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PART 3
COASTAL STRUCTURES AND RELATED PROBLEMS

ENSENADA

CHAPTER 23

RECENT ADVANCES IN COASTAL STRUCTURE DESIGN

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INTRODUCTION

The type and scope of work accomplished and responsibility and authority of the office to which he is attached is indicative of the interests of an author and of the information available to him. It is therefore pertinent to cover in this general section a summary of the responsibility and the delegated authority of the Corps of Engineers as regards Coastal Engineering.

The subject of recent advances in coastal structures is quite broad and complex and modern design practice incorporates old and new findings. This paper discusses various aspects of proper modern design of break- waters and jetties with special attention to newer findings, their proper application and the means by which the findings were made.

The Corps has the responsibility for the planning, investigation, design and construction of Federal civil works navigation projects. This consists generally of harbor and channel works. Their responsibility ex- tends to the control of all works, private or governmental, to assure that navigation will not be adversely affected. They are also charged with the responsibility of planning, investigation, design and construc- tion of Federal civil works projects involving shore protection from wave and currents, protection from effects of hurricane, tsunamis and tidal flooding, and of beach erosion control. The accomplishment of such a mission therefore includes research of an applied nature to permit ad- vancement in knowledge and technique. Most of this research is based on small scale model studies accomplished at the U. S. Army Engineer Waterways Experiment Station at Vicksburg, Mississippi, and at the laboratory of the Corps of Engineers Beach Erosion Board in Washington, D. C. In addition a limited number of prototype studies are being initiated in the charge of the staffs of various District Engineers.

RESEARCH

WATERWAYS EXPERIMENT STATION

Research accomplished at the Waterways Experiment Station is of two general types, Civil Works Investigations (CWI), which are applied research of a general nature, and Project Studies, which are investigations for a specific project. The combined results of these investigations and the analysis have been the subject of many reports from WES and are the basis of much advancement in the design of coastal structures, especially those

constructed of stone. Studies concerned with coastal structures include wave force on breakwaters, stability of rubble-mound breakwaters, orientation of harbor structures, effects of scale on harbor models, scale effect tests of rubble breakwaters and design of rubble wave absorbers. Project studies are many and varied. Especially for the more complex conditions, model studies are made for alignment, evaluation of effect, and stability of structures. Surge problems are also a popular subject for project model studies. These results for specific conditions are, of course, integrated into general test data for the purpose of developing design criteria. The findings of CWI and Project Studies are published as Technical Reports and Miscellaneous Papers, which are distributed throughout the Corps of Engineers and may be purchased at nominal cost by others.

BEACH EROSION BOARD LABORATORY

A wide variety of applied research generally pertaining to shore protection and beach erosion control is accomplished by the Beach Erosion Board staff. The research is of two types, Civil Works Investigation Studies, administered through the staff of Office, Chief of Engineers, and Beach Erosion Development Studies, administered by the Beach Erosion Board These studies are more or less overlapping in subject and include the following subjects, methods of by-passing sand, criteria for artificial beaches, design of shore protection structures, wave refraction and diffraction, uses of radio-active tracers in shoaling studies, and others. The results of the studies are published as Technical Memoranda and are integrated into Technical Report No. 4 at each time of its revision.

PROTOTYPE TESTS

To obtain data generally unobtainable in other ways, substantiate model data, and to give direction to future model programs, a number of prototype tests have been initiated. The locations selected for the presently proposed tests are the harbors at Umpqua River, Oregon; Morro Bay, California; Santa Cruz, California; and Nawiliwili, Maui, Hawaii. These studies have been recently undertaken and no results have yet been obtained. These studies consist generally of the following:

a. An as-built survey of the new or reconstructed structure. This includes marking of individual armor units for comparison with future surveys.

b. The installation of pressure type wave gages generally located in the deeper water to obtain a measurement of the impinging wave before it is significantly altered by structures or on-shore topography. In some cases the gage will be located on concrete blocks setting on the ocean floor and others, located in deeper water, will be placed on steel piles to eliminate loss of gage accuracy due to excessive depth.

c. Visual inspection will be made after each storm of significant amplitude. Where damages are noted in the structure, instrument type survey will be made to determine the quantity of loss of material.

d. An annual summary report will be made to record the results of the tests. It is assumed that these observations will continue over a period of 3-5 years and a completion report will be prepared upon termination of the studies. It is also probable that technical papers, based on these results, will be published in technical society publications.

CONFERENCES

Model tests and theoretical analysis are invaluable to the advancement of design techniques but equally important is the translation of the data into usable construction techniques. For instance, an almost indestructible structure can be constructed of fitted units keyed together or with units of extremely high area of contact. The fitting can be easily accomplished above the water line, but how can such complex work be accomplished beneath the water surface? Design criteria are prescribed by the Office, Chief of Engineers, based generally on past experience. As these new criteria are formulated and methods prescribed, what particular difficulties are the field offices encountering in applying these criteria and methods? To answer such questions as these and to obtain a consensus of opinion throughout the Corps of Engineers, a series of harbor design conferences were held. First, a general conference was held in Washington wherein the broad aspect was considered, and later a series of regional conferences were held to discuss specific problems of the various regions. The present criteria for the design of harbor structures have, in general, been formulated through the consideration of past experience, research accomplished at the Waterways Experiment Station and the Beach Erosion Board and through the opinions expressed in these conferences. In recent years the Corps of Engineers has placed great emphasis on the design of stone structures, both for new work and rehabilitation of existing structures. Therefore, the design portion of this paper is principally concerned with the advancements in technique of design of stone structures.

DESIGN NOTES

STRUCTURE TYPES

Rigid and Semi-rigid Structures - The principal structures of this classification are walls, revetments, breakwaters and jetties constructed of steel, concrete, timber or combinations. The principal coastal engineering features of these designs are the determination of topography, hydrography, still water elevation and wave characteristics with a final result of the specification of the forces to which the structure will be subjected Then, with the possible exception of toe protection, the design procedures are structural in character.

Determination of still water level is made by statistical study of tides or lake levels and the selection of a frequency to be used in design. The design still water level will contain additional elevation due to hurricane surge or wind and wave set-up during storms. This feature has been covered in previous publications or is to be covered by others of this conference.

Determination of design significant wave height has been covered by others and is presented in Beach Erosion Board Technical Report No. 4. However, about 13 percent of the waves in a train are higher than the significant wave and some may severely damage or even cause complete failure of a rigid or semi-rigid structure. Therefore, it is necessary to use a wave of height within the train occurring less frequently than the significant wave. For a rigid structure it is recommended that the 1% train frequency (that is, the maximum height that occurs once in a typical 100 wave train) be used for design wave (significant height x 1.6) For semi-rigid structures a value ranging between the 10% and the 1% train frequency, based on the effect of failure on the project, is generally used. General concensus is that the 10% train frequency (significant wave x 1.1) should be used for steel sheet pile cell design, as such a structure can absorb considerable racking. If the depth of water at the structure is less than required to support the wave that is based on generation potential, depth criteria should be used to determine the height of the design wave.

Upon determination of design still water level and design wave height forces are computed based on the character os the wave upon reaching the structure.

a. For breaking waves the theory of Minikin is best for computing wave force. Studies have indicated that force is maximum when the wave approach is normal to the centerline of the structure. The effect on force of waves approaching a structure at an angle has not been adequately investigated, but investigation of this aspect is planned for the near future. Meanwhile, it is general practice to reduce computed pressures according to $\sin^2 B$, where B is the angle between the approach direction of the wave and the normal to the structure alignment. This is an unverified method and should be used with caution.

b. For unbroken waves the theory of Sainflou is best for use. For waves approaching at an angle to the structure, pressure is often reduced by straight line extrapolation between full clapotis and the design wave height, depending on the angle of approach.

c. With present knowledge, computing pressure due to broken waves may be more complex than for the other two wave types. If a wave breaks near enough to a structure some of the energy is transmitted to it. This pressure would be related to the incident velocity and the run-up on the structure. In most cases the maximum force would be that caused by the wave that would break at the depth in which the structure is located rather than a larger wave breaking seaward.

Flexible Structures - Flexible structures are those composed of stone or concrete components. Design of flexible coastal structures are based on empirical data as it is impossible to determine accurately by analytical methods the effect of the interplay of forces on the individual units. Upon wave attack a large area of the cover layer can be displaced down the slope en masse or individual units may be lifted and rolled up the

slope or rolled down the slope. Short period wind waves impinging upon
a rubble-mound structure may break completely, projecting a jet of water
approximately perpendicular to the slope, or they may break partially
with a poorly defined jet, or they may establish an oscillatory motion
of water particles along the structure front similar to the motion of a
clapotis at a vertical wall. Because of the empirical nature of the
basis of design, and the amount of investigation made in recent years,
the discussion of rubble-mound structures is made in a separate section.

DESIGN OF RUBBLE-MOUND

Still Water Level and Design Wave - The determination of elevation
of still water level and height of the design wave is computed as pre-
viously described. The equation used in the design of rubble-mound struc-
tures has been devised and evaluated on the basis of using a design wave
equal to the significant wave height. There has been many discussions on
the desirability of using a statistical value higher than significant wave
height for use in the Hudson equation. However, due to the flexibility of
the structure and to the fact that design methods are empirical in nature,
the affect of using a rarer frequency is not known. Since the significant
wave height is an average of the highest one-third of the waves in the
train, about 13% of the waves in the train should be higher than the signif
icant wave. It is assumed that the rare occurrence of these higher waves
will not by themselves extricate components from the structure, that many
of the units that will be slightly moved will be reinterlocked by the
pounding of the smaller waves and that it is more economical to repair
slight damage than to design the structure for absolute stability. The
using of a slightly higher wave results in a much higher stone size re-
quirement since wave height is cubed in the numerator of the equation.

Height and Crest Width - The height of the structure depends primarily
upon the degree of protection that must be furnished, that is, the amount
of overtopping that might be tolerated. The amount of overtopping is a
function of the wave run-up and the width of crest. The structure crest
width depends on the size of stones in the cover layer and should be wide
enough to provide the necessary roadway if the structure is to be initially
constructed or to be maintained from the top. In cases of relatively sever
wave action, and especially if overtopping is to occur, the structure crest
width should be wide enough to allow for three stones.

Cover Layer Stability - The Hudson or WES equation is generally used
throughout the United States for determining the size of units and the
corresponding slopes required for stability when subjected to waves. This
equation is:

$$W_r = \frac{\gamma_r H^3}{K_\Delta (S_r-1)^3 \cot \alpha}$$ (1

where,

W_r = weight of the individual units in pounds.

γ_r = the specific weight of the component material.

K_Δ = experimentally determined coefficient.

S_r = specific gravity of the stone or cover unit material relative to the fluid in which it is immersed.

α = the angle of structure face.

Most coastal engineers are familiar with the application of this equation. Therefore, these remarks will be in reference to the evaluation of K_Δ and indications from recent model studies.

Based principally upon model studies using nonbreaking waves but with some amount of data from project studies using breaking waves, the K_Δ values shown in Table 1 are recommended for use as averaged values. These values should be modified depending upon the shape and roughness of the stones, the method of placement, the frequency of occurrence of the design wave, and the economics of frequent repair. The cover layers composed of units of the weight determined by this equation should be arranged as follows: (a) On structures located in relatively deep water and of sufficient height to eliminate overtopping, the cover stone should extend across the crest and down the seaward slope to a point one wave height below lowest still water. Below this elevation the stone may be reduced to one-half the size. The stone on the inside slope should be based on the size waves that can be generated within the harbor. (b) For structures in shallow water and no overtopping, the cover stone should extend across the crest and down the seaward slope to the bottom. (c) When overtopping is to occur, the large cover stone should extend as before on the seaward side as well as to at least still water elevation on the harbor side.

There are many who believe that for most conditions graded riprap provides the most economical and effective covering for a flexible structur While this method may be most economical for small structures, as the entir quarry production could probably be used, it is not effective for protectio from larger waves. Studies made for the design of riprap cover layers for railroad relocation fills at Ice Harbor and John Day Lock and Dam projects indicated riprap cover layers were not economical for wave heights greater than about 6 feet.

This conclusion was based on the following equation developed by Hudson for riprap sizes.

$$W_{50} = \frac{\gamma_r \, H_D^3}{K_{RR} \, (S_r-1)^3 \, \text{Cot} \, \alpha} \qquad (2)$$

and,

$$W_{max} = 3.6 \ (W_{50}) \tag{3}$$

and,

$$W_{min} = 0.2 \ (W_{50}) \tag{4}$$

where,

W_{50} = 50 percent size by weight of cover material, in pounds,

H_D = Design wave height for selected damage criterion, in feet.

K_{RR} = Experimental coefficient for riprap.

W_{max} = Weight of maximum size of riprap in the gradation, in pounds.

W_{min} = Weight of minimum size of riprap in the gradation, in pounds.

The average K_{RR} values determined in the model was 1.3 for a water depth of 20 feet and 1.7 for a water depth of 40 feet.

Table 1

AVERAGE K_Δ VALUES

ARMOR	CONDITION*			
	1	2	3	4
Rounded Stone, 2 layers pell mell	2.6	2.5	2.4	2.0
Rough Stone, 2 layers pell mell	3.5	3.0	2.9	2.5
Rough Stone, 2 layers placed**	5.5	5.0	4.5	3.5
Tribars, 2 layers pell mell	10.	8.5	7.5	5.0
Tribars, 1 layer uniform***	15	12	9.5	7.5
Tetrapods, 2 layers, pell mell	8.5	7.5	6.5	4.5
Quadripods, 2 layers pell mell	8.5	7.5	6.5	4.5

* Condition
    1. Trunk, nonbreaking waves
    2. Trunk, breaking waves
    3. Conical head, non-breaking waves
    4. Conical head, breaking waves

** Good placement with centerline of long diameter of stone placed normal to structure face.

*** For special conditions.

One of the major problems in the construction of structures subjected to wave action is the actual lifting and placing of the larger units. This problem became critical at Kahului, Hawaii since the nearness of the channe to the structures prevented flattening of the slope. With a 34-foot design

wave, concrete units weighing 35 tons were stable on a 1 on 3 slope except along the portion of the head where the jet action occurred. Available equipment would handle a maximum of 35 tons with a reasonable reach. The largest equipment available for limited length of time could be rigged and counterweighted so that 50 tons could be lifted with a maximum reach of about 50 feet. To compensate for these limiting conditions, units weighing 50 tons are to be placed in the area where damage occurs and 35-ton units along the remainder. Beneath a point one wave height below design still water level 18-ton units will be used.

A similar remedy was contemplated for use at the Oregon Harbor of Siuslaw, Umpqua and Coos Bay. Suitable stone was available for use in these harbors and a reasonable percentage could be expected to break in pieces in which the longest dimension would be about three times the smaller dimension. For these cases, the stone is to be placed in the critical areas with the long dimension perpendicular to the structure face. This additional friction and keying action will provide the desired stability. It must be recognized that such special placement can be accomplished only above the water surface, therefore, flatter slopes or larger units are required below mean still water level.

Both of these special placement procedures were tested in wave flumes at the Waterways Experiment Station. These tests indicated that such special placement is desirable especially in cases where the range in direction of wave approach is limited.

Tests using non-breaking waves have shown a more or less constant pattern of energy effect vertically along a structure. Breaking waves have shown greater variability in this effect. The shape of the wave at a breaking point apparently has a large affect on the pressure exerted and upon the distribution of this pressure. The non-breaking waves and earlier tests using breaking waves showed significant pressure ranging to about one wave height below still water level. Later project tests have indicated variability in this pattern. Some of the tests of the project model studies of Tsoying Harbor on Taiwan and Kahuluai Harbor, Hawaii, have indicated that breaking waves of some shapes exert pressures only a very short distance below still water level. Additional tests and analyses will be made to determine the governing factors in this variance in pressure distribution. However, until this effect is further investigat primary cover layers should extend at least one wave height below lowest still water level.

Head Stability - The design of the head of the breakwater and jetty is very critical since this portion of the structure is more frequently dan than any other. This is due principally because of the rounded shape which allows waves from all directions to break directly upon it. In this breaki a jet is formed across the slope which tends to roll out the units. Model studies have emphasized the fact that this damage generally occurs on a rear quadrant from the direction of wave approach. Several thoughts have been proposed to overcome this effect. One method is to flatten the slope in the direction of wave approach and steepen the slope of the rear quadrar

so that the wave will be forced to break earlier and that the water may
fall free after breaking. Special attention must be given to head design
as greater forces are acting on the head than on the trunk section. It
may be noted in Table 1 that separate K_Δ values are given for the head
and the trunk.

Concrete Components - Because of the need of heavier individual units
and also to the dwindling supplies of stone near out coasts, significant
attention has been given to the use of concrete components. In the United
States principal investigation has been on tetrapods, tribars and quadripod
Other shapes have been proposed, but at the present time sufficient infor-
mation is not available to the author to comment on these various shapes.

The first of the armor units, of significant value, was the tetrapod,
developed and patented in 1950 by the Neyrpic Laboratories of Grenoble,
France. The tetrapod is an unreinforced concrete shape with four truncated
conical legs projecting radially from a center point. The tetrapod was
discussed by Mr. Pierre Danel in the Proceedings of the Fourth Conference
on Coastal Engineering.

A more recent concrete shape is the tribar, developed and internation-
ally patented by Mr. R. Q. Palmer of the Corps of Engineers, Honolulu Dis-
trict. The tribar is an unreinforced concrete shape consisting of three ba
tied together by three radial arms. The use of tribars was discussed in a
paper by Mr. Palmer entitled "Breakwaters in the Hawaiian Islands", publish
in the Journal of the Waterways and Harbor Division, A.S.C.E., June 1960.
The quadripod was developed by the Corps of Engineers, and is a concrete
shape of four truncated legs with three of them in the same plane and
projecting from a center point. The fourth pod projects from the same
center with its centerline making an angle of 90° with the plane of the
other pods. In the United States tetrapods have been used at Crescent City
Harbor, California, Kahului Harbor, Hawaii and on Rincon Island, California
Tribars were used at Nawiliwili Harbor, Hawaii. Quadripods will be used on
the east breakwater of Santa Cruz Harbor, California, which is now under
construction.

The design of cover layers using concrete components is similar to
that for quarry stone structure. The Hudson equation may be used with the
applicable value of K_Δ. The principal point to consider in the design of
these structures is to contain them on the slope so that rolling movement
will not occur. Experience has indicated that principal damage to the unit
is the shearing of legs caused by rolling. To obtain this containing featu
it is generally necessary to provide a concrete cap with the possible addi-
tion of spaced concrete posts along the side of the cap to further reduce
the possibility of the units being rolled to the crest of the structure.
Casting, curing and placing techniques are discussed by Blume and Keith in
their article entitled "Rincon Offshore Island and Open Causeway", A.S.C.E.
Proceedings Paper No. 2170, published in September 1960, and by Deignan in
his paper entitled "Breakwater at Crescent City, California", published

in A.S.C.E. Proceedings Paper No. 2174 of September 1959. It is evident
that greater porosity is obtained by these concrete units and back pres-
sures, which are often a problem in stone construction, are reduced. The
feature in which more information is needed is on the actual durability
of the units in a prototype structure. Limited experience to date in-
dicates that their structural durability is adequate except in cases
where rolling on the slope occurs.

One of the major advantages of concrete components is the amount of
porosity obtained which permits dissipation of the energy of the attacking
wave. The amount of porosity in percent voids, determined from model units
for various cover units follows in Table 2.

Table 2

POROSITY OF VARIOUS ARMOR UNITS

ARMOR UNIT	NO. OF LAYERS	POROSITY IN %
Riprap	graded	37
Quarrystone	2	38
Quarrystone	3	40
Tetrapod	2	50
Quadripod	2	50
Tribar	2	54
Tribar*	1	47

* Uniform placement.

CONCLUSIONS

Design of coastal structures cannot, because of the variability of
forces and the nature of the phenomena, become a handbook process. Ingenui
and judgment will always be a major requirement of a designer. However,
greater understanding of the phenomena and the development of the empirical
data is needed for development of methods for designing more economical anc
efficient structures. To obtain this need there must be even greater coop-
eration between the designer and the researcher and the governments, the
universities and the individuals must place greater and greater emphasis
on oceanographic research.

This paper has attempted to point out some of the laboratory findings
that have been applied to practical design methods and the manner in which
it was applied. It is hoped that areas of needed research have also been
inferred.

RECENT ADVANCES IN COASTAL STRUCTURE DESIGN

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