynamic similarity is observed only to the extent that Froude criterion is satisfied. That is to say, forces due to gravity are assumed to dominate other kinds of forces including friction force, elastic force, surface tension and so on. This is a very common practice in hydraulic modeling since the criteria of modeling various forces at the same time are extremely difficult to fulfill. Also, like many other hydraulic modeling practices, fixed bed models are commonly used. Because of these restrictions, experimental investigations are limited to determine:

1) The stability of the structure against wave attacks (more appropriately, armor layers)

2) Wave runup and overtopping

Since models are treated as rigid bodies, elastic behavior is not usually modeled. Consequently, a wide variety of material can be used as long as the model remained rigid during testings.

3.2.5.2 Data Interpretation and Scale Effects

It is almost invariably true that the data obtained from breakwater testings suffers the problem of scattering and respeatability. This is simply because there are so many variables involved and the investigators are striving to obtain certain simple relationships among a few recognizable variables. Even if the data obtained can be accepted with certain confidence levels, and can be satisfactorily presented, a greater challenge remains to interpret them for prototype application. Without the latter step, the model testing is meaningless.

As we have mentioned before, the dynamic behavior (forces) of breakwater-fluid interaction is modeled in accordance with Froude criterion (or Froude law of scaling). Therefore, it leaves little choice but to interpret the data accordingly. The variables that could influence armor unit stability have been listed in 3.2.3.3. Among them, it is generally believed that inertial force created by the wave, the drag force created by the wave, and the friction and interlocking among armor units are the three most important factors affecting the armor stability for a given breakwater shape. The Froude criterion used in dynamic similitude only yields correct simulation of the inertial force. The other two factors will not be correctly scaled using the said scaling law. Under this condition, one faces two alternatives:

3) Wave transmission through porous structures

4) Wave force movement on vertical walls

Among them (1) is most directly related to breakwater structural design. Most laboratories perform their test in wave tanks equipped with wave generators capable of generating monochromatic waves. Recently, wave basins that permit three dimensional tests are also available. Laboratories in the Netherlands and Norway are now testing breakwater stability by irregular waves. Facilities capable of generating irregular waves are also available in the country. However, no breakwater test has been performed using these facilities.

In designing model testing, the selection of the linear scale is of first priority because it depends on many factors which follow:

a) Absolute size of model waves capable of being generated

b) Dimensions of test facilities

c) Limitation of water depth

d) Operation constraints

The effects of water depth and wavelength on the wave refraction and deflection characteristics require a geometrically undistorted model to ensure accurate simulation of wave action. Almost all the breakwater tests performed so far were using undistorted models.

The fluid most often used is water. This is because water is readily available and is more satisfactory than other fluids.

1) If the effects are separable in the model, the inertial force could correctly scale up and corrections could be made for the other two factors.
2) If the effects are inseparable in the model, the total effect has to be scaled up obeying Froude criterion and hoping that inertial effects dominate the rest.

In breakwater modeling, the second practice is followed. The scale effect is often noted but no solution has been found. Since field measurement is so difficult to obtain, there is no quantitative verification to completely endorse the current practice. It is safe to say that within the state of the art hydraulic models are indispensible for breakwater design, but one should not rely on it with over confidence.

4. EVALUATION OF CURRENT DESIGN PRACTICE

4.1 Deficiency

Before 1950, breakwater design and construction have experienced many setbacks and the confidence of engineers was quite shaky. Over the last two decades, substantial efforts have been given in research for better understanding and more rational methods of breakwater design. The hazardous nature has been appreciably lessened as evidenced by greatly reduced breakwater failures during this period. However, for one reason or another, breakwaters are still looked upon as structures of secondary consequence when failed, and engineers tend to show too great an optimism for favorable conditions and place heavy weight on economic considerations.

As is often the case in engineering problems involving environmental interactions, breakwater design suffers many deficiencies. These deficiencies can generally be categorized into two groups; (a) those resulting from lack of understanding, and (b) those resulting from new requirements. Deficiencies stem from lack of understanding belonging to one group whereas those resulting from new requirements belong to the other. Deficiencies of the first category have largely been addressed upon in the previous discussion. The deficiencies of the second category, specifically relating to applications of floating nuclear power plants, will be the main point of discussion. It should be pointed out that these two groups are not mutually exclusive.

The first problem one would like to address upon is the determination of design conditions and the related risk analysis. As just mentioned, current engineering practice tends to put too much emphasis on economics. The design parameters were often selected at the designer's discretion, often with very little research on the viability of their decision. One of the favorite excuses is that the structure failed because it encountered two one-hundred-year storms within five years. For breakwaters to be used as protective structures of serious safety consequence, much more stringent methods of analysis should be used. There seems to be two alternatives open for exploration. The first one is to select design_parameters based on probability of occurrence of extreme events and to perform meaningful risk analysis. This method requires a substantial data base to support the decision and its confidence level usually decreases as the data extrapolation extends beyond a reasonable period. The other method is to select the design parameters based on the limiting mechanisms that govern the process. For instance, it is a common practice to select design breaking wave height based on the water depth simply because of the fact that waves can physically never exceed certain limiting heights for a designated water depth. Unfortunately, not all the environmental parameters are as clear cut as breaking wave heights. Further research needs to be performed.

Another vital deficiency in the present breakwater practice is the understanding of breakwater response under earthquake loadings. There is practically no documented information directly related to breakwaters. Since technical tools are fairly well developed to handle structural analysis of continuum material under dynamic loads, a finite amount of research work may have the problem well in hand.

One of the problems that may haunt us for a while is the lack of clearly defined design requirements and damage criteria. The problem of safety margins in the usual sense of structure design is also lacking.

4.2 Recommendations

The design of breakwaters still remains, to a certain extent, an art. Experience and judgement played and still play important roles. Many aspects and problems need to be improved, understood more, or defined better. In the advent of new roles of breakwaters in the offshore installations, the following research needs to be carried out for safety purposes:

1) The design of breakwaters, both composite type and rubble mound type, against earthquake loading.

5. DESIGN REVIEW INFORMATION REQUIREMENTS

To facilitate appropriate design review, the following information should be furnished by the applicants to the fullest possible extent. Any additional information that will aid in design review should also be provided.

5.1 Narrative Description

A physical description of the breakwater shall be provided, supplemented with sketches or diagrams such that a detailed description of all pertinent features is provided including surrounding environment, geometry of breakwater, type of structures, major dimensions, typical views and geological setting. The function of the breakwater and its expected performance shall also be summarized.

5.2 Plans

The plans shall provide sufficient details with sizes, sections, and relative locations of various members of the breakwater. Different design water levels shall be dimensioned. Plans shall be drawn to a scale large enough to convey the information adequately.

5.3 Data Summary

Data pertinent to design should be provided. These data should include all the physical, chemical and engineering information cited or used in design and analysis sources of information and means by which information is obtained should be properly identified.

The following kinds of information are generally required:

a) Long-term wind statistics, in particular, storm or hurricane statistics. The information should include the speed, the duration and the direction.

b) Wave statistics,including wave height, period and direction. Information on wave spectrum.

c) Tides and storm surge information including both astronomical tide and meteorological tide. Other meteorological factors that influence the meteorological tide such be documented. Such factors include atmospheric pressure anomalies rainfalls.

d) Current information including wave, tide and wind induced current, current anomalies such as downwelling and upwelling.

e) Geological information including bottom topography and soil condition. Soil foundation data down to the point where the structure has negligible effect shall be provided.

f) Earthquake and tsunami data shall be provided.

g) Other factors such as ice, fog, temperature, when applicable.

5.4 Design and Analysis Procedures

The design and analysis procedures shall be described including:

a) Determination of Design Parameters--this shall include the methods, the hypothesis, the analysis and the results of determining design input parameters such as:

1) Design wind, wave and current conditions.

2) Design water levels and storm tides.

3) Design foundation information including physical properties,

mechanical properties, and dynamics properties.

4) Design earthquake input spectrum and design tsunami conditions.

5) Combinations of design parameters.

b) Determination of loads and loading combinations--this shall include the type of loads acting on these structures such as dead, live, earth pressures, static and dynamic loadings due to water and impact loadings. Loads resulting from natural phenomena; such as, earthquakes, tornados, hurricanes, and other time-dependent loads unique to the site should be described. The load factors used and their justifications.

c) Methods and Techniques--this shall include the assumptions made and the identification of boundary conditions. The expected behavior under load and the range of design variables that influence the results of the analysis should be provided. The design codes, standards, specifications, regulations, general design criteria, safety guides and other industry standard practices that are applied should be identified. The specific edition, date, or addenda of the applicable documents should be identified including any exceptions taken, and their justification, and/or application of substitute provisions. The limitations and assumptions made in the referenced documents should be discussed in relation to the procedures outlined for the design. The Degree of conformance with the related AEC safety guides should be discussed. Computer programs that are utilized should be referenced to permit identification with available published programs. Propriety computer programs should be described to the maximum extent practical to establish the applicability of the program and the measures taken to validate the program with solutions derived from other acceptable programs.

d) Documentation of Experimental Testing Program--a discussion of all experimental test programs initiated by the applicant to verify design parameters, to determine design conditions and to test performance should be included along with a list for each experimental test program. A more detailed summary of the experimental test program including such items as experimental design, procedure, data acquisition, data analysis and interpretation, applicability and limitation to prototype application and level of confidence should be included.

5.5 Materials, Quality Control and Special Construction Techniques

The materials, quality control procedures, and the special construction techniques used for the breakwater should be identified. A summary of the material specifications to provide information on the engineering properties of the materials and the quality control procedures that will be used to maintain quality should be submitted. A comparison of the actual materials used, results of the quality control program and results of the construction techniques should be provided (FSAR).

5.6 Testing and Inservice Surveillance Requirements

The testing and inservice surveillance requirements for the breakwater should be defined. The objectives of the surveillance and maintenance programs relating to crest width and elevation and the thickness and density of the armor and other areas important to the safety function should be provided. Details of the inservice surveillance and maintenance programs should be provided (FSAR).

6. DESIGN REVIEW GUIDELINES

To facilitate design evaluation and to maintain comprehensive but not excessive review practice, the review items discussed below are rated as essential (E), important (I), and optional (O) whenever possible, according to their relative importance to the structure safety. Commonly accepted practices and elaborations will be referred to the appropriate section(s) that deal with the specific topic.

6.1 Design Criteria

A. Function of Structure (I)

B. Expected Performance (E)-expected protection offered, structural integrity under various loading conditions, life of structure, etc.

Item: Basin Agitation

Common standard: none

Recommended standard: Under storm wave condition the plant agitations shall not exceed those set by the manufacturer for safe operation and safe shut-down conditions. Under long wave condition, the plant agitations shall not exceed comfortable operation conditions.

Item: Overtopping

Common standard: none

Recomennded standard: No blue water topping shall be allowed. Minimum amount while water topping spray is allowed only when justified.

Item: Structure Intergraty

Common standard: For rubble-mound type, damage under design wave condition shall not exceed 2 to 5% (Definition of damage varies). For composite breakwater no clearly defined criterion.

6.2 Design Parameters (Ref. Sec. 3.2)(E)

A. Design Environmental Parameters--Design environmental parameters shall, in general, including those listed in the following table; any additional parameters pertinent to the plant location shall also be reviewed, methodologies employed in determining these parameters and risk analysis shall be reviewed. (See Table 6.1)

B. Geological and Soil Information (E)--information to support 1) foundation design; 2) near field wave interaction analysis; 3) erosion assessment; 4) liquefaction analsyis shall be reviewed. The following information and analysis are generally required:

Hydrographic survey:

1. adequacy to permit design and construction of breakwaters--no common standard; 1 to 5-foot contour mappings are generally required depending on the size of the structure and the geomophology of the area.

2. adequacy to permit computation of near-field wave refraction and wave energy concentration--no common standard. Contours no more than 10-foot interval may be required up to $d/L \geq 1/2$ where d is the water depth and L is the longest wave length under consideration.

Soil information

1. For sandy and muddy sea floors, core samples are required. The depth of core and the density of coring should be adequate for foundation design

TABLE 6.1 DESIGN PARAMETERS

			Common Practice		
Item	Parameters	Elements	Method of Analysis	Risk Statement	Limitations and Remarks
1	Storm	Duration Frequency (return period)	Data extrapolation Statistical analysis	Return period	accuracy dependent upon data, its reliability, duration and frequency of collection
2	Tornado	Peak strength, Radius of inference, Advancing speed, Pressure difference	Fluid mechanics model (Sec. 3.2.1.1)	No	Not usually considered in breakwater design
3	Wind	Strength Duration Direction	Statistical analysis of data (Sec. 3.2.1.1)	Yes, Return period probability of encounter, confident level	Data dependent
4	Waves	Height Period Direction Spectrum	Statistical analysis (3.2.1.2) Wind wave spectrum prediction	Yes, same as wind	Data dependent deepwater waves only
5	Swell	Height Period Direction	Data analysis	No	Data dependent
6	Astronomical Tide	Type Spring tide-high and low	Tidal charts on table	No	
7	Meteological Tide (Surge)	Height	a) Data analysis b) Computation in terms meteological parameters (Sec. 3.2.1.3)	Yes, return period	no universally acclaimed method
8	Water Level	Mean Extreme high Extreme low	Survey plus computation (Sec. 3.2.1.3)	Yes	

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9	Current	Tidal Wind-driven current Wave induced	Field measurement Computation (Sec. 3.2.1.4)	No	
10	Earthquake	magnitude frequency of occur-rence, spectrum		Yes	Not commonly considered in breakwater design
11	Tsunami	period direction height in deepwater amplification in shallow water	Longwave computation	No	Water depth differential on breakwater is the major concern
12	Ice				Mostly descriptive
13	Rainfall				Minor influence on water level computation
14	Other meteo-logical data	Temperature Pressures, fog etc.			Minor concern in breakwater design

TABLE 6.1 DESIGN PARAMETERS

information. It is not uncommon to require deep cores 150 to 200 feet. Corings should be taken both along the axis of the breakwater and perpendicular to the axis to provide information for settling analysis and foundation failure analysis.

2. For rocky bottom, seismic profiles shall be reviewed to determine possible underwater fault, effects on underwater aquafer.

3. Laboratory test procedures and results shall be reviewed including the physical properties, mechanical properties and the dynamic properties of the foundation soil (Sec. 3.2.1.6). Test procedures and items, in general, follow standard soil testing practice.

Physical Properties	Mechanical Properties	Dynamic and Liquefaction Properties
Unit Weight	Consolidation	Grain Size
Grain Size	Shear Strength	Critical Void Ratio
Permeability	Pore Pressure	Resonant Column Test
Cohesion	Penetration Resistance	Triaxial Cyclic Test
Porosity		Shear Test Confinement Pressure Penetration Resistance

TABLE 6.2 COMMON SOIL PROPERTY TESTS

6.3 Forces, Stresses, and Strain Under Extreme Conditions

A. Combinations of Extreme Design Parameters (E)

The combinations of design parameters shall be reviewed including 1) values of design parameters, 2) basis for selecting various combinations, 3) consistency of assigned risk.

The environmental parameters most pertinent to breakwater design are wind, waves, swell, level of water, current, earthquake and tsunami. Extreme conditions of these parameters do not usually occur simultaneously. The selection of combinations depends to a certain degree on their statistical influences. The most conservative design condition would be the highly unlikely case that all the environmental parameters are statistically, totally dependent. On the other end of the scale, the least conservative situation would be under the assumption that all the parameters are statistically independent to each other. By analyzing the cause-effect of these parameters one should be able to reach a reasonable set of design conditions. For instance, if one assumes the statistical influence can be artificially differentiated at four levels--strong-dependent, moderate-dependent, weak-dependent and independent, a table like the following can be developed to aid in selection of design parameters.

	Wave	Swell	High Water	Low Water	Current	Earthquake	Tsunami
Wind	Strong	Moderate	Strong	Moderate	Strong	Independent	Independent
Wave		Weak	Strong	Weak	Moderate	Weak	Independent
Swell			Moderate	Weak	Moderate	Weak	Moderate
High Water				Independent	Strong	Independent	Strong
Low Water					Weak	Independent	Strong
Current						Independent	Moderate
Earthquake							Independent

TABLE 6.3 EXAMPLE OF STATISTICAL INFLUENCE OF ENVIRONMENTAL FACTORS

Thus, one may reason that a combination of high wind, high water, high wave, strong current and moderate swell is a likely one whereas a strong earthquake with moderate wave and swell and normal wind will be another combination with comparable degrees of risk if the individual design value is a priori determined at the same risk level.

B. Loads and Load Conditions, Types of Loads (E)(Sec. 3.2.3.1)

The following table provides the load conditions commonly considered in breakwater design. Detail force analysis is only performed in composite breakwater design. For rubble mound breakwaters, external forces are implicit design factors.

		Common Practice	Remarks
Static Load	Dead Weight		
	Buoyancy		
	Hydrostatic force due to differential water level		Caused by Tsunami
	Wind Load	Ignored	
	Current Load	Ignored	
Dynamic Load	Wave force: non-breaking	Saintlow or Minikin method	for vertical wall only
	Wave force: breaking	Minikin or Hiroi formula	still rely on experiment
	Wave force: broken	Momentum balance method	
	Earthquake	Ignored	input, in general, in terms of acceleration rather than loading.
	Ship Collision	Ignored	experiment

TABLE 6.4 LOAD TYPES IN BREAKWATER DESIGN

C. Stress Analysis Under Static Conditions--Essential for Composite Type or Caisson Structures: Standard practice of stress analysis on retaining walls, dam structures, or caisson design are followed. Analysis shall be performed for both reinforcement and concrete. For caisson design, side walls, inside walls, and bottom slabs shall be examined separately. The caisson as a whole is examined as a beam for sagging and hogging during towing, installation and in place on soil foundation. Table 6.5 provides a breakdown on load analysis for caisson structures.

Stress analysis is usually not performed for rubble mound structures.

D. Stress and Strain Analysis of Structure--Soil system under dynamic loading conditions (combination of earthquake and wave impact)(E). No standard method. Best available method in analysis is the finite element method developed for dam structure design. Some of the limitations of this method are:

1. The effect of water mass is not included.

2. The wave loading is not included.

3. The actual value of damping and stiffness coefficients have not been established.

In reviewing this fluid-structure-soil system, attention shall be paid to how these elements are treated. Also, the reasonableness of failure and safety criteria used in the analysis shall be evaluated. In the dam structure analysis, maximum allowable strain is generally adopted as failure criterion. This criterion is believed to be valid for composite type of breakwater. For rubble-mound breakwater such criterion is open to question.

E. In summary, the review of forces, stresses and strain on breakwater structures shall emphasize the following points:

1. The soundness of selecting design parameters and the combinations.

2. The appropriateness of assumptions, methods of analysis and boundary conditions.

3. The confidence level of the results and the expected range of variability of the results.

Element	Loadings	Computations		
		Launching Stage	Installation Stage	In-Service Stage
Outer Wall Seaside Leeside Side wall Inner Wall long short Bottom slab Seaside Leeside Central	Hydrostatic pressure Dead Weight buoyancy Internal Earth pressure Wave Force (treated as static load) Surcharge (superstructure) Foundation Reaction	Floating stability, bending moment, unit stress	Initial settling Differential settling Stress analysis	Bending moment unit stress differential settling (Longitudural and lateral) overturning (S.F.> 1.2) Sliding (S.F.> 1.2) Vertical loading (bearing stress sagging and hogging

TABLE 6.5 STATIC ANALYSIS OF CAISSON STRUCTURE

6.4 Stability of Structure and Structure Components

A. For composite type or vertical wall type of structures (E) (Sec. 3.2.3.2)--stabilities of superstructures, vertical walls or caissons, foundations and base material, problems of structure head, possible damage mode(s), vulnerability and safety margin. Table 6.6 gives the general concern on the stability of vertical-wall-type or composite-type breakwater.

UPPER STRUCTURE			
Failure Mode	Causes	Historical Case	Method of Evaluation
Collapses as a whole	excessive wave pressure excessive overtopping differential water level due to tsunami foundation failure	Yes	analytical
Section sheared off	inadequate bondings differential settling breaking wave force	Yes	No adequate method
Local damages	breaking waves deterioration	Yes	No adequate method
Structure ruptured	dynamic loading	no adequate document	Finite element method
Sliding	excessive horizontal loading	Yes	analytical
FOUNDATION			
Failure Modes	Causes	Historical Case	Method of Evaluation
Excessive settling	poor foundation material inadequate initial setling unexpected release of pore pressure liquefaction	Yes	field information
Sliding along diagonal surface	inadequate foundation shear resistance	Yes	analytical
Circular failure	poor foundation material	Yes	analytical
Undermining	excessive scouring	Yes	Experimental

STRUCTURE HEAD			
Failure Modes	Causes	Historical Case	Method of Evaluation
collapsing	same as upper structure	Yes	experiment
sliding and turning	same as upper structure	Yes	no adequate method

Table 6.6 STABILITY OF COMPOSITE TYPE OR VERTICAL TYPE BREAKWATER

B. For rubble mound breakwaters (E)(Sec. 3.2.3.3)--Stability of rubble-mound breakwaters is a complicated problem involving many inter-related parameters. Therefore, safety evaluation is not an easy task. Model test is almost indespensible. The solutions of many questions are still beyond the state-of-the-art even on a case by case basis. The following items are considered essential in breakwater safety review:

1. Definition of damage, degree of damage, vulnerability of damage to structure failure, range of expected variability of damage, and margin of safety.

2. Stability of armor layers, under layers, structure slope, and structure head.

3. Stability of foundations, berms (if any), rubble-mound bases (if any) against sliding, rotational failure, total settling and differential settling.

4. Liquefaction potential and structure vulnerability at various degrees of liquefaction.

5. Stability of foundation and toe protection against erosion.

6. Assumptions made and procedures followed in stability analysis-Influencing factors being specifically evaluated, neglected or lumped together and the justification for such neglecting or lumping.

It is also important to review the adequacy of the material for core and underlayer and the material and structural behavior of the armor units.

In reviewing damage criteria, Table 6.7, provides an example of the review format.

Damage Criteria = Damage Mode + Damage Zone + Degree of Damage + Safety Margin <u>Damage Criteria</u> Total Functional Failure Partial Functional Failure Severe Structure Damage Much Structure Damage Moderate Structure Damage Little Structure Damage Slight Structure Damage No Structure Damage	
<u>Damage Mode</u> Collapse (no case history) Bleaching Sections Exposure of Underlayer Leaching Core Material and Undermining Foundation Failure Scouring and Erosion Slope Instability Armor Unit Dislocation Armor Unit Breakage	increasing degree of severity ↑
<u>Damage Zone</u> Structural Head Water Level Zone—mean water level $\pm$ design wave height High Water Zone—above water level zone below structure crest zone Leeward Zone Zone of Berms and Toe Low Water Zone(s)—below water level zone above berm	increasing vulnerability ↑
<u>Damage Extent</u> In terms of percentage per unit area.	

TABLE 6.7 EXAMPLE OF DAMAGE CRITERIA FOR RUBBLE MOUND BREAKWATER

Apparantly, a conservative damage criterion shall possess the following combinations:

1. the criterion of damage mode is on the lower scale of severity
2. the damage zone is designated at low vulnerability region
3. the percentage of damage is low
4. the safety margin is high

In the current practice, the damage criterion (often referred to as the stability criterion) is to allow a certain percentage of armor unit dislocation (from 2 to 5%) under design wave condition. Although the allowed damage mode is low on the severity scale, the criterion does not speak to the crucial problem of damage zone and how rapidly the damage will propagate to higher mode if design conditions are slightly exceeded.

In rubble-mound breakwaters, the stability of armor layer is one of the most important safety items and shall be carefully reviewed. Some guidelines in this respect are discussed here. Again, one should be cautioned that the problem of armor stability is a complicated one. Descretion must be exercised to deal with each case on an individual basis.

Stability Computation

The stability computed in accordance with existing formulas shall serve only as a first approximation. The final stability of the armor units shall exceed, at least equal to, that computed.

Stability Formulas		Factors Effecting Armor Unit Stability			
Formulas (Table 3.10)	General Restrictions	Parameters		Safety Importance	Considered in the formula
Iribarren (Spain)	Scale effect nuclear armor unit treated as rigid body.	Wave	Height	Yes	Yes
Svee (Norway)		Wave	Length	Maybe	No
Hedar (Sweden)	only considers a limited number of variables explicitly.	Wave	Angle of approach	Maybe	Only at right angle
U.S.S.R.		Wave	Randomness	Maybe	No
Hudson (U.S.A.) (Most widely used)	For breakwaters of steep slope (1V to 3H as limit) and non-overtopping.	Wave	Breaking or nonbreaking	Yes	Partially
		Storm Duration		Yes	No
		Depth of Water		Maybe	Partially
		Degree of overtopping		Yes	No
		Current		Yes	No
		Earthquake		Yes	No
		Armor Unit	Shape	Yes	Yes
		Armor Unit	Unit Weight	Yes	Yes, (not for abnormal unit weight)
		Armor Unit	Manner of Placement	Yes	Yes
		Armor Unit	Number of layer	Yes	Yes
		Armor Unit	Coef. of Friction	Maybe	No
		Armor Unit	Location	Yes	Structure trunk and head
		Armor Unit	Structural Characteristics	Yes	No

TABLE 6.8 VARIABLES AND FORMULAS IN STABILITY COMPUTATION OF ARMOR UNIT

Stability Test

At present, experimental test is indespensible to establish the actual stability of armor units. To insure structural safety, satisfactory answers shall be provided on the effect of various parameters (listed in Table 6.8) to the armor stability. Anong them, the following are the more important ones that are commonly neglected in routine test:

a. The actual simulation of incident wave and the probability of multi-directional waves and swells.

b. The stability of armor units under dynamic loading including earthquake loading.

c. The margin between low-damage mode and high-damage mode and the margin between slight or no damage and heavy damage shall be investigated. If the margin is small and is within the expected variability of test parameters or the scattering of test results, the structure is not acceptable.

d. The combined effects of current and waves on armor stability, in particular, at the structure head area.

Armor Unit

The armor unit shall be reviewed in the following aspects:

a. The material--it is commonly recommended that a dense, watertight (low permeability), high quality, high-strength concrete shall be used. It is also recommended that the concrete should not be less than about 5,000 psi at 28 days.

b. The reinforcement--if no reinforcement is used, it shall be so justified through experimental testing, in particular, under conditions simulating breaking waves (the ability of armor unit to withstand wave impact loading).

c. The stability under damaged condition-- 1) the stability of broken unit, 2) the interlocking ability at advanced stage of damage, 3) the stability of armor units at waves exceeds the design condition, 4) the stability of armor unit for overtopping water.

In liquefaction and related structure vulnerability the following items shall be reviewed:

1. Sections that are vulnerable to liquefaction--usually areas of low surcharge and areas of high pore pressures.
2. Damage potential of breakwater at various degrees of liquefaction (no documented case on breakwater failure due to liquefaction).
3. Damage mode and damage zone due to liquefaction.

In determining the adequacy of core and underlayer design, the following items are of importance:

1. The grading of core materials to provide structure integrity and excessive leaching of material.
2. The adequacy of voids and porous behavior to avoid excessive built-up in hydrostatic pressure.
3. The compatibility with armor units in earthquake resistance.

6.5 Hydraulic Model Test (Essential for Rubble Mound Structure)(Sec. 3.2.5.1)

In reviewing the model test, the following items shall be reviewed:

A. Scope and comprehension of test program--ideally, the model test shall cover the effects of environmental factors crucial to structure safety, among them:

1) the effects of water wave

2) the wind load

3) the current effect, and

4) the earthquake load.

However, the standard practice in breakwater model test only considers the first item. For breakwater to be used as protective structure of floating power plant, full justification shall be provided if the other effects are not evaluated through experiment.

B. Modeling techniques and limitations--including the soundness of the simulitude criteria and the limitation of test results in design application. Since dynamic similarity cannot, in general, be fully preserved, justification shall be given as to why certain dynamics aspects are ignored in the test and what corrective measures are to be taken when the results are applied to prototype design.

C. Test conditions, procedures and repeatability of test results--the test shall cover the full range of design values. Cases crucial to structure safety shall be tested exceeding the design conditions to insure appropriate safety margins. According to past experience such cases may include the degree of wave overtopping design wave height exceedance. The effects of earthquake duration shall also be carefully examined; although, no historical cases have been documented to indicate how sensative the structure safety to earthquake duration. Tests with widely scattering results shall be carefully reviewed to insure that in no case the envelope of the scattered data exceeds the design safety criteria.

D. Data collection techniques, procedures of data analysis and interpretation, and error analysis--this shall include the range and reliability of data collection system, the sensitivity and tolerance of instrumentation, the caliboration of stability of sensing elements. For error analysis, one must be assured that the data collected are of uniform quality and that the amount of information is sufficient to allow meaningful analysis.

E. Application to design, scale effects, level of confidence and range of variability--of particular importance in this category is the scale effects. In certain cases, models of different scales may have to be used to establish the appropriate scale effect.

6.6 Structure Vulnerability Against Accidents (E)

The assumptions, the procedures of analyses and/or experiments, risk analysis and structure vulnerability shall be reviewed. If experiments are required, it is important to insure that the appropriate dynamic behavior of the system is simulated. For ship or airplane collision, for example, the elastic behavior of the breakwater must be properly simulated.

6.7 Hydraulic Effects of Breakwaters (O)

A. Degree of wave runup and overtopping, wave energy transmissibility when structure is intact and at various stages of damages shall be reviewed to determine the expected functional performance of the breakwater as claimed by the applicants.

B. Wave reflection, deflection and diffraction characteristics due to the wave-structure interaction shall be reviewed to determine whether zones of high energy concentration within and outside the breakwater exist.

6.8 Materials, Quality Control and Construction Techniques (I)

A. Material specifications including physical properties, engineering prospectives and chemical and biological stability in marine environments.

B. Quality control standards and procedures and results.

C. Material handling procedures, including manufacturing, transport and installation.

D. Construction programs and techniques and inspection.

6.9 Testing and Inservice Surveillance Requirements (I)

A. Scope and criteria of testing and inservice surveillance program for safety evaluation.

B. Risk statement, maintenance schedule repair requirements.

C. Inspection techniques and procedures.

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