

## Chapter 32

### INTERLOCKING PRECAST CONCRETE BLOCK SEAWALL

by

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#### ABSTRACT

As a result of a survey of damage caused by the severe storm of March 1962 which affected the entire east coast of the United States, a new and specially-shaped interlocking concrete block was developed for use in shore protection. This block is designed to be used in a revetment-type seawall that will be both durable and economical as well as reduce wave run-up and overtopping, and scour at its base or toe. A description of model investigations conducted on the interlocking precast concrete block seawall and results therefrom are presented. It is shown that effective shore protection can be designed utilizing these units.

#### INTRODUCTION

As a result of the severe east coast storm of 5-9 March 1962, a study was made of certain types of structures with a view to low-cost positive means of protecting backshore property from wave action and severe flooding. The storm showed that natural features in the form of a wide, high beach berm and a wide belt of sand dunes are the best protection to the backshore area. Rapid increase in population of beach areas has resulted in the leveling of many dune areas by the developers and a loss of this natural protection. In some areas attempts are being made to restore this natural protection by providing wide beach areas by means of artificial beach fill and encouraging the natural rebuilding of the protective dunes by using a system or series of sand fences. In other areas, such as built-up commercial areas along the beach, rebuilding of dunes is virtually impossible because of intensive land development, although wide beaches can be provided by artificial beach fills.

The March 1962 storm also graphically illustrated the limitations of timber bulkheads (as differentiated from seawalls) to withstand the direct forces of severe storm waves. The failure of these bulkheads, which resulted in extensive backshore damage, can be attributed to (1) the loss of the beach fronting the structure which allowed larger waves to break against the face of the wall and (2) the loss of fill material behind the wall caused by overtopping waves.

For reasons cited above, the need is very apparent for a protective structure which will reduce wave run-up and overtopping, and not induce scour at the seaward toe. During high water stages associated with storms

vertical walls are especially likely to produce conditions which will induce beach scouring. Once this scour has occurred, even ordinary waves break directly against the walls, and the natural rebuilding or accretion of the beach under normal wave and tide conditions is greatly inhibited by the reflected wave. It is well known that a rubble revetment or seawall would be most effective in absorbing wave energy and in reducing wave run-up and overtopping, but this type of structure also introduces a few undesirable features: the first of these is that it limits access to the beach to those areas where suitable stairways across the rough surface of the rubble slope are provided; the second is that it introduces a safety hazard to those people who may cross the rubble slope to the beach and third, it presents an unattractive or non-aesthetic appearance. An interlocking concrete block type of seawall would minimize these undesirable features and was therefore selected for study.

#### CRITERIA FOR SELECTION OF DESIGN

Primarily, the most important factor of the block design is the interlocking feature, the secondary features are the size and weight of the precast concrete shape. Blocks should be heavy enough, recognizing the interlocking feature, to be stable under design-wave conditions, and yet consideration should be given to the weight for the handling of individual units with small or light crane equipment. Another feature is surface roughness. It is recognized that the greater the surface roughness the greater the reduction in wave run-up, overtopping, backwash and reflection. Other features are those of appearance and utility. The precast concrete wall is more attractive than a rubble slope and provides easy access to the beach anywhere along its length without the need for special ramp areas.

#### DESIGN CONDITIONS

Basically the interlocking precast concrete block stepped-face seawall is designed in conjunction with a protective beach, the wall being a last line of defense against wave action accompanying severe storm surges. Minor flooding rather than complete destruction of backshore development may occur when protective beach defenses are temporarily eroded away or overtopped. The concrete block seawall as envisioned would extend from the elevation of the crest of the dune seaward on a 1 on 2 slope to mean low water (see Figure 1). The beach fronting the seawall would have to be severely eroded before large storm waves would impinge directly against the seawall. Studies of the March 1962 east coast storm showed that maximum erosion of the beach face (vertical height) could be as great as 8 feet below mean low water for a storm of this magnitude; therefore a cutoff or toe wall is incorporated in the overall design to prevent undermining and failure during severe storms. The sheet pile cutoff wall should extend from the toe of the slope of the wall face to -10 feet MLW as a minimum, or to a greater depth depending on the anticipated depth of scour of the fronting beach during storms in the area where the structure is located. This cutoff wall may be constructed of timber, concrete, or steel sheet piling. Walers and tie backs would be required on the wall in order to prevent its failure from excessive pressure from the backshore side of the wall, that is, the load created by the sand and concrete blocks.

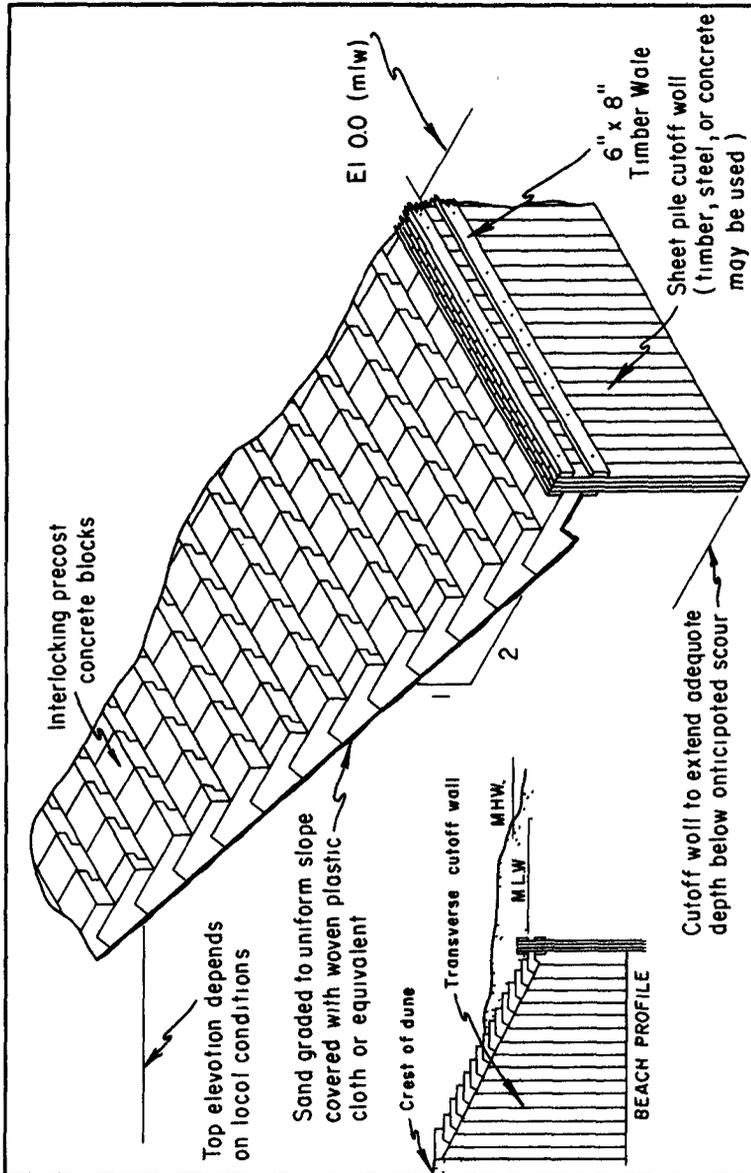


FIGURE 1. STEPPED TYPE INTERLOCKING PRECAST CONCRETE BLOCK SEAWALL

The design wave height (equivalent deep water wave  $H_0'$ ) was selected as 6 feet with a breaking height on the seawall varying from 9.0 to 11.0 feet depending on the wave period and the depth of water immediately seaward.

In any precast concrete block design, consideration of a properly designed underlying filter is of prime importance, for without the filter the sand would be lost through the joints between the blocks. This loss of foundation material can be caused by piping of the sand through the joints due to release of hydrostatic pressures resulting from wave action which in turn can cause undermining and result in failure of the seawall section. One type of filter in use is a plastic (polyvinylidene chloride resin) cloth. This material, woven in a mesh fine enough to be impermeable for sand with average particle diameter of 0.08 mm, has, up to the present, operated satisfactorily as a filter. Properly graded gravel blankets also make satisfactory filters. A combination of the plastic and gravel filter can be used should foundation conditions and hydrostatic pressure relief warrant. Transverse cutoff walls are required to compartment the structure at regular intervals along the shore front thereby minimizing the chance of total failure should one of the compartments become unravelled during sustained damaging wave conditions.

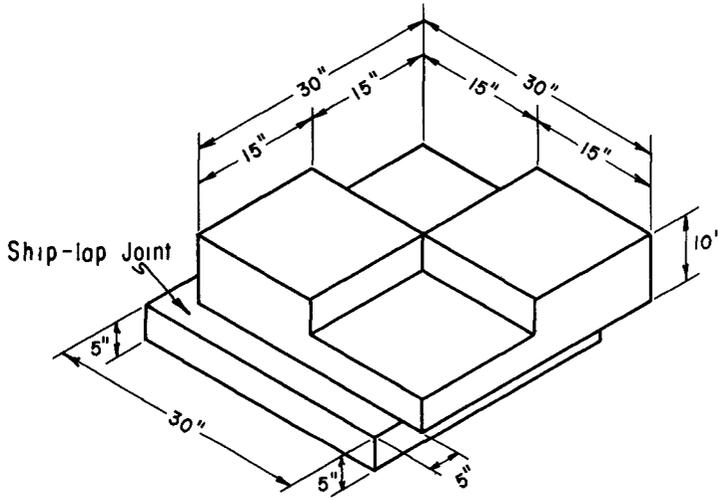
#### BLOCK DESIGN

The use of interlocking concrete blocks is not a new concept for shore protection. Such blocks have been used extensively both in The Netherlands and England, but only recently has their use obtained any degree of prominence in the United States. Typical blocks, both in Europe and the United States, are generally square slabs with ship-lap type interlocking joints. The types of block considered for this study are shown in Figure 2. Design A, a "waffle" type interlocking block, is designed to lie flat on the surface of the graded slope, and its outer face has alternately raised squares, similar to those on a waffle iron, to provide surface roughness. The joint is of the ship-lap type and provides a mechanical interlock with adjacent blocks. Design B is a step-type inclined-face interlocking block. The name is derived from the fact that after placement the riser face of the step is normal to the slope and thus inclined with the horizontal. The block is interlocked with adjacent blocks by means of ship-lap joints on two sides and by an overlapping projection extending behind the block on which it rests. Design C is a step-type vertical-face block of the same design as the inclined-face block except that the riser face is vertical rather than normal to the slope.

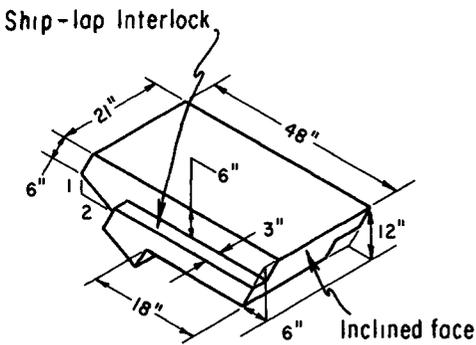
#### MODEL TESTS

##### GENERAL

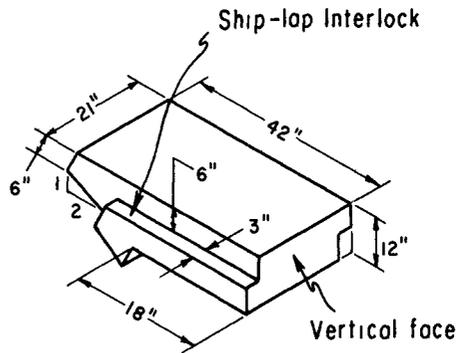
The reaction of waves to a structure and the reaction of the structure to the waves are independent, simultaneously occurring functions to be considered in determining the feasibility of a structure for shore protection. Thus model tests of proposed designs were conducted first to determine which



**DESIGN-A  
WAFFLE TYPE**



**DESIGN-B  
INCLINED-FACE**



**DESIGN-C  
VERTICAL-FACE**

**FIGURE 2. INTERLOCKING PRECAST CONCRETE BLOCK**

wall-type most effectively dissipated or absorbed the energy of the waves and second, to determine the stability characteristics of such a structure under the action of waves in order to eventually find the optimum wall design.

#### WAVE RUN-UP TESTS

The first series of tests on wave run-up were carried out in a wave tank of 72-foot length. Of specific interest was the height above still water to which the waves rose on the structure. Four wooden models were tested in this series, each built to a 1:16 scale. Two stepped-face seawalls, each having a 1 on 2 face slope, were tested, one being a vertical riser stepped-wall and the other an inclined riser stepped-wall but each with identical step height. The riser of the latter was normal to the back slope. The vertical riser stepped-wall was also tested on a 1 on 3 slope. The details of these steps are shown in Figure 2 (Design C). A "waffle"-type block wall was tested on a 1 on 2 slope. A model of the waffle-type block is shown in Figure 2 (Design A). In addition to testing the four alternative designs under "deep water" conditions, that is, with the toe of the structure extending to the bottom of the wave tank, the inclined riser stepped-seawall on a 1 on 2 slope was tested while fronted by an arbitrarily placed beach representing extreme erosion conditions. This prototype condition is represented in Figure 3. Each model was subjected to a wide range of wave conditions, ranging in prototype height from 0.4 to 10.7 feet and in prototype period from 2.9 seconds to 18.8 seconds. The prototype water depth in the tank was 20 feet at all times.

The results of the run-up tests are presented in Figures 4 through 7. Relative run-up ( $R/H_0'$ ), or actual run-up ( $R$ ) (where  $R$  = vertical distance) divided by the equivalent deep water wave height ( $H_0'$ ), is shown as a function of wave steepness ( $H_0'/T^2$ ), or that is equivalent deep water wave height ( $H_0'$ ) divided by the square of the wave period ( $T$ ), for constant depth ( $d/H_0'$ ) at the structure's toe, or water depth ( $d$ ) at the toe of the structure divided by the equivalent deep water wave height ( $H_0'$ ). The equivalent deep water wave height ( $H_0'$ ) is that deep-water wave height corresponding to the actual wave height ( $H$ ) at depth ( $d$ ) which has been corrected for shoaling effect but not for refraction. There was no refraction effect pertinent to these tests. The parameter chosen to represent wave steepness,  $H_0'/T^2$ , differs from the true steepness,  $H_0'/L$ , by the constant 5.12 through the relationship  $L_0 = 5.12 T^2$ , where  $L_0$  is the deep water wave length. Additional data from smooth seawall tests have been made available and are presented for comparison.

The data as presented allows a comparison to be made of the relative effectiveness of the wall-types for reducing the run-up. It can be seen that the types of relationships obtained for the roughened walls are very similar except for the wall fronted by a beach. For this exception a curve with a "camel's back" shape consistently appears. The first maximum on this curve is associated with the wave that shoals and breaks just on the structure. It is apparent from the data that any form of roughness is far better than none at all from the standpoint of reduced run-up. The difference in run-up between a smooth wall and the least effective roughened wall

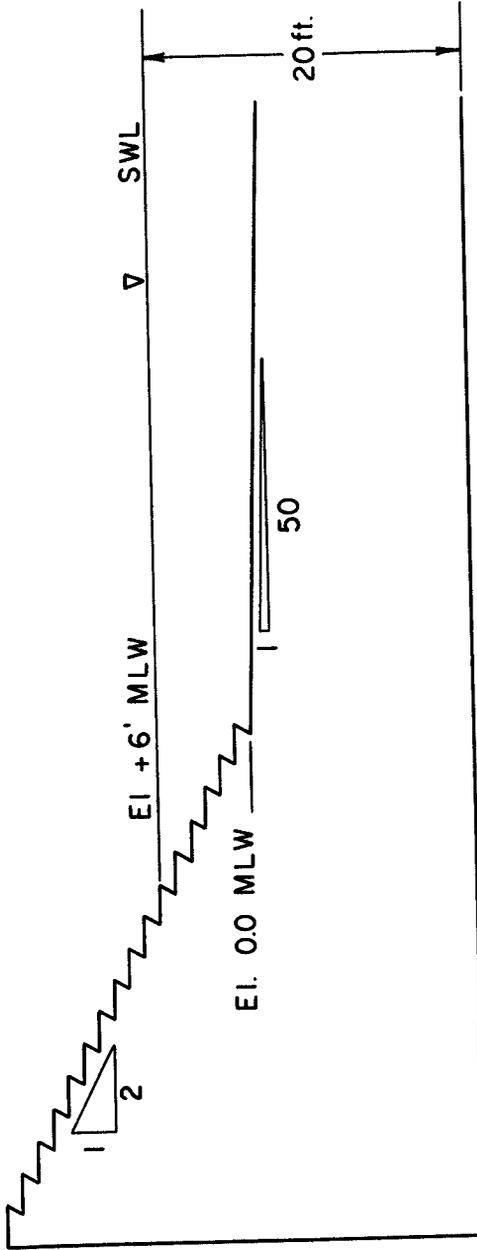


FIGURE 3. PROTOTYPE CONDITION OF RUN-UP TEST - SEAWALL  
FRONTED BY PARTIALLY ERODED BEACH

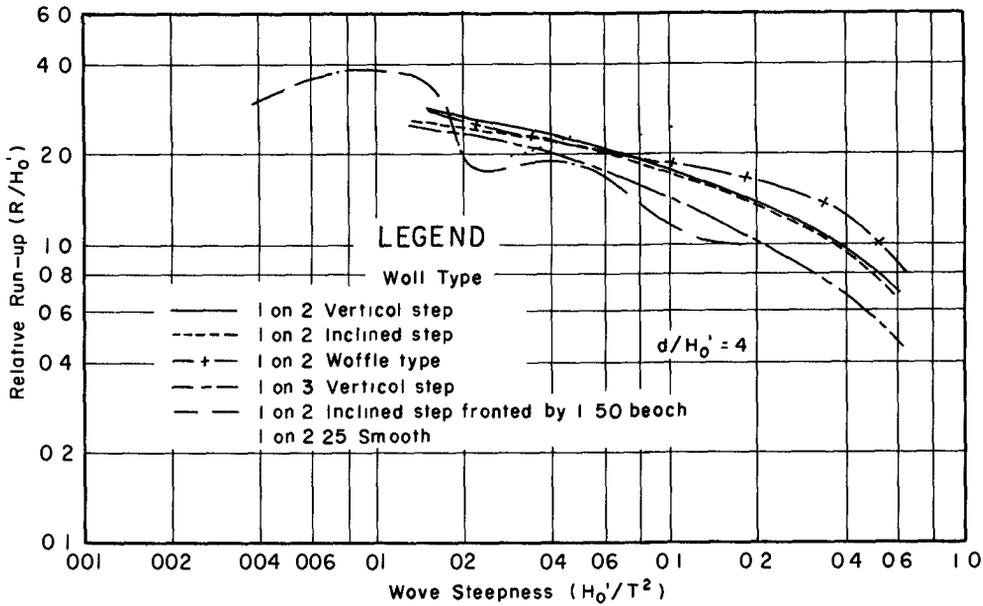


FIGURE 4 RELATIVE RUN-UP ( $R/H_0'$ ) ON ALTERNATE SEAWALL DESIGNS VS WAVE STEEPNESS ( $H_0'/T^2$ ) FOR CONSTANT STRUCTURE DEPTH ( $d/H_0'$ )

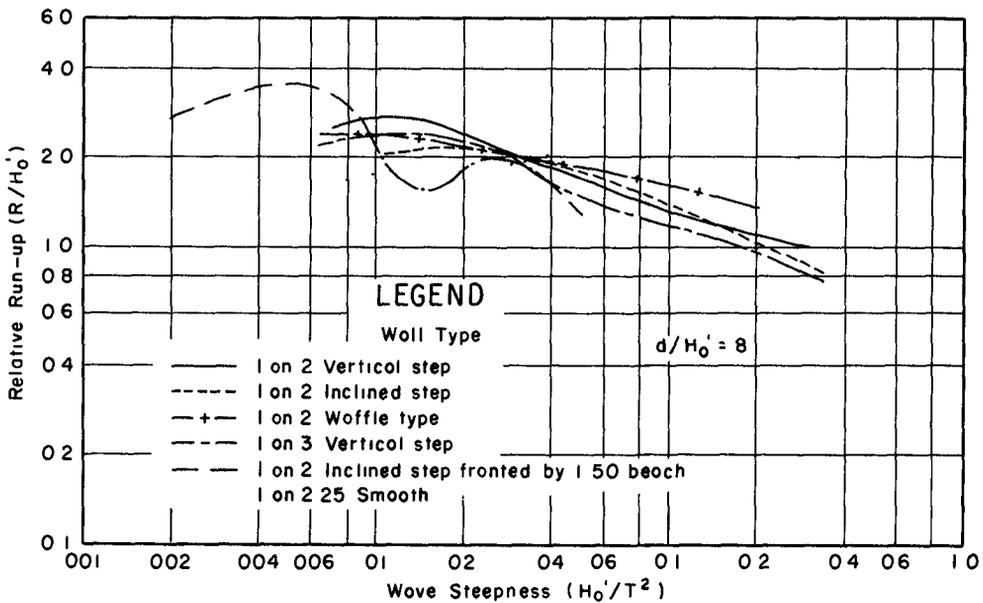


FIGURE 5. RELATIVE RUN-UP ( $R/H_0'$ ) ON ALTERNATE SEAWALL DESIGNS VS WAVE STEEPNESS ( $H_0'/T^2$ ) FOR CONSTANT STRUCTURE DEPTH ( $d/H_0'$ )

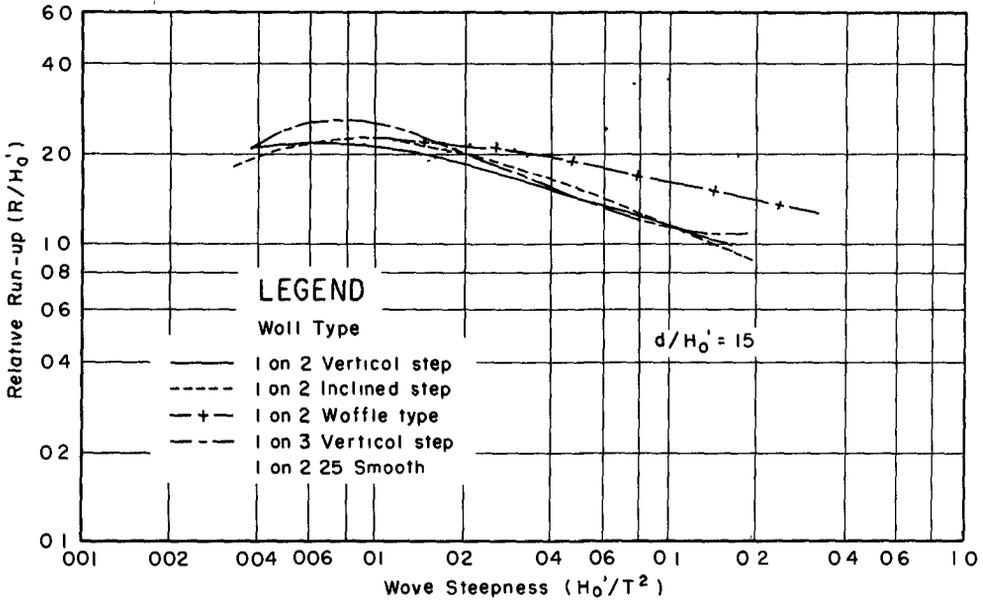


FIGURE 6 RELATIVE RUN-UP ( $R/H_0'$ ) ON ALTERNATE SEAWALL DESIGNS V WAVE STEEPNESS ( $H_0'/T^2$ ) FOR CONSTANT STRUCTURE DEPTH ( $d/H_0'$ )

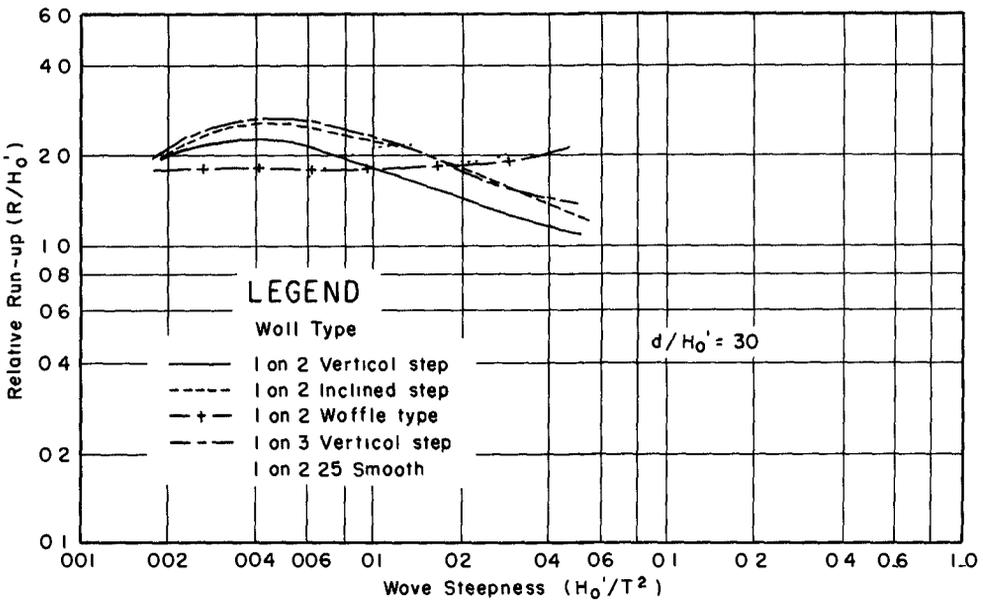


FIGURE 7 RELATIVE RUN-UP ( $R/H_0'$ ) ON ALTERNATE SEAWALL DESIGNS WAVE STEEPNESS ( $H_0'/T^2$ ) FOR CONSTANT STRUCTURE DEPTH ( $d/H_0'$ )

is generally much greater than the individual differences between roughened walls. There is, however, a consistent pattern appearing in the comparative data for the roughened walls. The waffle-type block is the least effective design tested for reducing run-up, especially for steeper waves. This is not to say that waffle-type block walls are less effective in dissipating wave energy than stepped-face walls for all conditions of relative face roughness. For these tests only one actual condition of relative roughness for each type of wall was utilized. Little difference in the run-up characteristics on the vertical-riser stepped seawall and the inclined-riser stepped seawall can be seen. There is, however, an indication that the inclined-riser steps might dissipate slightly more wave energy than the vertical-riser steps. This difference, if it exists at all, is quite small. The effect of a flatter wall slope for the wave conditions of major interest is beneficial. For the steep or storm waves, the run-up on the 1 on 3 wall is appreciably less than that on the 1 on 2 wall. If the height of a prototype seawall is a critical factor, then a flatter slope should be used. The flatter slope will, however, increase the number of precast concrete block units required and thereby increase the cost of the wall. The beach in front of the seawall causes the steeper waves to break in front of the structure, thereby dissipating much of their energy before reaching the wall. The data shows that the maximum relative run-up ( $R/H_0'$ ) on the wall in shallow water is reached by waves that break directly on the structure. Figure 8 indicates the relative run-up vs wave steepness observed during the stability tests at a scale of 1:10; the data actually represent the relative run-up for the beach in its maximum eroded condition and eroded to mean low water at the toe of the wall.

To summarize the run-up tests for the proposed seawall designs, the most effective structure for reducing wave run-up is the stepped-face seawall, possibly with inclined-face block; however, any advantage held by the inclined-face block over the vertical-face block must be slight if it exists at all. The waffle-type is the least effective design for reducing wave run-up. The effect of decreasing the slope of the wall is to decrease the relative run-up ( $R/H_0'$ ) for steep waves. The existence of a beach in front of the wall causes the relative wave run-up to be low except for waves of very low steepness ( $H_0'/T^2$ ). The maximum wave run-up is reached when waves break directly on the structure. When waves break offshore the wave run-up decreases accordingly. Those run-up tests conducted during the stability tests indicated that the run-up and overtopping were greater for surging waves, as differentiated from plunging waves, at the time of breaking.

#### STABILITY TESTS

As a result of wave run-up tests, it was determined that the first series of stability tests would be conducted on the inclined-face block seawall. It was recognized that the use of the inclined-face blocks (Figure 2, Design B) presented a conservative test condition in that the inclined faces would be subjected to greater lifting forces than would vertical-face blocks (Figure 2, Design C). The stability test section was constructed as a 1:10 scale model of a typical prototype installation.

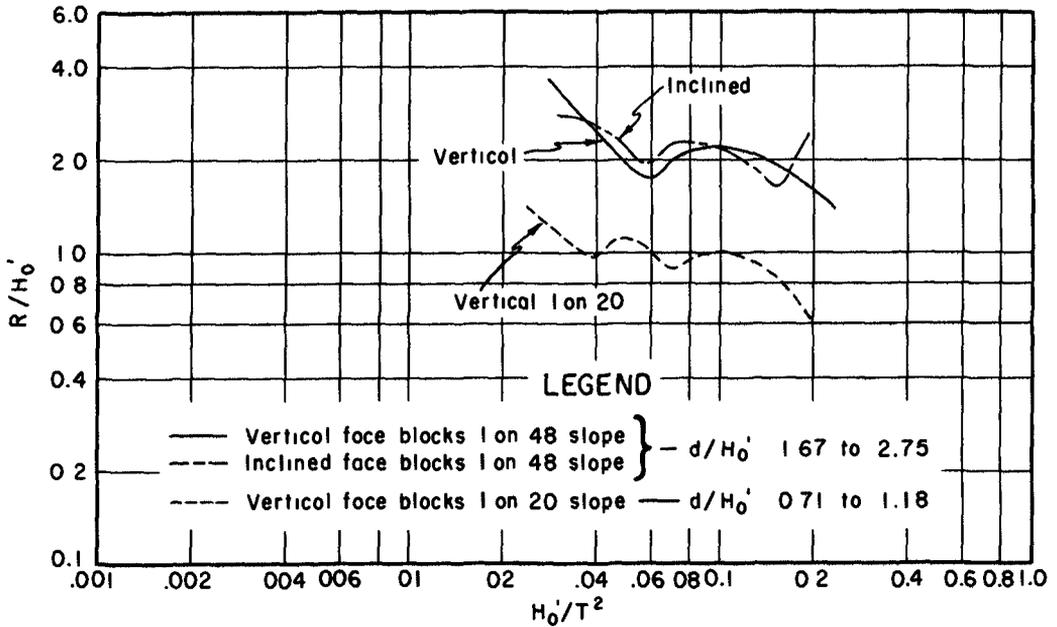


FIGURE 8. WAVE RUN-UP ON CONCRETE BLOCK SEAWALL

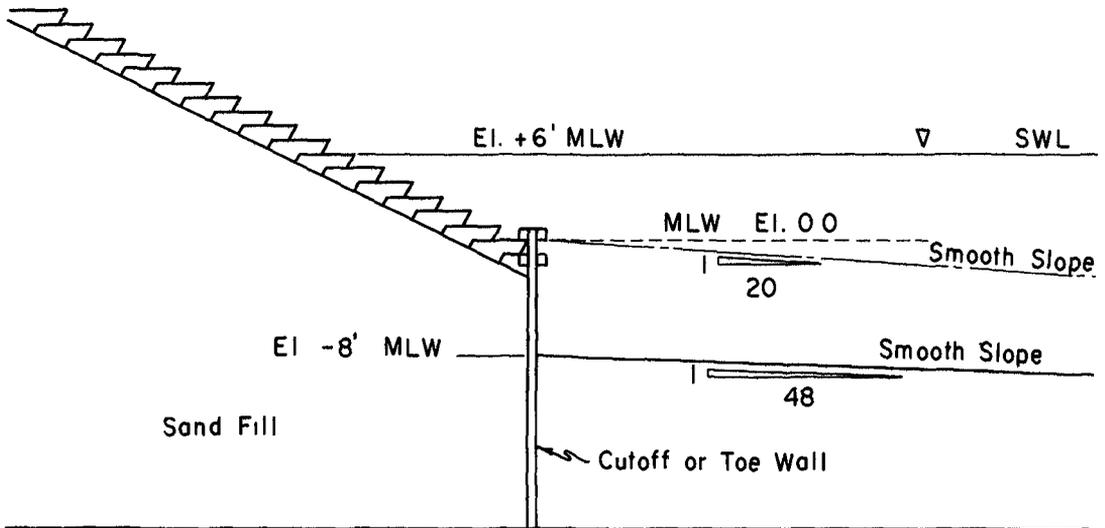


FIGURE 9. PROTOTYPE CONDITIONS OF STABILITY TESTS

The prototype design wave height (H) for this seawall was estimated to be in the order of 6 feet where H equals the wave height unaffected by reflections which would exist at the structure site.

The first series of tests was conducted under the following conditions (see Figure 9): inclined-face blocks (Design B) extending from MLW to El. +23 feet with a beach slope of 1 on 48 fronting the toe wall and scour to El. -8 feet (MLW) (maximum erosion conditions), and a design water level at El. +6 feet (MLW). (All dimensions given are at prototype scale.) A maximum wave height (H) of 8 feet (equivalent deep water wave height,  $H_0' = 8.4$  feet or wave height at breaking,  $H_b = 13.9$  feet) did not produce any significant displacement of the blocks, even after a (prototype) duration of 10 hours, for a wave period of 7 seconds; however, failure of the inclined-face blocks did occur for a 13-second wave period when wave height (H) was 10.2 feet (or  $H_0' = 8.6$  feet,  $H_b = 17.6$  feet). The duration of the latter test conditions producing failure of the wall varied from failure after only 1.25 hours to no failure after 10 hours. It thus became apparent that another variable in the test procedure was affecting the duration time to produce wall failure, and this was concluded to be the method of anchoring or restraining the top row of blocks (at El. +23 feet) against displacement by overtopping waves. Looking also at the mode of failure, it was realized that the blocks were being forced up the slope by a succession of plunging or surging waves and the degree of restraint was the prime factor in determining the stability duration for the wall.

A second series of tests was conducted on a combination of vertical-face blocks and inclined-face blocks under the maximum erosion condition (previously designated as -8 feet MLW). The vertical-face blocks were installed to El. 15 feet (above MLW) or 9 feet above design water level and inclined-face blocks from El. 6 feet to 23 feet. In the previous series of tests, the seawall of inclined-face blocks started to fail at El. +8 feet (or 2 feet above the design water level) whereas the combined block seawall failed at the first or lowest row of inclined-face blocks or El. +16 feet. This failure resulted from a 10.2-foot wave height (H) ( $H_0' = 8.6$  feet and  $H_b = 17.6$  feet) and a 13-second wave period after a duration of 8.4 hours.

The failure at this first or lowest row of inclined-face blocks indicated that these blocks are not as stable as blocks with vertical faces, thus confirming an original concept relative to inclined and vertical-face blocks.

The third series of tests was conducted on the vertical-face blocks under the maximum erosion condition. The vertical-face block seawall was subjected to the following wave conditions which were the maximum capable of being generated by the available equipment.

Wave Period (T)	Wave Height		
	H	$H_0'$	$H_b$
7 seconds	8.1	8.4	13.9
10 seconds	7.7	7.2	13.3
13 seconds	10.2	8.6	17.6

The blocks from MLW through E1. +23 feet under the maximum wave conditions were completely stable and showed no sign of movement.

The fourth series of tests was conducted on the vertical-face blocks as in the previous series of tests except that the top row, rather than being at E1. +23 feet, was lowered to E1. +12 feet. The blocks at this lowered elevation were not anchored as in the previous test. The purpose of this series of tests was to determine the wave conditions (height and period) under which the block seawall and specifically the top row would be stable with an extremely eroded beach condition fronting the structure and still water level at E1. +6 feet. This condition closely approaches an actual prototype installation without the use of a concrete parapet wall at the top of the slope or any anchoring of the top row of blocks. For an actual prototype installation it is anticipated that some method of anchoring will be used for the top row of blocks, either in the form of a concrete parapet re-entrant-face type wall or a continuous concrete beam.

It was determined that the top row of blocks (E1. 12 feet) was stable as no discernible movement occurred for the following wave conditions.

Wave Period (sec)	Wave Height (ft)			Type of breaking wave
	H	$H_0'$	$H_b$	
12.6	6.1	5.2	10.6	Surging
10.0	5.6	5.3	9.8	"
7.0	5.7	6.0	9.8	"
5.7	5.8	6.3	10.1	Plunging

The values obtained for wave height (H) appear to verify the original estimate of a 6-foot design wave.

#### CONCLUSIONS

It may be concluded from the results of model tests conducted to date on interlocking precast concrete block seawalls that:

a. In addition to wave steepness ( $H_0'/T^2$ ) the relative wave run-up ( $R/H_0'$ ) is influenced by the following:

1.  $(d/H_0')$  relative depth of water at the toe of the structure.
2. The type of block roughness, that is, waffle, inclined-face or vertical-face.

b. The vertical-face blocks are more stable than the inclined-face blocks.

c. The stability number,  $N_s$ , (as derived by R. Hudson of the U. S. Army Engineer Waterways Experiment Station, in his rubble-mound stability tests), has a minimum value of 12.8 for these vertical-face blocks, but

this large value is attributed to a great extent to the mechanical interlock of the ship-lap joints.

d. The failure of the concrete block occurs in either of two ways:

1. In the case of the high seawall with the top row of blocks partly restrained, the blocks at or slightly above SWL are forced up the slope by each incident breaking wave, and when sufficient space between individual blocks has been created to render one ship-lap joint ineffective, a gradual dislodging of the blocks in the area of SWL follows.
2. In the case of a low seawall with the top row of blocks unrestrained, the top row is lifted up and displaced by the uprush of the overtopping wave and progressive failure results.

e. The vertical-face blocks can be used to provide a stable structure for incident wave heights greater than 6 feet when adequate restraint or anchoring is provided to the top row of blocks, either in the form of a low parapet wall or a beam.

#### ACKNOWLEDGMENTS

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