

CHAPTER 26

DESIGN AND CONSTRUCTION OF JETTIES

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INTRODUCTION

The purpose of this paper is to present a brief outline of the general engineering procedure for the siting and design of jetties and the methods of constructing such structures. After a general presentation of the formulae proposed by various engineers to determine the size and weight of individual pieces of stone or other material which should be used under various wave heights, this paper will be devoted principally to the construction of rubble stone jetties. This is the type principally used on the Pacific Coast of the United States. In this paper the word "jetty" designates a structure extending into a body of water to direct and confine the stream or tidal flow to a selected channel.

PURPOSE OF JETTIES

Jetties are usually placed at the mouth of a river or entrance to a bay to aid in deepening and stabilizing a channel for the benefit of navigation. Properly located jetties will confine the discharge area, promote scour, and extend into deep water the point where the current slackens and transported material is deposited. Jetties also protect the ship channel from waves and cross-currents and from longshore sand movements.

TYPES OF JETTIES

Factors which influence the type of jetty and design of the structure include: the physical characteristics of the site area and its exposure to wind, waves, currents, and tides; possibility of ice damage; meteorological conditions and their effect on water conditions and currents; sea conditions including the determination of maximum wave which should be designed for and littoral currents which may affect sand movements in the locality.

The cost of construction and maintenance is usually the controlling factor in determining the type. Different types may be practical in any locality and the cost of each type, as well as the annual cost of maintenance for different types, should be estimated for comparison before final decision is made. The availability of materials will greatly influence the cost and, for some types, would affect the cost of maintenance. Rubble-mound structures require periodic repair of portions damaged by storm waves but, even though damaged, the structure as a whole still functions, whereas breakage of a vertical face monolith structure generally leads to total destruction of portions of the whole structure and high cost of repair.

The seven general types of jetties are briefly described as follows:

(1) Random stone: A rubble-mound structure is in fact a long mound of random stone. The larger pieces are placed on the outer face to afford protection from destructive waves, and the smaller sized stones are placed in the interior of the structure. This type is adaptable to any depth, may be placed on any kind of bottom, and absorbs the wave energy with little reflected wave action. This type requires relatively large amounts of material. If not carried high enough, storm waves may sweep entirely over the jetty and cause a secondary wave action in the protected area, and if the voids between the stone are too large a considerable portion of the wave energy may pass through the structure. Cross-sections of two random stone structures are shown in Fig. 1.

(2) Stone and concrete type is a combination of rubblestone and concrete. This type ranges from a rubble-mound structure, in which the voids in the upper portion of the rubble are filled with concrete, to massive concrete superstructure on rubble-mound substructure. The mound is used either as a foundation for a high

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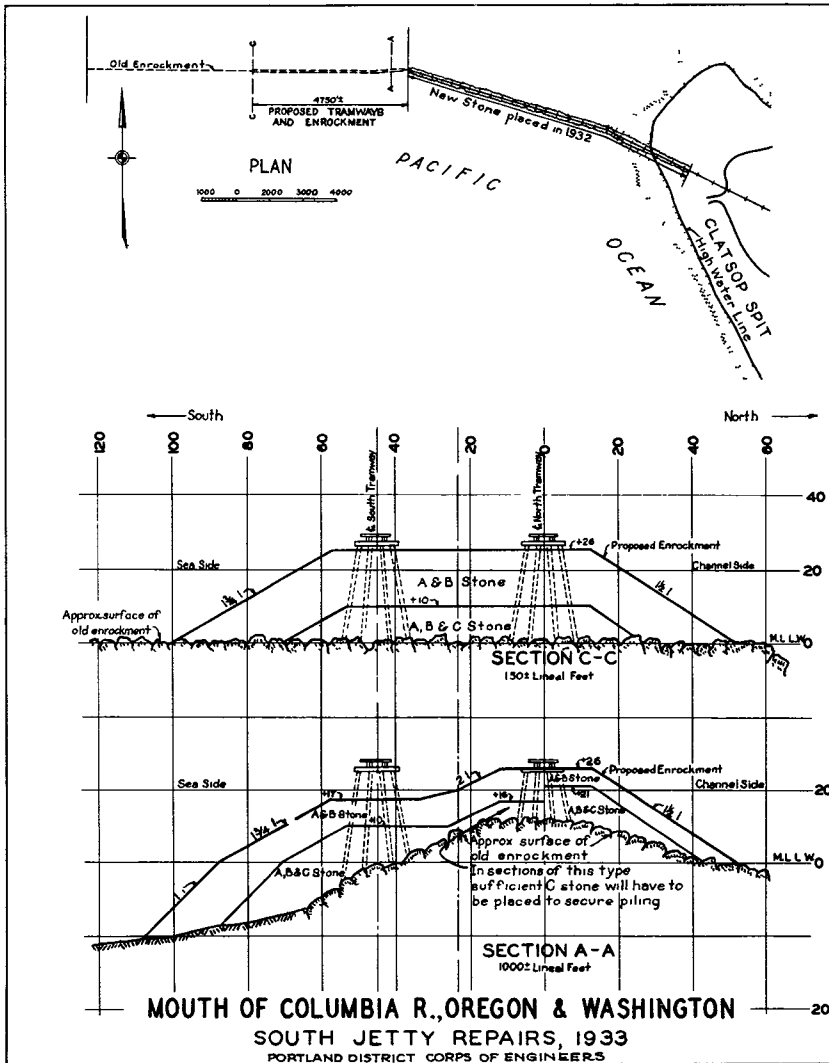
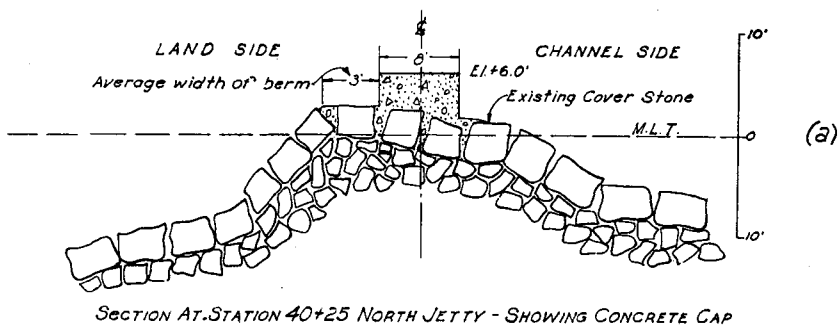


Fig. 1

concrete superstructure or as the main structure surmounted by a concrete cap with vertical, stepped, or inclined face. This type requires less material, and is used where the foundation is soft or subject to scour. The superstructure may be undermined by wave recoil down the face; rubble foundations require time to become permanently stable and should be placed years before the superstructure. This type of jetty, when properly designed and constructed, gives very satisfactory service. Cross-sections of stone and concrete jetties are shown in Figs. 2 to 7.

(3) Caisson type: The first caissons were built of iron but today they are built of reinforced concrete, floated into position, settled upon a prepared foundation, filled with stone to give stability, then capped with cap stones or concrete slab, and, occasionally, parapet walls are added. Some caissons have a reinforced concrete bottom which is an integral part of the caisson, while others, such as the ones used in constructing the Welland Ship Canal, are bottomless and are closed with a temporary wooden bottom which is removed after the caisson is placed on the foundation. Caissons are suitable for depths up to 35 ft. Foundations are either rubblestone alone or piling and rubblestone. Riprap of heavy stone is used alongside to prevent scour, to provide resistance against sliding, and to prevent weaving under wave action. On sand bottom, considerable riprap is required. The top

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TYPICAL SECTION SO. WEST JETTY - STATION 0+00 TO 15+90 - SHOWING CONCRETE CAP
FREEPORT HARBOR, TEXAS

Fig. 2

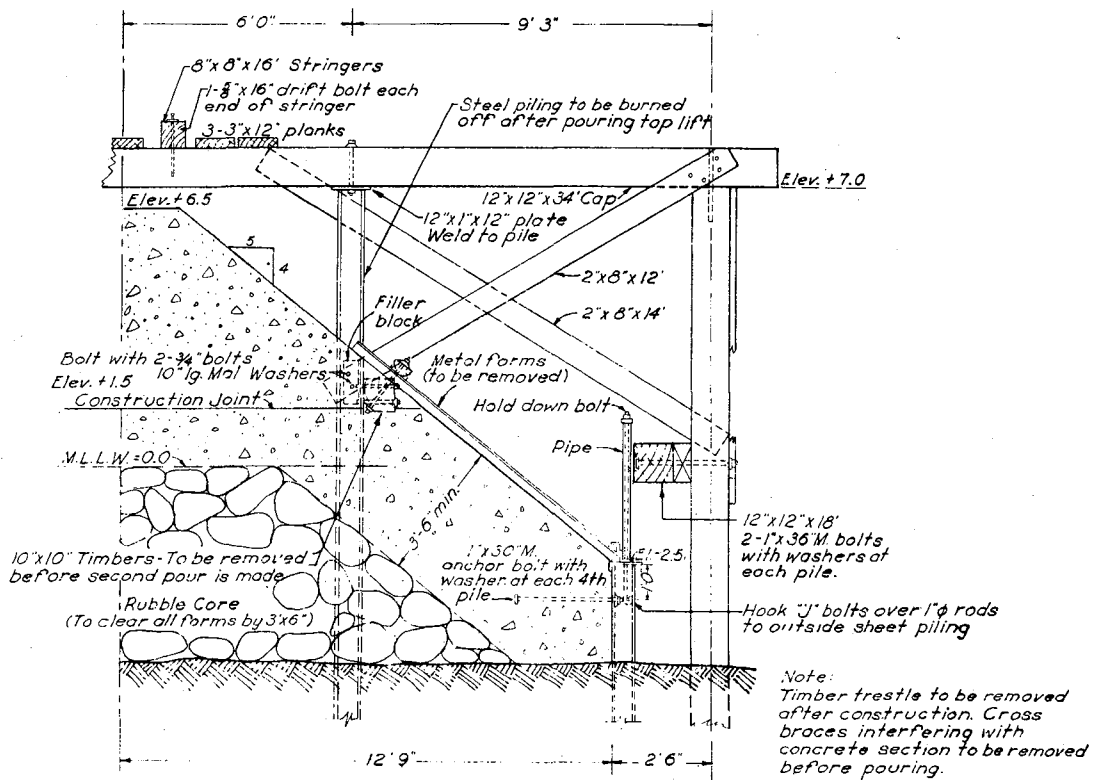


Fig. 3. Typical Section, Nome Harbor, Alaska -- West Jetty

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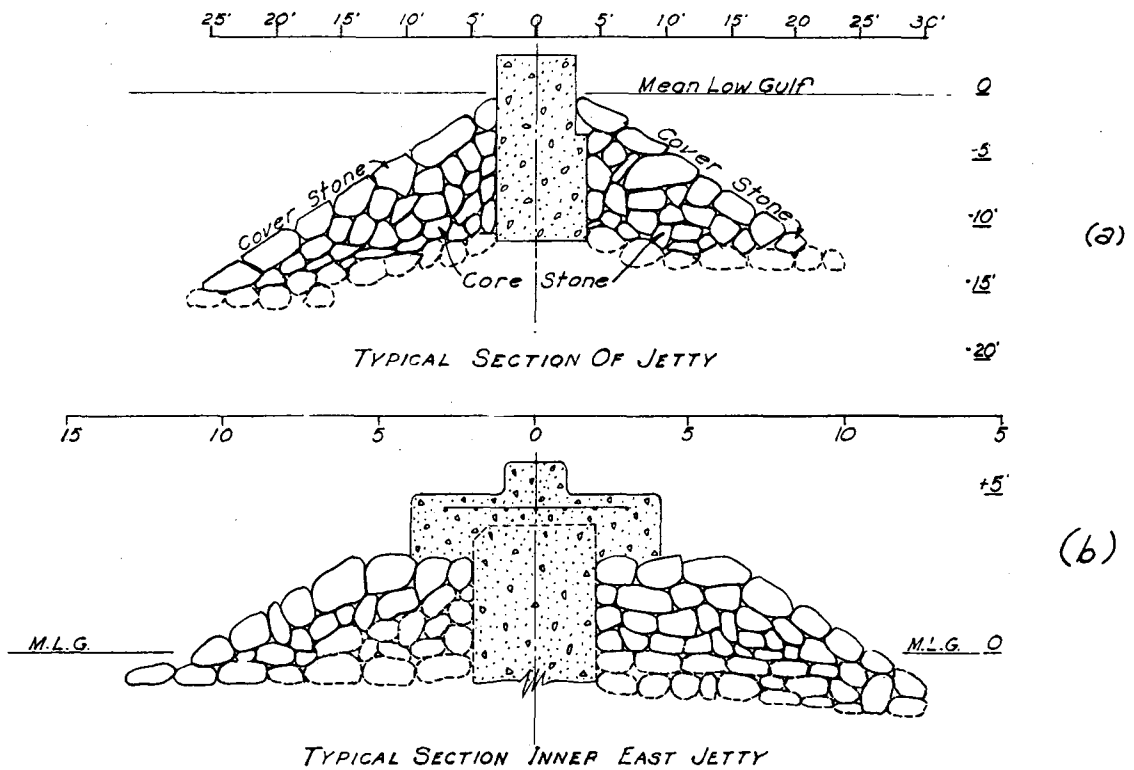


Fig. 4

Entrance to Mississippi River, Southwest Pass and South Pass

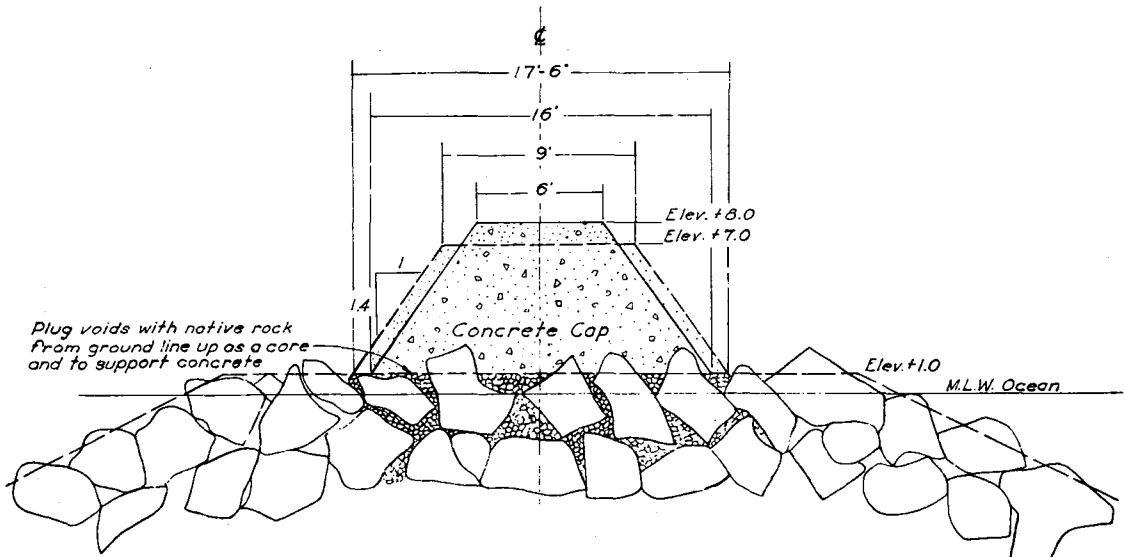


Fig. 5

Typical Section of Concrete Capped Jetty, Lake Worth Inlet, Florida

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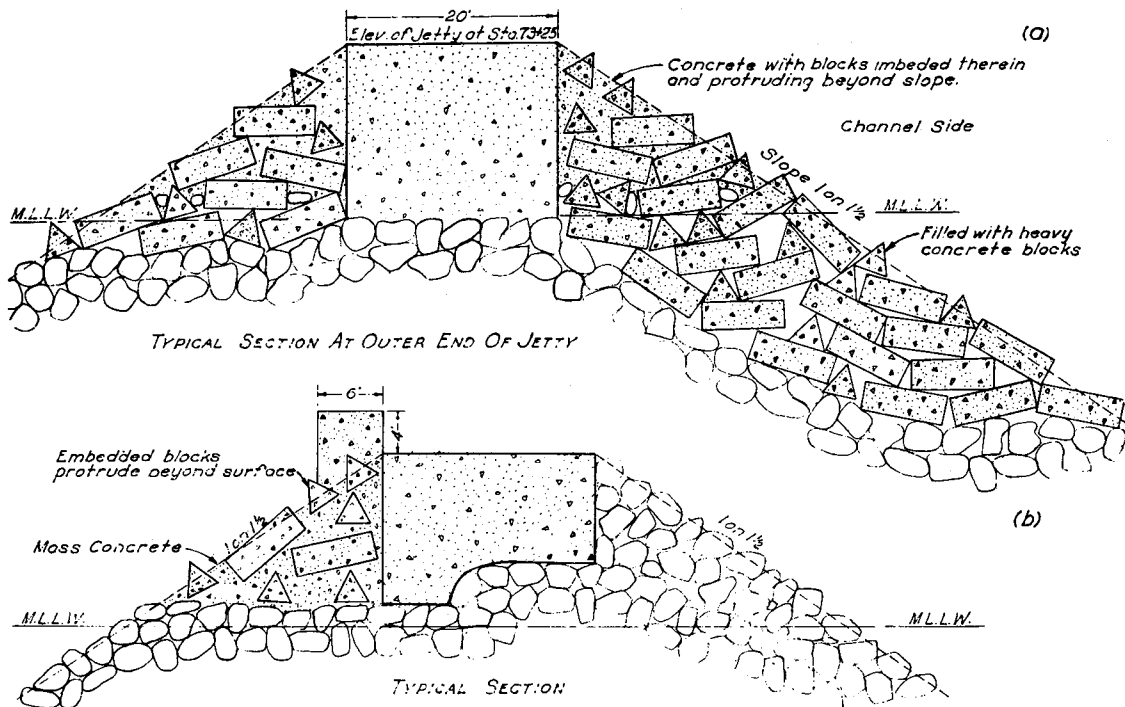


Fig. 6. North Jetty -- Humboldt Harbor and Bay, California

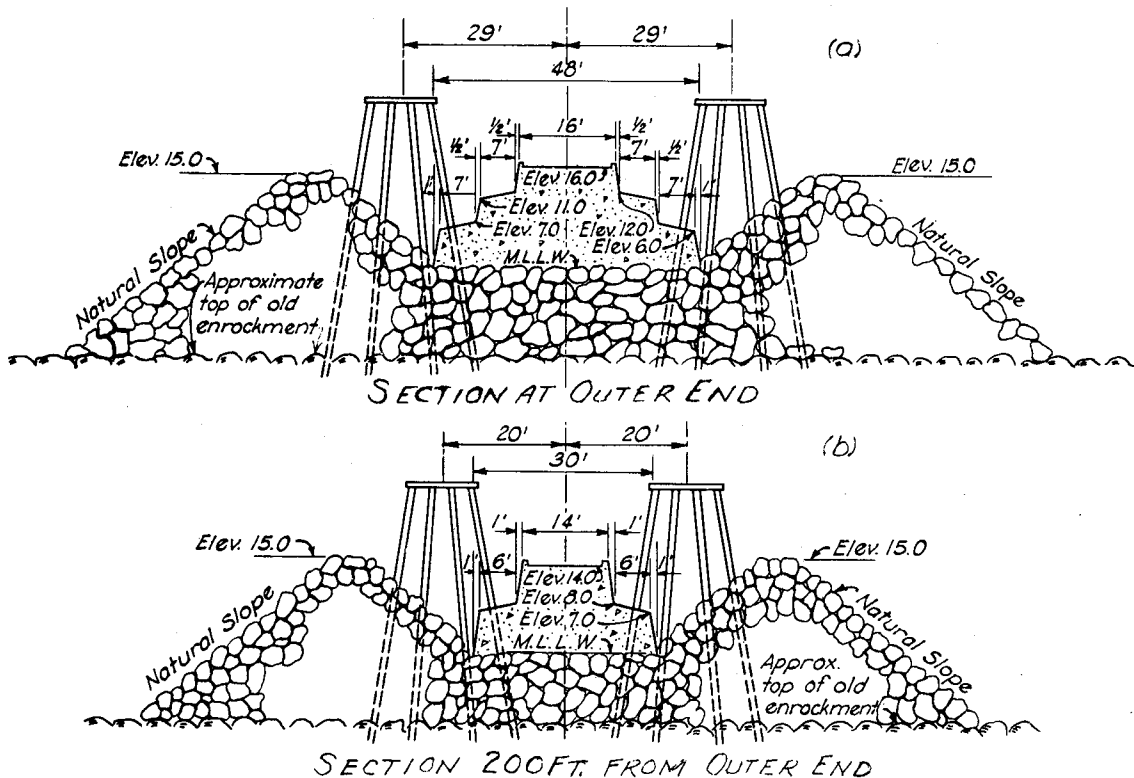


Fig. 7. North Jetty -- Umpqua River, Oregon

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of the foundation rubble is dressed with crushed stone and leveled by a diver before the caisson is placed. Periods of calm water are necessary to float the caisson into position and sink on the foundation. If properly designed and placed, caissons are satisfactory. Cross-sections of concrete caisson structures are shown in Fig. 8.

(4) Sheet pile types include timber, concrete, and steel sheet pile structures. Timber is not suitable where marine borers can exist. Use of concrete piling is restricted by driving limitations. Steel sheet piling is used in several types of structures such as a single row of piling, with or without buttresses; two parallel rows with cross walls, and the cells thus formed filled with suitable material; and cellular steel sheet pile structures. The cellular type structure is widely used for breakwaters in the Great Lakes area. The life expectancy of steel piling depends upon water conditions at site. Steel piling may be used on any foundation where piling can be driven, permits rapid construction, but is subject to damage by sudden and unexpected storms during construction. Details and sections of steel sheet pile structures are shown in Figs. 9 to 11.

(5) Crib types are built of timber, and some of the compartments are floored. The cribs are floated into position, settled upon a prepared foundation by loading the floored compartments, after which all compartments are filled with stone. The structure is then capped with a timber superstructure which is usually replaced by concrete when the timber decays. Stone-filled cribs can withstand considerable

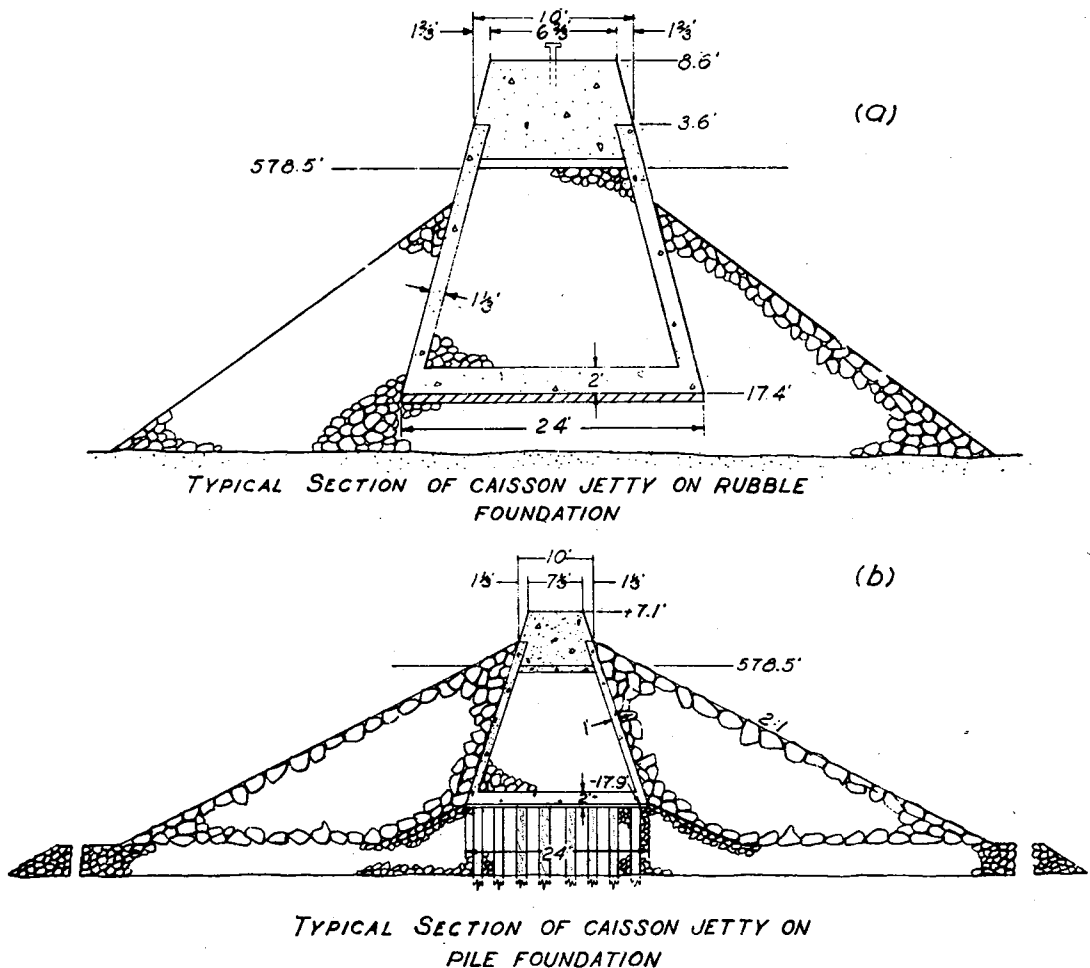


Fig. 8

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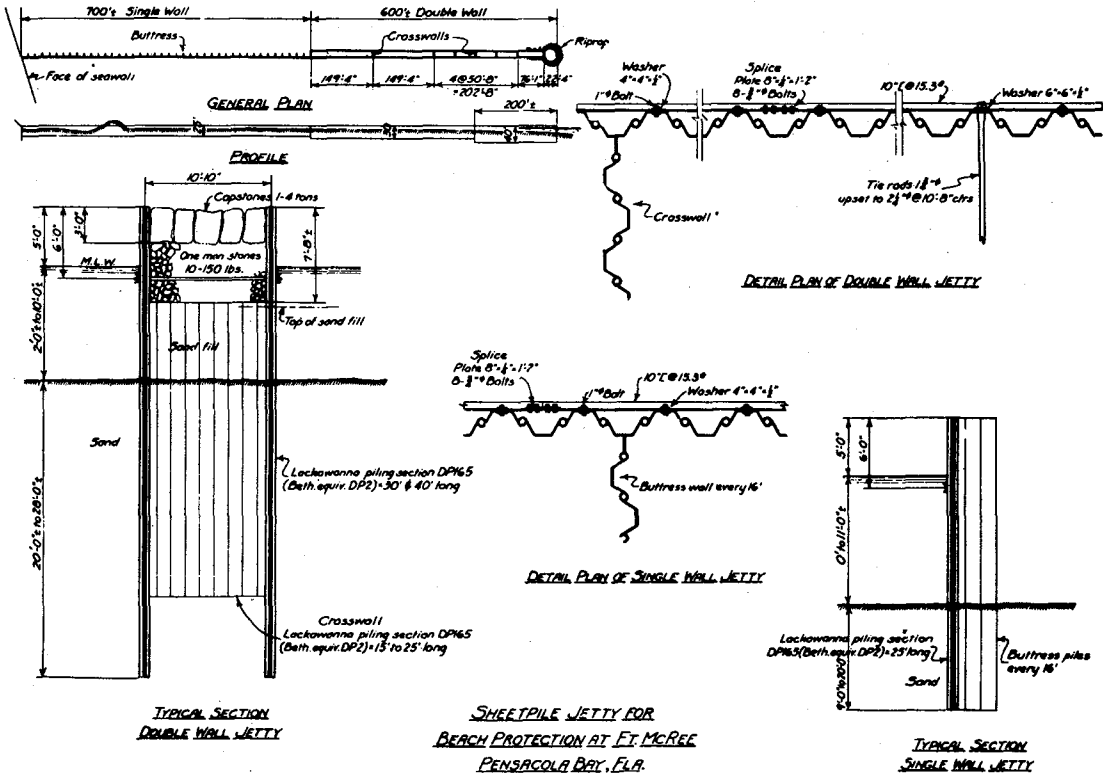


Fig. 9. Sheetpile Jetty for Beach Protection at Ft. McRee, Pensacola Bay, Fla.

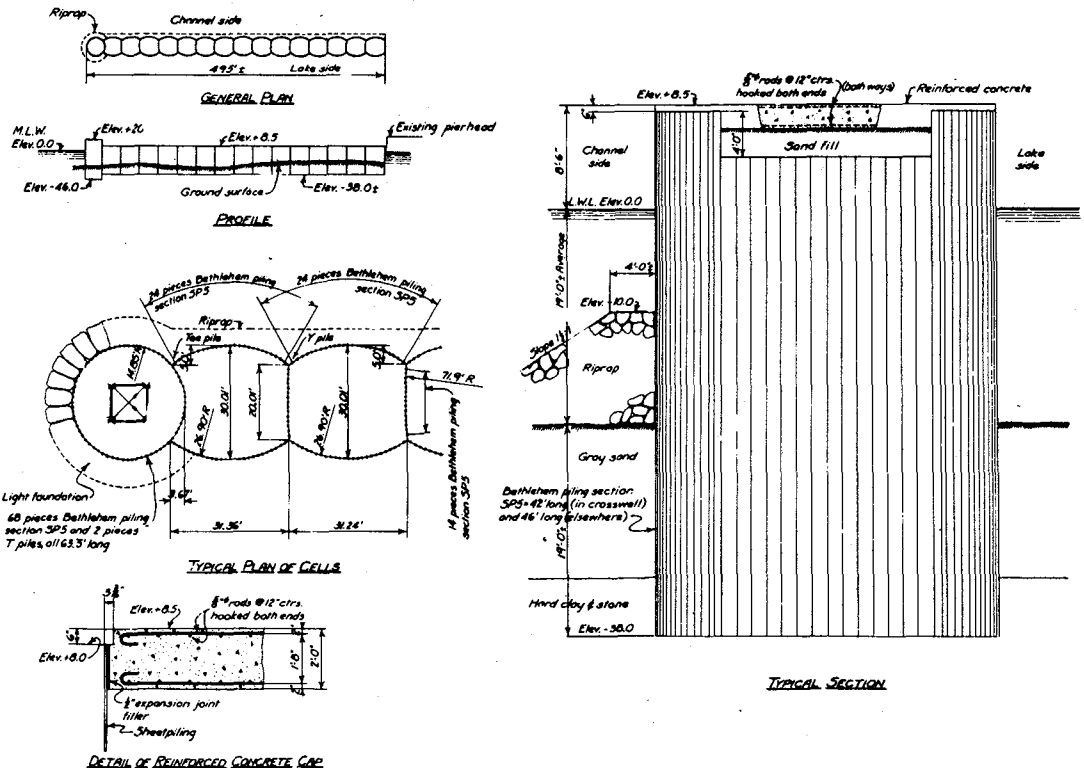
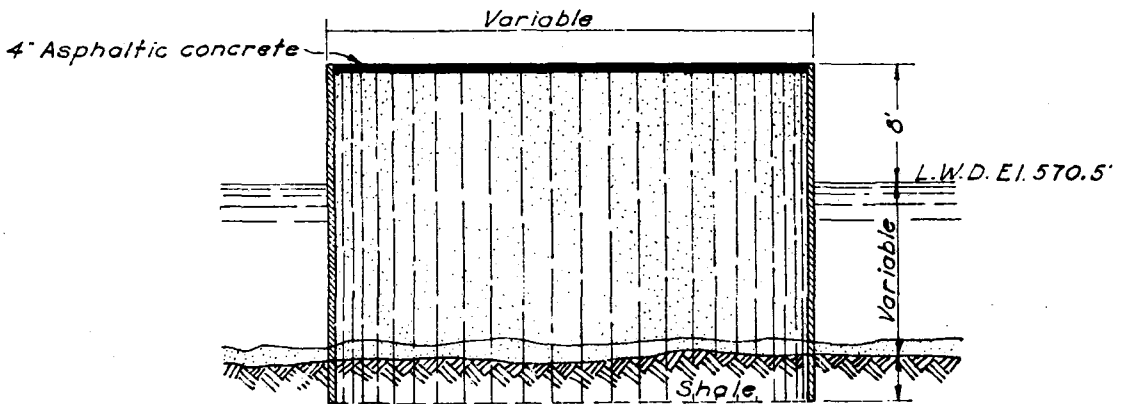


Fig. 10. Details of Steel Sheet Pile Jetty

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TYPICAL SECTION A-A

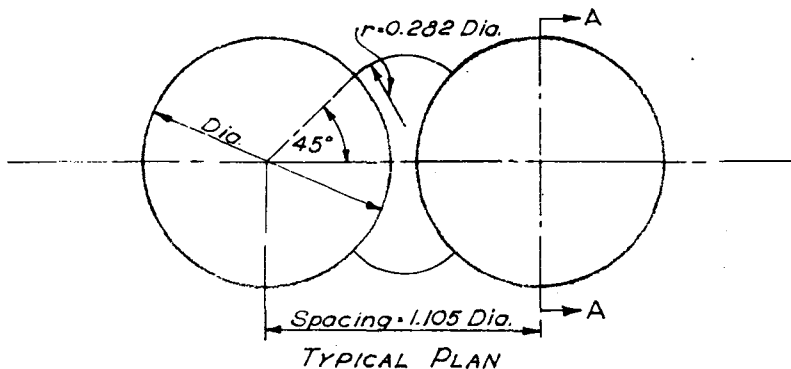


Fig. 11. Circular Type of Cellular Jetty

settlement and racking without rupture. Such structures are suitable for depths up to 50 ft. or more. Foundations are the same as for caissons but do not require such careful dressing. Settlement of foundation will bleed stone from the unfloored compartments but arching of stone over a cavity may prevent detection of stone loss by visual inspection. Timber structures are not suitable for salt water where marine borers may occur. In fresh water, timber-crib structures give long and satisfactory service.

(6) Solid-fill jetties are sometimes required to stop sand movement as well as direct currents. A core of well-graded stone, having a minimum of voids, with a cover of larger stone and an armour of heavy rubblestone is a common type. Caisson and sheet-pile structures are two other types of solid-fill structures.

(7) Asphaltic materials have been used to fill the voids of rubblestone structures above the low-water line. The record of such structures is not impressive (see Fig. 12).

FACTORS AFFECTING DESIGN

The physical characteristics of the site must be determined by hydrographic and topographic surveys and sub-surface investigations. Surveys should extend up and down the coast to provide a complete record of conditions before construction, for future needs of comparison. Usually the fundamental problem is to create and maintain a satisfactory channel across the bar. Two jetties are necessary except under very unusual conditions. In determining the area of section between the jetties, requirements of navigation as well as tidal and stream flow must be con-

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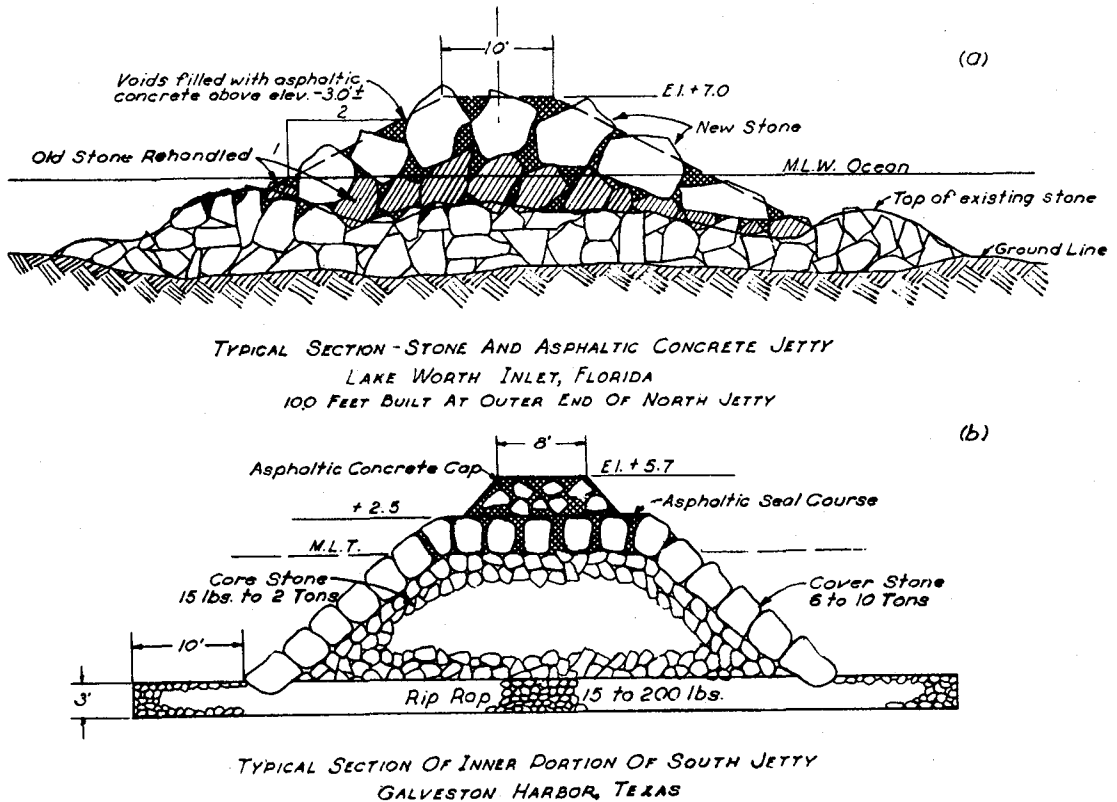


Fig. 12

considered. On small streams or bays, navigation may require channels considerably in excess of the ideal hydraulic section. The natural movement of sand along the coast requires thorough study, as do wave forces developed at the site and their direction of approach. These subjects have been discussed at this conference and will not receive further consideration in this paper. Model studies of ocean entrances have not consistently given reliable results and, while the technic of making such studies is improving, their results must be accepted with caution. Selection of the exact site for the jetties is a big problem, often not given sufficient study. The cost of construction should not be the sole criterion of jetty location. Local sources of materials is a factor in design, often a controlling one.

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Design of a jettied entrance resolves itself into two problems; namely, first the fundamental one of siting the structures so as to create and maintain a satisfactory channel through the entrance, and second, to resist the forces created by ocean storms with a minimum amount of maintenance. Both problems involve many factors difficult to evaluate. As a preliminary step in making a layout for jetties at the entrance to any harbor, a careful study with field observations should be made of direction of tidal flow, both flood and ebb. On the north Pacific Coast where there is a wide diurnal inequality in the tidal range, the ebb is the stronger and predominant influence. The direction of storm waves should also be considered. In general, works should be laid out to conform with the main ebb current, rather than to oppose this, with a view perhaps to reducing wave action in the channel.

Several attempts have been made to determine the relation between the tidal prism and the sectional area of the entrance channel for natural maintenance of

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the channel without excessive currents. In a study of the relation of tidal prisms to channel areas of waterways on the Pacific Coast O'Brien (1931) proposed the formula:

$$A = 1000 \bar{V}^{0.85} \quad (1)$$

where: $\bar{V}$ = volume of the tidal prism between MLLW and MHHW in square mile-feet.

A = area of the entrance channel section below mid-tide in square feet.

A study of the Sacramento, San Joaquin, and Kern Rivers in California (U.S. Congress, 1934) yielded a ratio of 1.08 square feet of sectional area per acre-foot of mean tidal prism for unrestricted estuaries and a value of 0.82 for restricted estuaries. Other values have been suggested for the ratios between the sectional area of discharge channel and acre-foot of tidal prism, but no one value is generally applicable to all entrances and results are often disappointing. As pointed out by M. A. Mason in a report of the Committee on Tidal Hydraulics (1950), "Study of the jettied entrances of the United States shows that the predicted results of the improvement as to entrance channel conditions were actually achieved in very few cases, either with respect to concentration of flow or protection."

Where a natural section can be found near the entrance, as is generally the case, more dependence should be placed in this than in theoretical or computed sections. The section produced by nature has been developed and maintained throughout the years and represents the effect of a summation of all the various influences, both favorable and unfavorable, which are at work in the vicinity. The main ebb velocity needed for the maintenance will be found to be much higher than is required for channel maintenance in non-tidal waters.

Navigation requires the channel to be of sufficient dimensions, proper direction, and to be easy to enter and follow. Vessels should not be required to move along a weather shore when entering or leaving the entrance, and they should not have a beam sea in a narrow channel. These requirements fix the direction of the channel in the quadrant of the prevailing heavy storms. The effect upon adjacent beaches must be carefully studied and, if possible, the final design should include features to prevent deterioration of the adjacent beaches. Artificial feeding of the down drift beach may be necessary. If the channel axis is approximately perpendicular to waves from the heaviest storms (may differ from wind direction), the tendency of the sand to be driven into the channel will be minimized. Severe storms from different directions will require a compromise. Papers on the natural and artificial movement of sediment have been presented in Part 3 of these proceedings and the subject will not be covered in this paper.

Since cost of transporting materials to the site is generally a large percentage of the cost of the work, a thorough search of the locality should be made for materials suitable to use, such as natural rock of sufficient hardness and strength to resist wave action.

Wave action is the most important source of the forces which a jetty must resist. The fundamentals of wave theory and the development of basic design data have been covered in Part 1 of these proceedings and further treatment of those subjects is considered unnecessary. With wave characteristics determined, the type of jetty and the depth of water at the structure both enter into determining the wave force applied against the structure. If the water is deep enough for waves to reach a vertical surface, approximately perpendicular to their line of travel, before breaking, a wave will rise against the vertical surface to about twice its normal height without breaking and be reflected back against the following wave with an apparent cessation of horizontal movement. The result is a standing wave which oscillates vertically against the surface with the same time period as the open-water wave but about double its normal height. This phenomenon of oscillating waves is known by the term "clapotis." At the International Navigation Congress held in Cairo in 1926, the term "standing wave" was adopted for the French term "clapotis" but since the term "standing wave" is employed to describe the wave of the hydraulic jump in stilling basins in dam design the term "clapotis" is generally used. Reference is made to Chapters 22, 23, and 24 for a discussion of the different methods of determining the forces of an oscillating wave.

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In the United States jetties are seldom constructed with vertical walls extending to depths sufficient to avoid breaking waves. It is common practice in this country to construct rubble-mound jetties and most of those which have an upper section with a vertical or stepped face are of the stone and concrete type having a rubble mound extending from sea bottom to about mean low water with the concrete section above that elevation. Such jetties have generally given satisfactory service, especially if the rubble mound is able to resist the wave action. Occasionally, the rubble mound may need some repairs but if such repairs are made when necessary, the concrete super-structure will generally not be injured.

Most jetties are subject to breaking waves in some portion of their length. This may not be the case when the jetty is first constructed, but due possibly to the upsetting of natural forces, changes in the bottom configuration may produce conditions which will cause breaking waves. The extreme shock pressure which may result from a wave breaking against a vertical face may be avoided by a stepped face which breaks up the wave by absorbing the wave energy in a series of lesser impacts which are not simultaneous. A sloping wall may pass the wave over the super-structure and cause injury to the channel slope of rubble.

Rubble-mound types of marine structures have been constructed for thousands of years but there has been a dearth of engineering literature dealing with the design of such structures. Design of rubble marine structures has, in general, been based upon the behavior of existing structures and the experience of the designing engineer. European engineers have favored flat side slopes while American engineers have consistently used natural slopes ranging from 1 on 1.1 to 1 on 1.5. No attempt was made to theoretically determine the size of stone or the slope to be used until de Castro (1933) published the following formula to determine the weight of individual stones required for stability on the side slope of the structure considering the design wave.

$$W = \frac{704 H^3 s}{(\cot\phi + 1)^2 \sqrt{\cot\phi - 2/s} (s - 1)^3} \quad (2)$$

in which:

W = weight of individual stones, in kilograms

H = wave height, in meters

s = specific gravity of the stone

ϕ = angle of side slope with the horizontal.

de Castro's formula is based upon the following theoretical considerations and approximations:

- (a) The destructive action of a wave is proportional to its energy and assuming as a rough approximation, that storm waves heights are proportional to their length, uses H^3 as the value of the wave energy.
- (b) The weight of the stone necessary to withstand the wave energy varies directly as the density in air, and inversely as the cubes of their density submerged in water or $\frac{s}{(s-1)^3}$.
- (c) The stability of a stone subject to wave action is inversely proportional to some geometric function of the slope upon which it rests.

Subsequent to de Castro's work Iribarren (1949) published the following formula:

$$W = \frac{k_1 H^3 s}{(\cos\phi - \sin\phi)^3 (s - 1)^3} \quad (3)$$

in which:

k_1 = 15 for natural rock-fill structures, and

k_2 = 19 for artificial block structures

and all other symbols are as used in de Castro's formula (equation 2). Within the past few years there has been considerable discussion among coastal engineers regarding the merits of the Iribarren formula.

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Presenting the formula in terms of units of measurement used in the United States, it becomes:

$$W_1 = \frac{k'_1 H^3 s}{(\cos\phi - \sin\phi)^3 (s - 1)^3} \quad (4)$$

in which:

W_1 = weight of individual stone, in tons of 2,000 pounds

k'_1 = 0.000468 for natural stones

k'_1 = 0.000593 for artificial blocks

H = design wave height, in feet

s = specific gravity of the stone or blocks

ϕ = angle of the slope with the horizontal

In recent years Mathews (1948) of the Los Angeles District, Corps of Engineers, submitted for discussion, the following formula:

$$W_1 = \frac{6 w H^2 T}{(w - 64)^3 (\cos\phi - 0.75 \sin\phi)^2} \quad (5)$$

in which:

T = wave period in seconds

w = unit weight of the stone in lbs. per cu. ft., and

all other symbols are as given for equation 4.

From observations on hydraulic placer mine operations Rodolf, co-author of this article, had concluded that the stability of a stone subjected to the action of a stream of water from an hydraulic "giant" was approximately inversely proportional to some power of a function of the slope that had been generally accepted for determination of earth pressure back of a wall, namely $\tan(45^\circ - \phi/2)$, and applying the results of his mining-experience, he submitted the following formula:

$$W_1 = \frac{H^2 T s}{600 \tan^3 (45^\circ - \phi/2) (s - 1)^3} \quad (6)$$

This formula is intended to contain a small factor of safety to provide for the occasional individual wave which is higher than the highest wave.

The formulas of de Castro (1933), Iribarren (1949), Mathews (1948), and Rodolf are of a common type but show a considerable variation of results. As a means of verifying his coefficients Iribarren (1949) compared slopes determined by his formulas with a known example (Iribarren and Nogales y Olano, 1950). There follows a description of data used for this comparison:

"Unfortunately those necessary details of each particular case, and especially the damages experienced, are difficult to obtain. Therefore, in this study, we are going to make special mention of one of singular interest.

We refer to the interesting compilation on the part of Argel concerning weights of stones or blocks and their corresponding stable slopes at various depths, after repairs of numerous damages to deficient slopes. This outstanding compilation was made by Messrs. J. Larras and H. Colin in their article of December 1947 published in the periodical Travaux (see references at end of chapter).

From page 609 of that compilation are obtained the following data, showing the corresponding batters, depths, and weights of blocks or stones.

Materials	Minimum weight (Metric tons)	Maximum batter	Minimum Depth, (meters)
Artificial blocks	50	5/4	- 5
Natural stones	4	3/2	- 11
" "	1	3/2	- 14
Quarry waste	-	2/1	- 18

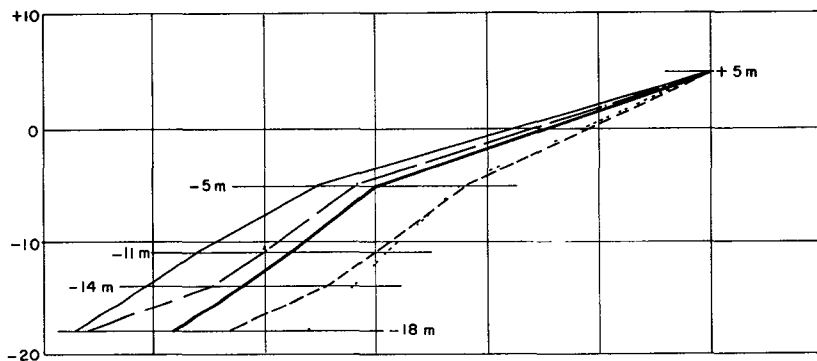
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There can also be adopted, as a stable upper surface batter, that of 3/1 formed by 50-ton concrete blocks, adopted for strengthening the North dike which, according to the cited article, has withstood perfectly even the worst storm (3 February 1934) ever suffered by the port of Argel and which destroyed a large part of the Mustafa dike. Likewise, we can adopt the toe depth of 35 meters, which this North dike reaches."

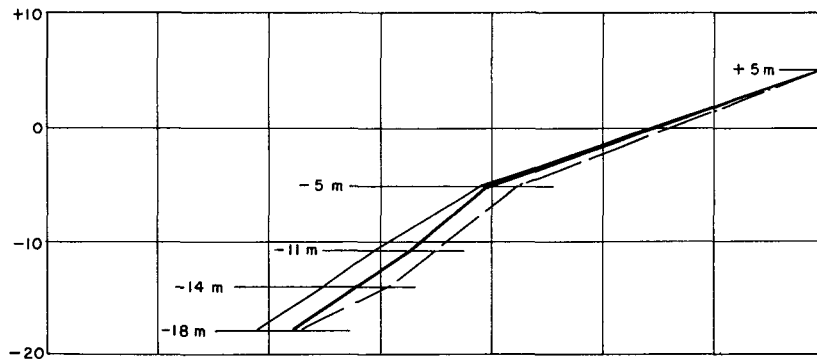
To measure the accuracy of the other three formulas given above, slopes for the conditions used by Iribarren were computed by the formulas of de Castro (1933), Mathews (1948) and Rodolf and in the comparison of the results shown in Table 1, the profiles determined by each formula are arranged in the order of their accuracy as determined by comparison with measured profile (Fig. 13).

TABLE 1
COMPARISON OF CALCULATED AND MEASURED BREAKWATER SLOPES

Measured slope	Depths	Rodolf	Iribarren	Mathews	de Castro
1 on 3	+5 to -5	1 on 3.18	1 on 3.51	1 on 2.19	1 on 2.22
1 on 1.25	-11	1 on 1.37	1 on 1.83	1 on 1.32	1 on 1.18
1 on 1.5	-14	1 on 1.55	1 on 1.53	1 on 1.44	1 on 1.02
1 on 1.5	-18	1 on 2.85	1 on 1.67	1 on 2.11	1 on 1.07



a SURFACE WAVE HEIGHT 9.7 METERS



b SURFACE WAVE HEIGHT 9.05 METERS

——— IIRIBARREN de CASTRO
 - - - RODOLF ——— ACTUAL
 - - - - - MATHEW

Fig. 13
Comparison of computed slopes with actual slope of rock dike at Argel, using formulas proposed by de Castro, Iribarren, Mathews, and Rodolf

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The height of wave breaking on the dike had been reported as 9 meters, but Iribarren deduces that the wave was actually 9.7 meters and this height was used in computing the slopes shown above. Then, making new assumptions, a height of 9.05 meters is computed by Iribarren and he recomputes the slopes. The following tabulation shows the results from the formulas of both Iribarren and Rodolf.

TABLE 2
COMPARISON OF IRIBARREN AND RODOLF FORMULAS

Measured slope	Depth	Iribarren	Rodolf
1 on 3	+5 to -5	1 on 3.04	1 on 2.75
1 on 1.25	-11	1 on 1.68	1 on 1.21
1 on 1.5	-14	1 on 1.43	1 on 1.29
1 on 1.5	-18	1 on 1.51	1 on 2.04

From the above comparison it appears that both Iribarren's and Rodolf's formulas closely approximate the real profile and the following quotation from Iribarren can be applied to both formulas:

"In this way we obtain a theoretical profile, shown in Fig. 3', similar to 3, even more closely approximating the real profile; but the really interesting fact is that both figures, whose calculated wave heights differ by less than 8 percent, a degree of approximation that we consider difficult for anything practical to really exceed, are now authentically confirmed by the very important direct observation from this interesting compilation. It is also now confirmed, undoubtedly, through authentic direct observation, despite the simplifications one is forced to introduce into the complex subjects of maritime engineering that the degree of approximation really obtained is superior to that of many calculations of engineering on terrestrial subjects, in which, even legally, are imposed large safety factors, generally greater than two and frequently approximating three."

A formula, similar to the above formulas is deduced by Epstein and Tyrell (1949). Tests now being conducted by the U.S. Waterways Experiment Station will investigate experimentally rubble-mound breakwaters. The formulas should be checked by the experimental determinations.

The following table gives the angle and value of the term $(\cos\phi - \sin\phi)^3$ for different slopes.

TABLE 3

Slope	Angle	$(\cos\phi - \sin\phi)^3$	Slope	Angle	$(\cos\phi - \sin\phi)^3$
1 on 1.5	33°41'	0.0214	1 on 3	18°26'	0.2530
1 on 1.75	29°45'	0.0515	1 on 3.25	17°06'	0.2898
1 on 2	26°34'	0.0894	1 on 3.5	15°57'	0.3238
1 on 2.25	24°58'	0.1308	1 on 4	14°02'	0.3853
1 on 2.5	21°48'	0.1729	1 on 4.5	12°32'	0.4375
1 on 2.75	19°59'	0.2139	1 on 5	11°19'	0.4825

There are several factors affecting the permanence or stability of rubble-mound structures which are not taken into account in the above formulas. Among these may be mentioned:

- ① 1. The angle or direction at which seas strike the breakwater. This vitally affects the slope presented to the sea.
2. The height of the structure.
3. The grading of stone sizes.
4. The manner of placing.
5. Shape of stones and texture or composition.

At the end of a jetty or breakwater the seas often strike in a direction almost at right angles to the line of the jetty and the result is a running sea parallel to the rock surface and a slope considered in the formulas as zero opposed

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to the force of the sea. In all the formulas proposed this calls for the smaller sizes of stone. As a matter of fact, however, the largest stone is required in such a position. At the end of the south jetty at mouth of Columbia, although built of the heaviest stone available, this cross sea caused a raveling or cutting off of the super-structure enrockment to low-water level, at a rate of about 300 ft. of jetty per annum, until such action was prevented by construction of a heavy concrete superstructure terminal.

Similarly, at the mouth of the Umpqua River, heavy "wing" blocks of concrete (136 tons) were poured on a level-rubble foundation at low-water elevation to protect the footing of a 1,700-ton main block, yet one of the 136-ton blocks 15 ft. x 15 ft. x 8 ft.) was shifted horizontally 20 to 25 ft. during the first storm. The other blocks could not be observed. These experiences show the fallacy of depending on small or moderate size stone on flat slopes if exposed to a sea which can break against or across it, and throws doubt on the use of functions of the slope of the structure as a principal determining consideration. If a breakwater or jetty is not high enough to prevent the seas from crossing over in force, stones will be removed from the crest (zero slope) and the back side of the structure also will be attacked. The back slopes of concrete capped structures of moderate height, are often attacked by such over wash.

It is, accordingly, evident that while a reliable formula for size of stone in breakwaters and jetties is most desirable, there are so many influencing factors and construction limitations involved, which cannot well be taken into account in a formula, that the problem usually resolves itself into the more practical considerations of the materials available at reasonable cost and the methods which can be used for construction; always bearing in mind that large stone of high specific gravity should be used, if available, for highly exposed structures. The larger the better.

In addition to currents caused by waves, littoral or alongshore currents must be considered when designing for protection against scour. Where river currents are present, special precautions may be necessary to guard against excessive scour along the toe and at the end of the jetty. If the proposed structure will obstruct or change the direction of river currents, scour at and around the end during construction may and probably will occur. Where original depth on the line of the structure is not sufficient to furnish protection to the base, the scour by currents around the end of the structure during construction may not be objectionable, as material will be removed without cost and thus permit the base of the structure to be placed at a depth assuring better protection, but will necessitate the use of more rubble than indicated by the original profile. Cases on record show that as a jetty was constructed outward from the land connection, scour around the end caused depths nearly twice those shown on the original profile, and necessitated the use of two or three times as much rubble stone as the original profile indicated. Possibilities of such scour should be investigated in determining the amount of rubble for the foundation. If advisable to control the scour, a mat of stone spread over the area in advance of jetty construction, or a rubble apron extending a long distance ahead of the main enrockment, are possibilities for controlling or preventing the scour. It cannot be stated with certainty that such scour can always be avoided.

Analysis and design of a jetty structure follow the usual methods and procedures for design of any structure. Resistance to overturning, safety against sliding, and maximum pressure against the foundation are all investigated and designs are developed to satisfy the requirements as for a land structure, such as a dam or retaining wall. It is assumed that the reader is familiar with the usual design procedure and further discussion of the details of design in this paper is not considered necessary. The factors that must be considered in design of a jetty, which do not enter into design of a land structure, are: siting of the jetty to accomplish desired results yet provide for safe navigation, height of structure to provide protection to the sheltered area, avoidance of a location where the waves will build up, and the angle at which the waves strike the structure.

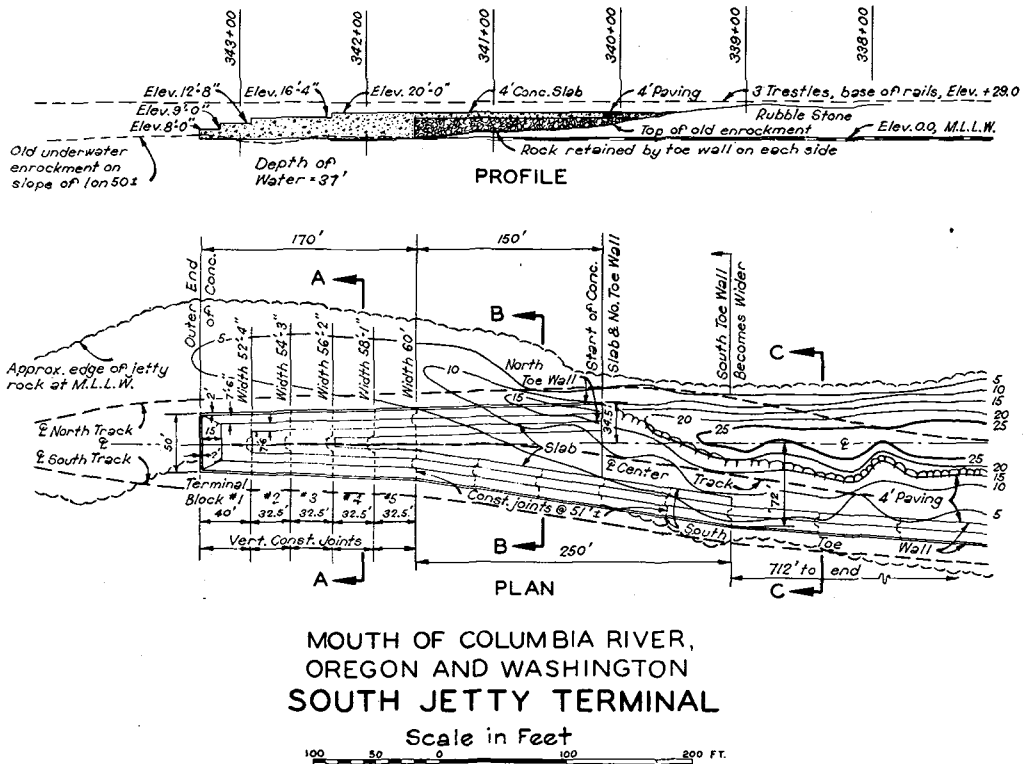
COASTAL ENGINEERING

AIDS TO NAVIGATION

The District Commander of the U.S. Coast Guard should be furnished the following information: (a) advice as to authorization of a project involving construction of a Jetty, (b) the proposed construction schedule, (c) maps showing the final location of the structure. During construction, temporary aids to navigation should be maintained, if necessary. Information as to installation or discontinuance of these aids should be furnished to the Coast Guard so that such information may be included in "Notice to Mariners" published by the Coast Guard. Changes in depths and location of channels should be reported to the Hydrographer, Hydrographic Office, U.S. Navy, Washington, D.C.

CONSTRUCTION OF JETTIES

Railroad trestles are often used for the construction of rubble jetties in areas subject to frequent heavy sea action, where the use of floating plant is impracticable. A single-track trestle will suffice for construction of a jetty having up to about 40 feet top width. The trestle should be several feet above finish grade of jetty to allow free dumping and placing under the trestle. Small rock may be dumped to reinforce the trestle bents. Larger rock is handled on flat cars and may be pushed over the sides by means of a crawler-type shovel, which moves from one flat car to the next, or standard dump cars may be used. The largest rocks used as a final covering, especially on the seaward side, are dumped in the same manner and placed in final position by means of a track crane. For construction by trucks the structure is built to approximate finished section as it progresses into the sea and a roadway is maintained on top of the jetty, or on a trestle roadway. The large stones are placed by crawler-type crane. Sections of a jetty section showing use of single and double trestle and track are shown on Figs. 14 and 15. Fig. 16 shows the detail of a trestle.



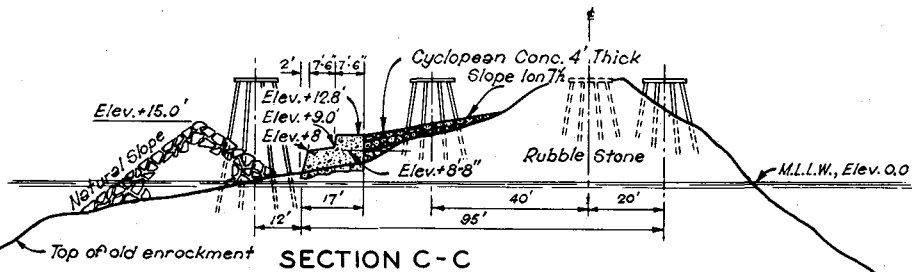
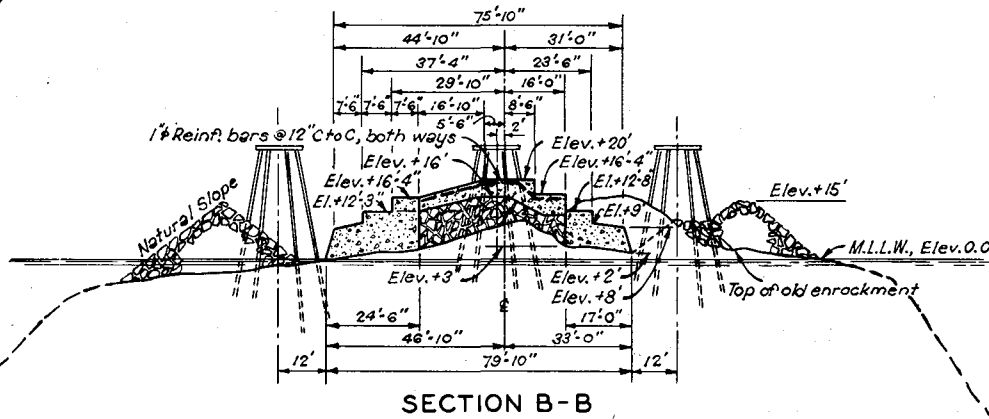
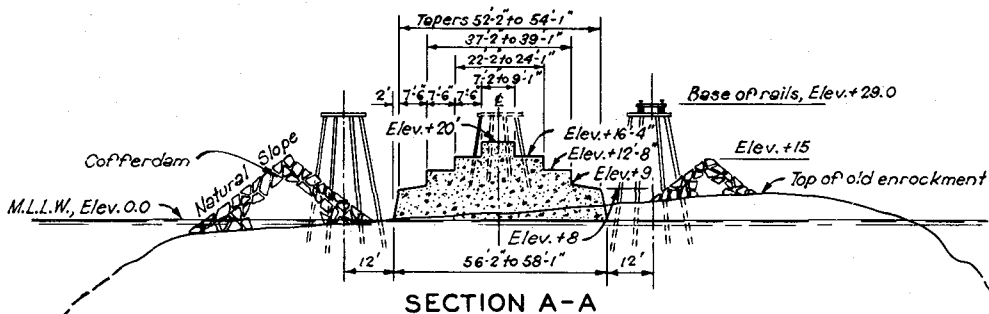
MOUTH OF COLUMBIA RIVER,
OREGON AND WASHINGTON
SOUTH JETTY TERMINAL

Scale in Feet

PORTLAND DISTRICT, CORPS OF ENGINEERS

Fig. 14

DESIGN AND CONSTRUCTION OF JETTIES



MOUTH OF COLUMBIA RIVER,
OREGON AND WASHINGTON
SOUTH JETTY TERMINAL

SCALE 1" = 40'



PORTLAND DISTRICT, CORPS OF ENGINEERS

Fig. 15

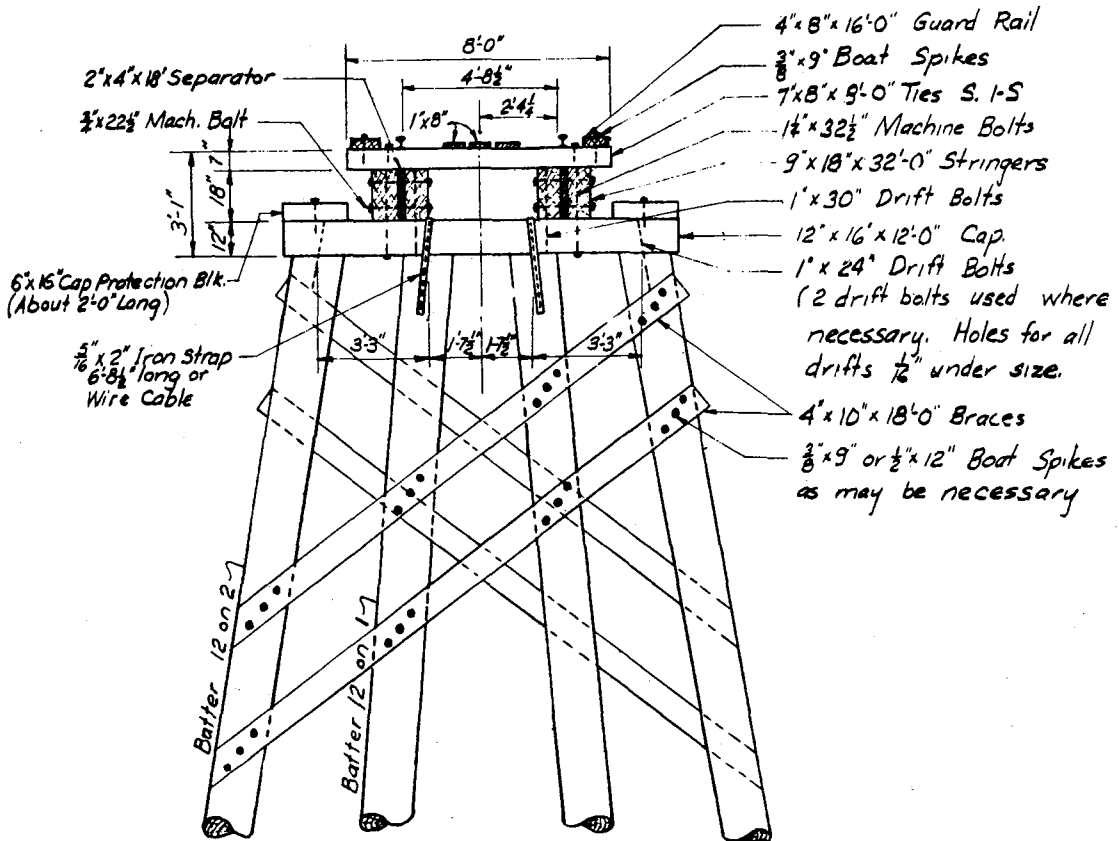


Fig. 16. Detail of Jetty Trestle, South Jetty, Columbia River, Oregon

Floating plant may be used for the construction of a jetty in waters where wave action is not too continuous or where the vessels may work in sheltered water on the lee side of the structure. This method consists of moving the material to the site in vessels and either dumping the material or placing by floating derrick.

Quantities of stone used in jetty construction may be determined by weighing, or by measurement of volume by cross-sectioning and calculation of weight. The former method is preferred. If the structure is built with floating plant the weight of the stone is usually determined by gages placed on the barges to measure the displacement of the vessel when loaded and the weight of the stone is calculated therefrom.

CONCRETING EQUIPMENT

The methods employed in placing the concrete portion of a composite break-water or jetty vary with each individual project, being influenced by such factors as the magnitude of the project, plant readily available, transportation facilities, and weather and sea conditions. Depending upon the design of the structure, concrete must sometimes be poured below the mean water level, thus necessitating the use of extensive protective cofferdam arrangements on a wide stone base. Specifications usually require the continuous pouring of each monolith block to completion. This may call for fast, concentrated effort between tides.

In areas of heavy wave action the attack on the structure is often severe between low water and half-tide level. It is accordingly necessary to base the concrete superstructure at elevation of mean-lower-low-water or lower, if practicable.

DESIGN AND CONSTRUCTION OF JETTIES

The equipment and materials involved in concreting operations should be located conveniently close to the structure site. To accomplish the desired proximity, construction of access roads may be necessary. Stock piles of aggregate material in various gradations, as well as dry storage facilities for cement should be grouped about a central batching plant. On large jobs careful attention should be given to equipment and arrangements for high speed economical operation.

For the most efficient control of operations, the mixer should be reasonably close to the placing area. For small sections employing a single tramway, a side platform supported by piles has been used. For larger sections, with a tramway on both sides, the mixer usually can be supported by a platform between the tramways. On large jobs requiring large daily placement from trestle work it may be found impracticable to provide sufficient storage space at the site and mixed concrete may have to be transported a considerable distance (3-1/2 miles on railway cars at mouth of Columbia). The concrete is transported by dump buggies, trucks, or railway cars, using inclined chutes for placement to avoid a free vertical drop of more than about 5 ft.

In closing this paper it may not be amiss to again quote from Iribarren when he states:

"However it is not logical to apply strict results, obtained by means of the application of theoretical formulas to sea conditions when it is the general rule to employ ample factors of safety for land conditions."

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