CHAPTER 24

DESIGN OF BREAKWATERS

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INTRODUCTION

As the name implies, a breakwater is a barrier constructed to break up and disperse heavy seas, to shield the interior waters of a harbor from winds and waves, and to provide shelter and protection for ships, shipping facilities, and other harbor improvements. Breakwaters are structures used to improve a naturally protected (sheltered) harbor or to create a sheltered harbor at locations required for shipping, refuge, recreation, etc.

Breakwaters may be roughly divided into two main groups, the vertical-wall type and the rubble-mound type. A possible third group, the composite type, consists of the wall-type placed upon a rubble-mound foundation. Since the experience of the San Francisco District, Corps of Engineers, has been limited to the construction of rubble-mound breakwaters and jetties and inasmuch as practically all breakwaters on the Pacific Coast are of rubble-mound construction, the second half of this paper has been limited to the consideration of the design of this type of structure. The first half of the paper discusses general subjects (choice of location and type of breakwater, etc.) relevant to both types.

Until recently, the design and construction of breakwaters was largely an empirical "art" based mainly on the designer's observations of the performance of previously constructed breakwaters. Great latitude was given personal discretion and judgment, since those factors which might influence or standardize design were little understood.

It was not until 1923 that the problem of wave forces on vertical-wall structures was effectively attacked, and not until 1938 was an adequate solution evolved for the same problem in relation to sloping-faced structures. Knowledge, both theoretical and empirical, of forces on the first type of breakwater has been extended by Benezit (1923), Lira (1927), Sainflou (1928), Molitor (1935), Cagli (1935), Gourret (Catena, 1941-43), Iribarren (1949), and Minikin (1950). The work of Iribarren (1949) on sloping-faced structures, with additions in collaboration with Nogales y Olano (Iribarren and Nogales y Olano, 1950a, 1950b), appears to be the most adequate formulation of this problem. Epstein and Tyrell (1949) also developed a formula very similar to that of Iribarren (1949) for sloping-faced structures. Recently, Mathews (1948) and Rodolf have each developed formulae for the solution of this complex problem (see Chapter 26).

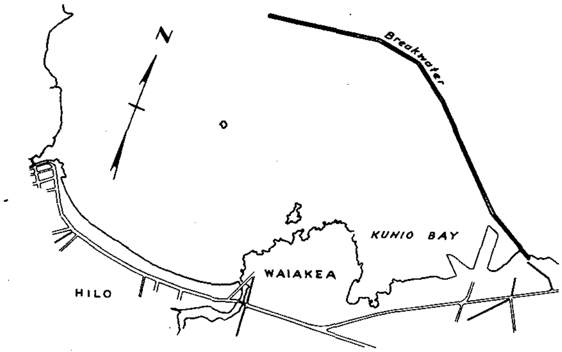
Conjecture in design consideration was further reduced when it became possible to forecast storm waves with a fair degree of accuracy (Sverdrup and Munk, 1947). This procedure, in conjunction with those for graphical construction of refraction diagrams (Johnson, O'Brien, and Isaacs, 1948) makes it possible to determine wave heights at the breakwater location, from which the "design wave," the wave which the structure is designed to resist, may be chosen.

At the present time, the "art" in breakwater design, i.e., that portion of the design problem not already fairly rigidly determined, has been restricted to choice of site, type of structure, selection of design wave, and materials to be used. Most other aspects of the problem have been more or less standardized. This paper will present fundamental principles and general and specific criteria to be applied in breakwater design with particular reference to the rubble-mound type.

SITE CONSIDERATIONS

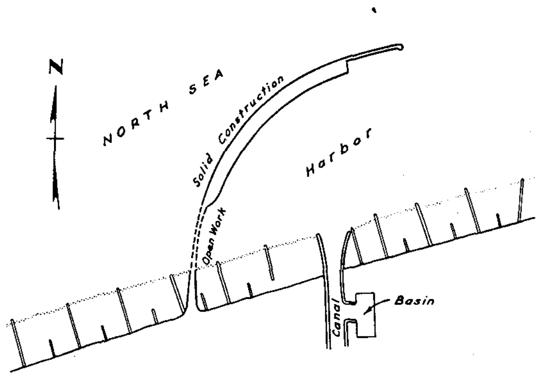
Protection offered and location of structures. The primary purpose of breakwaters is to protect harbor areas from the action of heavy seas and in locating them there is only one hard and fast criterion to apply; i.e., the structure or structures should offer the desired protection at the least possible cost. The seven following examples of types of breakwaters used at different locations present some general considerations.

a. A single breakwater extending from one shore such as Hilo Harbor, T. H. (Fig. 1-A). The harbor is a bay, or is at least so protected naturally that exposure to wave action is restricted to one direction. The breakwater may be straight or curved and extend either perpendicularly or obliquely from shore.



HILO HARBOR, HAWAII, T.H.

Fig. 1-A

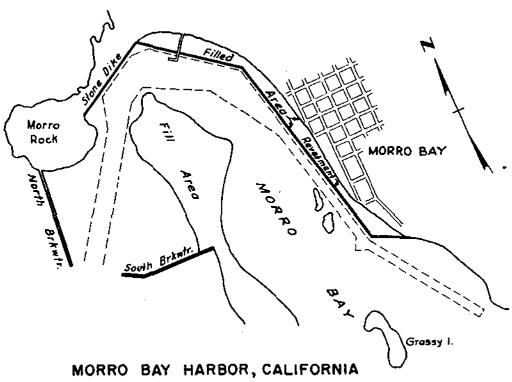


ZEEBRUGGE HARBOR, BELGIUM

Fig. 1-B

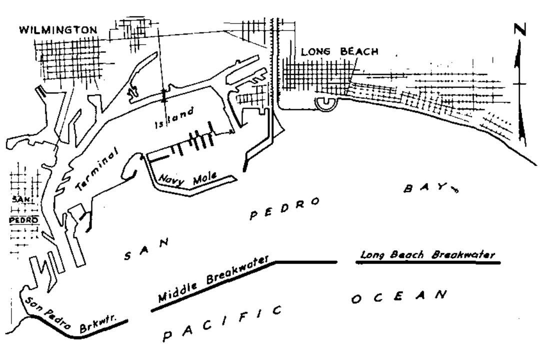
b. A single breakwater attached to shore by an open viaduct as Zeebrugge Harbor, Belgium (Fig. 1-B). The harbor is in a region of considerable littoral drift, the interruption of which is considered to be injurious either because of accretion in the harbor or shore erosion.

c. Twin breakwaters such as Morro Bay, California (Fig. 2-A). The harbor has little or no natural protection as in the case of an open roadstead or crescent-shaped bay.



MORRO BAY HARBOR, CALIFORNIA

Fig. 2-A

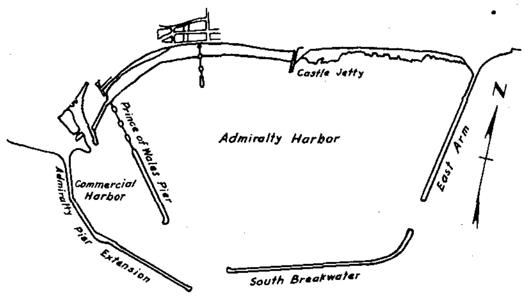


LONG BEACH HARBOR, CALIFORNIA

Fig. 2-B

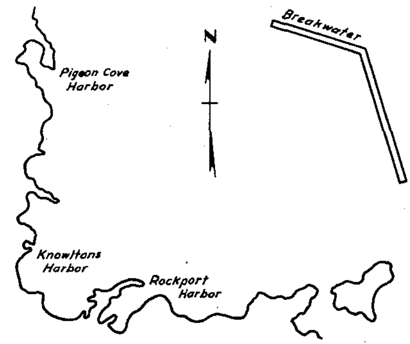
d. More than two breakwaters such as Long Beach Harbor, California (Fig. 2-B). Basic conditions are the same as in paragraph 3, but traffic and required harbor capacity considerations dictate multiple entrances. Colombo Harbor, Ceylon, or Dover Harbor, England (Fig. 3-A), are also examples of multiple breakwaters which completely enclose a harbor.

e. A single isolated (detached) breakwater such as Sandy Bay Harbor, Massachusetts (Fig. 3-B) and Santa Monica, California (Fig. 5). The Sandy Bay Harbor is located where the coast is indented somewhat and is fairly well protected. Cost and traffic considerations are factors which must be considered together with other design criteria.



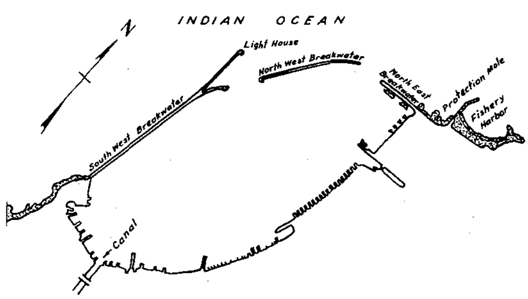
DOVER HARBOR, ENGLAND

Fig. 3-A



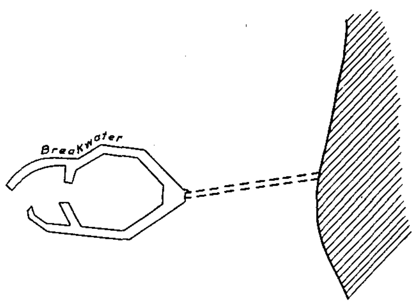
SANDY BAY HARBOR, MASSACHUSETTS

Fig. 3-B



COLOMBO HARBOR, CEYLON

Fig. 4-A



HUNDESTED ISLAND HARBOR, DENMARK

Fig. 4-B

f. Overlapping breakwaters such as have been constructed at Colombo Harbor, Ceylon (Fig. 4-A), and Genoa Harbor, Italy. The prime purpose of the overlap is to provide additional protection at the entrance or more protection inside the harbor.

g. Island breakwaters such as the Danish fishery harbor at Hundested Island (Fig. 4-B). Such breakwaters form a completely artificial harbor lying entirely outside the zone of littoral drift.

From the above examples it is seen that the "desired protection vs. least cost" rule is too broad to make the choice of site problem a simple and straightforward one. One must determine the general extent of protective works needed, the number, widths and clearances of entrance channels to be provided, and the necessity for additional protective works. Also one must weigh and balance initial costs with maintenance charges for a particular placement of breakwaters. For these determinations, studies must be made of the size of harbor to be provided or protected, the size and number of vessels using the harbor, the extent of protection from heavy swell needed for efficient harbor operation, and the maintenance problems a certain placement of structures would cause (i.e., interruption of littoral drift may cause harbor shoaling and require maintenance dredging).

Hydrography, topography and foundation investigations. After a tentative site has been selected (the necessity for construction already having been demonstrated), an intensive local-condition investigation must be made prior to any construction. Complete and detailed hydrographic surveys are necessary, not only of the tentative site but of adjacent areas. This phase of the investigation should be complete to the extent of supplementing standard survey procedures with bottom samplings, measurements of existing currents, and measurements, if possible, of amounts and directions of materials transported by these currents. These data will permit appraisal of possible alternate locations, will furnish information on foundation conditions, will provide a permanent record of conditions prior to construction, and will make possible -- through use of prior surveys, old charts, maps, etc. -- a study of any changes the bottom has undergone.

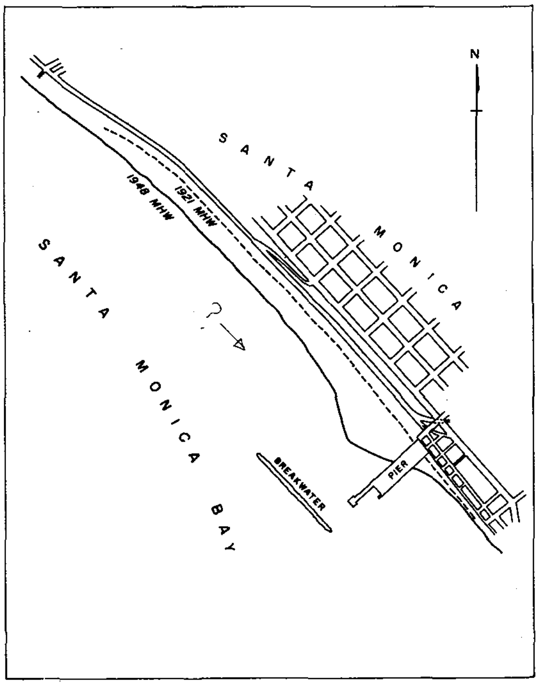


Fig. 5
Santa Monica Harbor, California

This last study, with the evaluation made of existing currents and material drift, should be used to determine, if possible, the effects of construction on shore erosion and deposition. A breakwater, in a sense, is a perturbation introduced into the natural state of a shore line, and, as such, will certainly change this state. As an example, a detached breakwater running approximately parallel to a beach will stop wave action shoreward of it, thereby interrupting, in this area, the natural state of material transport due to waves. A shoal may then form between the beach and the breakwater, and the beach downshore of the area will be eroded (see Fig. 5).

In addition to the hydrographic surveys, topographic surveys are needed showing land details in the vicinity of the breakwater and along the shore for a considerable distance on either side of the breakwater or breakwaters contemplated. These should be used as a record of shore conditions prior to construction to show the extent of any future changes the structures may cause, as well as for determining the location of the structure.

Foundation conditions must be known accurately for proper choice of the type of breakwater to be used at a particular site. The foundation analysis can be made by means of probings, washborings, or drillings. As discussed later the thoroughness of this investigation will be determined by the structural types contemplated, and the permissible settlement of these types.

Availability of materials. A factor to be considered in the choice of type of structure, as well as the exact choice of its site, is the availability of materials for construction. For instance, hydrographic and topographic surveys may indicate that construction of a breakwater is feasible at two locations, each of which would adequately provide harbor facilities desired for the area. Two or more types of breakwaters may be under consideration for each site. The cost of material transportation (a function of the distance it must be moved), and the type of materials readily available at each site then become determining factors in choosing between the two sites and between the breakwater types.

For a rubble-mound structure, the size of rock necessary to resist wave action is a function of the density of the rock, namely, $s/(s-1)^3$, where "s" is specific gravity. It is obviously desirable to use as dense rock as possible for maximum resistance. For maintenance considerations, the stone should be resistant to abrasion and deterioration by wave action. Of two sites, one may have nearby a quarry suitable for supplying only relatively small rocks, or rocks of low density. The other site's prospective quarry may be capable of providing higher density and larger stones but is located at a greater distance from the proposed harbor. A balance must be struck between the low initial cost but high maintenance costs at the first site, and the high initial cost but low maintenance charges chargeable to the second site.

For concrete or pile structures, similar cost balances must be made. The availability of aggregates of acceptable qualities will be a major factor in determining initial costs.

SELECTION OF TYPE

Types suitable for various conditions. As previously noted, breakwaters may be divided into two main groups: the vertical-wall type and the rubble-mound type. Under vertical-wall breakwaters, various sub-types exist, depending on methods and materials used in construction. These, with some general criteria of suitability, are listed below:

a. Masonry wall -- suitable in depths up to 65 ft.; requires a firm foundation; needs very little maintenance; and may be adapted for use as quays. They must not be exposed to breaking waves since the combination of high pressures due to the breaking wave (Iribarren and Nogales y Olano, 1950b) and only a small settlement of foundation can bring about total destruction (Catena, 1941-43).

b. Timber crib -- suitable in depths of from 10 to 40 ft. Cost of construction is high. Unsuitable in salt water.

c. Caisson -- suitable in depths of from 10 to 35 ft. Heavier than timber-crib structures. Suitable in both fresh and salt water.

d. Steel sheet pile -- suitable in depths up to 40 ft.; may be used in any kind of foundation into which steel piles may be driven.

None of these vertical-wall types should be used in regions where waves may break upon the structure.

Rubble-mound types are adaptable to any depth of water, are suitable on nearly all foundations, and may readily be repaired. However, they do require relatively large amounts of material, and are not suitable for use as quays without major modification.

Foundation considerations. Foundation materials vary from solid rock to soft mud, and each gradation must be dealt with differently. The quantities which need to be evaluated are firmness (compressibility), homogeneity, durability, and scourability.

A vertical-wall breakwater may be placed directly on the bottom if the bottom is firm, homogeneous, and not readily scoured. As the firmness and homogeneity lessen, a stone foundation must be put down to distribute the structure's weight; and as the bottom material becomes more susceptible to scour, rip-rap must be added to prevent scour at the toe of the structure itself. For very soft bottom materials, a pile foundation may be placed or a trench dug and filled with sand or rock. Particular attention must be paid to the possibility of settlement, for a masonry wall-type structure is subject to complete failure if its foundation settles any appreciable amount. Therefore, if a rubble substructure is used, sufficient time should be allowed for settlement before the superstructure is added. (The destruction of the Mustapha breakwater at Algiers in February 1934 is an example of complete failure of a wall-type breakwater, where "after the crest of an enormous wave passed over the breakwater without breaking, and before the passage of the following trough, the superstructure was clearly seen to have resisted the force of the wave. Then a slight trembling of the superstructure was noted, followed by the complete collapse of the breakwater as the trough of the wave passed . . .") (Catena, 1941-43.)

As noted before, a rubble-mound structure may be used on almost all types of bottoms. If the bottom material is extremely poor, it may be necessary to remove and replace it with sand or other suitable material to form a satisfactory foundation for the structure. A sand blanket may be used on certain unsatisfactory materials to form a spread foundation for the breakwater.

DESIGN OF RUBBLE-MOUND STRUCTURES

Forces. Iribarren (1949) published a formula for the calculation of stone size in rubble-mound breakwaters which, because of its rational basis, some verification, and its ease of application, has found wide use in design of these structures. It succeeded in relating stone size or weight to breakwater slope, stone density, and, most important, wave height at the breakwater.

Its use, though, was limited, for not until 1943 was any progress made toward predicting waves engendered by ocean storms. Prior to this, the only data available to predict wave heights were sporadic and generally valueless visual data gathered by unqualified shore observers. (Wave-height observations seem to double or treble the true size of swell when its magnitude is in any way out of the ordinary.)

With the advent of wave forecasting procedures and of wave refraction evaluation methods (see Chapters 4 and 8), the wealth of wave information contained in long-compiled synoptic weather maps become available to the breakwater designer, and the application of formulae, such as that of Iribarren (1949), dependent upon wave characteristics in determining structural characteristics, became more valid. Below is the Iribarren formula, in standard American units, for surface and above-surface stone weights:

$$W = \frac{k'_1 H^3 s}{(\cos\phi - \sin\phi)^3 (s - 1)^3}$$

For weights of stone at depths below the surface, H is replaced by H_1,

where

$$H_1 = \frac{\pi H^2}{L_o \sinh^2 \frac{2\pi d}{L}}$$

W = weight of stone in tons (2,000 lbs.)

k_1' = a constant = 4.68×10^{-4} for rubble
= 5.93×10^{-4} for artificial blocks

H = corrected wave height (ft.) (see below)

s = specific gravity of stone

ϕ = angle of the slope with horizontal

L = wave length (ft.)

d = depth below still water level (ft.)

In the original formulation, Iribarren (1949) suggested that H be the wave height expected at the toe of the structure. In the latest paper by Iribarren and Nogales y Olano (1950b) the determination of H is governed by a theoretical wave-steepening effect of the breakwater.

There are definite limitations to the formula. It is apparent that for slopes between 45° and 90° the $(\cos\phi - \sin\phi)^3$ term in the denominator is negative and therefore the stone weight also is negative. For a slope of 45° this term approaches zero and the stone size becomes infinite. Neither of these last statements has physical meaning. In general, though, seaward slopes are less than 45°, varying only between 1 on 1-1/4 and 1 on 1-3/4, and in this range of slopes the formula may be used.

Maintenance considerations. Unlike the rigid, vertical-wall type of breakwater, the rubble-mound type, when subjected to severe wave action, is not prone to complete failure. Rubble structures, not being monolithic, will follow more of a process of disintegration; that is, wearing away or dislodging stone by stone, rather than total collapse, and the damaged structures, if anything, will offer a more stable base for any repairs. This repairable feature makes necessary a decision between the relative costs of initial construction and maintenance in designing a rubble-mound breakwater.

It is possible, by use of synoptic charts, to predict with some accuracy a maximum design wave and, if the data are complete enough, to predict also the yearly frequency of the larger waves. It is also possible to design a breakwater to withstand, with minor repair, the largest wave expected. However, it undoubtedly would be less expensive initially, though more costly from a maintenance standpoint, to design breakwaters for a wave smaller than the maximum, and a breakwater so designed may show a lower total annual cost, including interest, amortization and maintenance cost, than one adequate to resist all storms.

We have a prime example in the San Pedro Bay breakwaters of the importance of the decision to be made between immediate and future costs. Between April 20 and 24, 1930, prior to the construction of the Middle (Los Angeles-Long Beach) detached breakwater, large waves entered the Bay and caused extensive damage to the inner Long Beach breakwater (O'Brien, 1950). In 1939, waves of destructive amplitudes caused great damage to the then partially completed detached breakwater and some damage to the San Pedro breakwater. In the first case, the swell was engendered by a southern hemisphere storm and in the second case the swell resulted from a tropical storm immediately to the south of Long Beach. (Note that wave forecasting techniques were not in usable form until 1943.) In both cases, the infrequency of occurrence of waves as destructive as these suggested that future designs be drawn for smaller but more frequent waves. The breakwaters were restored to their original conditions with no additional provisions for withstanding storms of the magnitude of these two.

Determination of crest width and elevation. When using the formula of Iribarren (1949), breakwater crest heights may be determined by using the technique of calculating breaker characteristics at a sloping face breakwater (Iribarren and Nogales, 1950b). In the case of rubble-mound structures where vessels are not likely to be moored at, or near, the structures, it is not always necessary to completely obstruct the waves, although the volume of water passing over the top should not be sufficient to cause undue disturbance in the harbor.

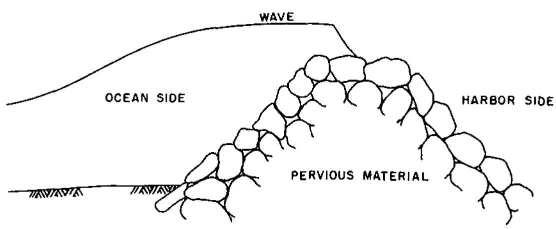


Fig. 6

Crest widths are determined more roughly. If the breakwater is so designed (for reasons of economy) that some higher waves will pass over the crest, sufficient width must be allotted to withstand forces caused by these waves. Reference to Fig. 6 will clarify the preceding statement. The cap and armor stone being pervious, an impinging wave will cause water to surge through the structure. At the harbor crest edge, there will be three concurrent phenomena:

a. The weight of the stone will be decreased by the buoyant force due to submergence, and

b. The surge through the breakwater will give rise to a force acting to dislodge the stone, and

c. The portion of the wave passing over the crest will tend to dislodge the stone.

Increasing the crest width acts to lessen these rupturing forces by first decreasing the magnitude of the surge through the permeable structure (friction and turbulence), and second, by decreasing the energy to dislodge available to the wave passing over the structure ("bottom" friction and turbulence).

Other factors in determining crest width are the method of construction decided upon, and the use to which the breakwater will be put in addition to its primary function of dispersing heavy seas. Placement by trestle requires dumping space aside the trestle. Placement by truck requires providing sufficient width for truck maneuverability. It may be desired to lay a road on the breakwater in which case its use will determine the structure's width.

Determination of slopes. Few specific criteria for determination of slopes exist, though for normal wave attack, slopes 1 on 1-1/2, to 1 on 1-3/4 will maintain their slope, with slight flattening, in deep water. It has been the practice to make the slope of the tip of a breakwater as flat as possible (up to 1 on 2) since this section is always exposed to normal (perpendicular) wave attack. Iribarren and Nogales y Olano (1950b) indicate that the stone-size slope relationships they have developed give stable configurations. This verification is based on Larras and Colin (1947-48) determination of stable slopes for the breakwater at Argel after repairs to numerous deficient slopes.

An interesting aspect of the slope determination problem comes to light when the mechanics of wave force exerted on either vertical-faced or sloping-faced structures are examined. For non-breaking waves, forces on vertical structures are a function of the height of the standing wave (or clapotis) created by complete wave reflection, whereas, mound breakwaters, because of their slope, are always subject to attack by breaking waves. The force caused by the impact of a fully breaking wave is much greater than the force arising from the same wave being completely reflected (Morison, 1948). As an extreme, if the slope of a breakwater were decreased to the point where it became relatively flat beach, wave breaking then would cause almost no reflection. Here we have two extremes of wave action; the vertical-wall causing complete reflection, and the horizontal-slope causing no reflection. Certainly, slopes in between these extremes cause a combination of breaking and reflected waves, with some intermediate slope causing a half and half division.

This division is of little importance, especially on the Pacific Coast, in the design of outer breakwaters where the depths of construction are such that waves are likely to break at, or even slightly before, the structure. Such a wave engenders very high shock pressure, and a discussion of the division of energy available in the breaking or reflected wave is academic. However, for breakwaters in the interior of a harbor, placed there mainly to further reduce wave action, the division between reflected and breaking waves is important. A vertical-faced structure will reflect unchanged a non-breaking wave and the successive interaction between reflected and incident waves will only add to roughness of sea in the harbor. A sloping-faced structure, in causing waves to break, will decrease the amplitude of those reflected. In a discussion of the problem by Iribarren and Nogales y Olano (1950a), two points of importance may be mentioned:

a. To the findings of workers of the Laboratory of Delft, giving ratios of reflected to incident wave heights, they apply the batter (inverse of the slope) as a parameter and find quite consistent results, i.e., the ratio of reflected to incident wave heights increases almost linearly with decrease in batter (increase in slope).

b. They propose the slope $\frac{8}{T}\sqrt{\frac{H}{2g}}$ as the limiting one between reflection and breaking. (T = period of wave and H = wave height.)

Zones of stone and stone classification. As pointed out previously, stone used should be dense and resistant to abrasion. Stone sizes are determined by design wave characteristics though absolute consistency in size and weight cannot be expected. General criteria for zones of stone and placement are fairly easy to establish.

The stone used may be roughly classified as follows:

a. (Armor stone). This is the principal protective covering, which is exposed to the most violent wave action. Its adequacy determines the success or failure of the structure.

b. (Secondary protective covering). The armor stone, being necessarily large, will make for large voids in the principal protective region. A breaking wave would tend to wash away the smaller and unclassified materials constituting the core if no secondary covering is used. This stone, then, consists of rock smaller than armor but still large enough to resist by itself the turbulent flow through the primary covering voids.

c. (Core). This serves as support for the protective cover, and in itself prevents the propagation of swell through the breakwater. It consists of rubble of different sizes so graded and placed as to present maximum compactness with a minimum of cavities. The minimum size of its constituents is limited since this portion of the work is "floated" in place and, therefore, must be of sufficient size to place itself by action of gravity alone -- in spite of currents or swell.

The thickness of the zones vary but again practice has set up general criteria. The armor stone should be a minimum of 2 layers thick (very roughly 10 ft. for 10 tons average weight of stone). The secondary protective stone should be 3 to 4 layers thick (about the same width as the principal protective covering for 5-ton average weight).

CONCLUSIONS

In a way, a paper such as this, dealing concisely with design consideration for breakwaters, can never be adequately prepared. The field is not a closed one. Each of the preceding paragraphs by itself would warrant at least as much discussion as this entire presentation. The selection of site offers numerous complexities which should be fully dealt with. There is much disagreement on methods of calculating stone sizes and slopes. The formulae of Mathews, Rodolf, and Epstein disagree with that of Iribarren, but the applicability of these formulae versus Iribarren's has not been touched on.

There are problems which, at the present time, have no solution. In the determination of crest width, for example, some factors contributing to instability of harbor-side stones have been mentioned but not evaluated.

However, the purpose of this paper is to present only a statement of the problems, techniques, and criteria now encountered in the field of breakwater design. If it in any way stimulates discussion of these factors, its hoped-for function will have been fulfilled.

ACKNOWLEDGEMENT

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REFERENCES

Benezit, V. (1923). Essai sur les digues maritimes verticales: Annales des Ponts et Chaussees, Partie Technique, vol. 93, Report No. 27, pp. 125-159.

Cagli, E.C. (1935). Design of breakwaters with vertical sides. Effect of wave action. Mathematical determination and methods of construction. Lessons gained from experience: XVI International Congress of Navigation, Brussels, 2nd section; Ocean Navigation, Report No. 80, pp. 1-30.

Catena, M.M. (1941). Algunas ideas sobre la inestabilidad de la infraestructura de escollera en los diques de paramento vertical: Revista de Obras Publicas, vol. LXXVII, pp. 60-63 (Translation available at Waterways Experiment Station, Vicksburg, Miss.).

Catena, M.M. (1941-43). Diques de abrigo en puerto: Revista de Obras Publicas, Part I, vol. LXXVII; Part II, vol. LXXIX, pp. 319-23; Part III, vol. LXXIX, pp. 365-372; Part IV, vol. LXXIX, pp. 407-411. (Translation available at Waterways Experiment Station, Vicksburg, Miss.).

Catena, M.M. (1943). Harbour breakwaters: Dock and Harbour Authority, vol. 24, pp. 5-7, and 32-33.

Chao, H., Chu, T., and Hsu, Z. (1945). Study of harbor design: vol. II, pp. 32-36. Waterways Experiment Station, Corps of Engineers, Vicksburg, Miss.

Epstein, H., and Tyrell, F.C. (1949). Design of rubble-mound breakwaters: XVII International Navigation Congress, Sec. II, Communication 4, Lisbon, pp. 81-98.

Great Lakes Division, Corps of Engineers (1938). Study of wave force: Prepared in Office of the Division Engineer, Cleveland, Ohio (mimeographed).

Iribarren Cavanilles, R. (1949). A formula for the calculation of rock fill dikes (A translation): Bulletin of the Beach Erosion Board, vol. 3, pp. 1-16, Corps of Engineers, Washington, D.C.

Iribarren Cavanilles, R., and C. Nogales y Olano (1950a). Talud limite entre la rotura y la reflexion de las olas: Revista de Obras Publicas; pp. 65-72; A translation appears in the Bulletin of the Beach Erosion Board, vol. 5, No. 2, pp. 1-12, Corps of Engineers, Washington, D.C.

Iribarren Cavanilles, R., and C. Nogales y Olano (1950b). Generalization of the formula for calculation of rock fill dikes, and verification of its coefficient: Revista de Obras Publicas pp. 227-239. (Translation available at Beach Erosion Board, Washington, D.C.).

Johnson, J.W., O'Brien, M.P., and Isaacs, J.D. (1948). Graphical construction of wave refraction diagrams: Hydrographic Office, U.S. Navy, Publ. No. 605.

Larras, J., and Colin, H. (1947-48). Les ouvrages de protection du port d'Alger a talus inclines: Travaux, vol. 31, pp. 603-609; vol. 32, pp. 163-168.

Lira, J. (1927). Le calcul des brise lames a parement vertical: Genie Civil, vol. 90, pp. 140-145. (Translation available at the Beach Erosion Board, Corps of Engineers, Washington, D.C.).

Mathews, W.J. (1948). A re-evaluation of rock size formulas for rubble breakwaters: Los Angeles District, Corps of Engineers, (unpublished).

Minikin, R.R. (1950). Winds, waves, and maritime structures; studies in harbour making and in the protection of coasts: Charles Griffin and Co., Ltd., London.

Moliter, D.A. (1935). Wave pressure on sea-walls and breakwaters: Trans. Amer. Soc. Civil Engineers, vol. 100, pp. 984-1002.

Morison, J.R. (1948). Wave pressures on a vertical wall: Tech. Report No. HE-116-298, Institute of Engineering Research, University of California, Berkeley (unpublished).

O'Brien, M.P. (1950). Wave refraction at Long Beach and Santa Barbara, California: Bulletin of the Beach Erosion Board, vol. 4, pp. 1-14, Corps of Engineers, Washington, D.C.

Sainflou, G. (1928). Essai sur les digues maritimes verticales: Annales des Ponts et Chaussees, Partie Technique, vol. 98, No. IV, pp. 5-48.

Sverdrup, H.U., and Munk, W.H. (1947). Wind, sea, and swell: Theory of relations for forecasting: Hydrographic Office, U.S. Navy, Publ. No. 601.