MISCELLANEOUS PAPER H-74-2

CONCRETE ARMOR UNITS FOR PROTECTION AGAINST WAVE ATTACK

REPORT OF AD HOC COMMITTEE ON ARTIFICIAL ARMOR UNITS FOR COASTAL STRUCTURES

Edited by

R. Y. Hudson

January 1974

Sponsored by Office, Chief of Engineers, U. S. Army

Published by U. S. Army Engineer Waterways Experiment Station
Hydraulics Laboratory
Vicksburg, Mississippi

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FOREWORD

The Ad Hoc Committee on Artificial Armor Units for Coastal Structures was formed by the Office of the Chief of Engineers in December 1970. The mission of the committee was to develop material that will provide guidance for decisions that must be made with respect to the selection and design of concrete armor units for use as protective cover layers of coastal structures subjected to wave attack. The guidance included in this report has utilized the most up-to-date technical knowledge available to the Corps of Engineers that can be verified. Any judgmental statements made are based on the joint experience of the members of the Ad Hoc Committee. Members of this Committee are: Norman L. Arno, North Central Division (NCD), Chairman; Robert Y. Hudson, Waterways Experiment Station (WES); Robert A. Jachowski, Coastal Engineering Research Center (CERC); and Orville T. Magoon, South Pacific Division (SPD).

The review and assistance of Mr. Neill E. Parker (OCE), Dr. R. W. Whalin (WES), and Mr. D. D. Davidson (WES) in publishing this report are gratefully acknowledged.

Director of WES during the preparation and publication of this report was COL G. H. Hilt, CE. Technical Director was Mr. F. R. Brown.

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FIGURES B1-B3

NOTATION

A	Surface area, sq ft
d	Water depth referred to a selected still-water level, ft
d_b	Depth of water in which waves of given dimensions will break, ft
D	Damage to a breakwater cover layer, percent
h	Crown elevation referred to a selected still-water level, ft
H	Wave height, ft
H_b	The height of a wave that breaks in depth d_b, ft
H_{DW}	The selected design-wave height, ft
H_o	Deepwater wave height, ft
k_Δ	Experimental coefficient in equations 6 and 7
K_D	Experimental coefficient in equations 4, 5, and 9 corresponding to given percentages of damage to the cover layer
K_s, K_r, K_{fp}	Coefficients of shoaling, refraction, and friction-percolation, respectively, in the equation $H_{DW} = K_s, K_r, K_{fp}, H_o$
L	Wavelength in depth d, ft
L_o	Wavelength in deep water, ft
n	Number of layers of armor units
N_r	The required number of armor units for a given surface area of the cover layer and a given value of n
P	Porosity (void ratio) of cover layer, a number less than one
R	The height of wave runup, measured vertically, above the selected still-water level, ft
S_r	Specific gravity of the armor unit relative to the water in which the structure is situated ($S_r = \gamma_r/\gamma_w$)

t, t_1, t_2	Thickness of armor-unit layer, first underlayer, and second underlayer, respectively, ft
w_c	Width of crown, ft
W_r, W_1, W_2	Weight of armor unit, stone in first underlayer, and stone in second underlayer, respectively, lb
α	Angle of breakwater slope measured from the horizontal, degrees
γ_r	Specific weight of armor unit, pcf
γ_w	Specific weight of the water in which the structure is situated, pcf
ϕ	Coefficient in equation 9

CONVERSION FACTORS, BRITISH TO METRIC UNITS OF MEASUREMENT

British units of measurement used in this report can be converted to metric units as follows:

Multiply	By	To Obtain
inches	2.54	centimeters
feet	0.3048	meters
square feet	0.092903	square meters
pounds	0.45359237	kilograms
tons	907.185	kilograms
pounds per cubic foot	16.0185	kilograms per cubic meter
pounds per square inch	0.070307	kilograms per square centimeter

SUMMARY

A large number of special-shaped concrete armor units, for use in the protective cover layer of rubble-mound structures exposed to storm-wave action, have been developed throughout the world in the past 20 to 25 years. Quarrystone armor units may be used when available at a competitive price and when the wave conditions at the structure site are not too severe. The purpose of this report is to provide design information and guidance in selecting the shape and size of concrete armor units for use in constructing rubble-mound structures that will be stable at a minimum of cost.

The factors that determine the choice of design waves are described; the development of a stability formula and stability coefficients for different amounts of damage to the structure are presented; the advantages of using high-density concrete in the fabrication of armor units are stressed; and the casting of armor units, the problems of breakage, and the legal aspects of using concrete armor units of special shape are discussed.

Based on the results of test data available to date, it was concluded that the tetrapod, tribar, and dolos armor units should be considered for use in the design of rubble-mound breakwaters and jetties when the use of quarrystone is not feasible. The dolos armor unit is believed to be the most efficient, and procedures for the design of a typical breakwater cross section, using these armor units for the protective cover layer, are presented. The use of hydraulic model studies to determine the optimum design of proposed breakwaters is discussed.

Appendices A and B present summaries of the Corps of Engineers experience in the use of concrete armor units, and flow diagrams for decision-making in the design of rubble-mound structures.

CONCRETE ARMOR UNITS FOR PROTECTION AGAINST WAVE ATTACK

PART I: INTRODUCTION

1. Designers of rubble-mound structures and the facings of earth-fill dams for protection against wave action are finding that the need for better armor units is increasing. The use of concrete armor units has several advantages, especially in areas requiring protection from the destructive energy of large waves and in areas in which stone costs are high. Compared with stone, concrete armor units are more complex in shape; and design information on both hydraulic and structural factors is required.

2. A chain of basic decisions must be made when selecting and designing a concrete armor unit and the section of the structure on which it will be placed. These decisions and various considerations are discussed in this report to provide guidance to planners and designers in the selection of concrete armor units for use on structures that are subject to wave attack.

3. Concrete armor units have been used on various rubble-mound structures throughout the world. Structures designed and constructed by the Corps of Engineers that employ concrete armor units are:

Location	Armor Unit	Date
Crescent City Harbor, Calif.	25-ton*tetrapods	1957
Crescent City Harbor, Calif.	42-ton dolosse	1973
Kahului Harbor, Hawaii	33-ton tetrapods	1957
Kahului Harbor, Hawaii	35- and 50-ton tribars	1963
Nawiliwili Harbor, Hawaii	18-ton tribars	1959
Lajes Harbor, Azores	16-ton tetrapods	1963, 1970
Santa Cruz Harbor, Calif.	28-ton quadripods	1963
Humboldt Bay, Calif.	12-ton tetrahedrons	Circa 1940
Humboldt Bay, Calif.	100-ton cubes	1950, 1964
Humboldt Bay, Calif.	42- and 43-ton dolosse	1971-1972

* A table of factors for converting British units of measurement to metric units is presented on page ix.

PART II: DESIGN PARAMETERS

Types and Purposes of Structures Using Concrete Armor Units

4. Coastal structures may be classified in accordance with their purpose, i.e., by functional considerations, or by their construction characteristics. Classified according to purpose, they may be breakwaters, groins, jetties, seawalls, and revetments. The primary purpose of breakwaters is to protect harbors, anchorage areas, reaches of shoreline, and basin areas from wave action; jetties serve to confine a stream of tidal flow, maintain a channel, or stabilize an inlet location; groins are provided to control the rate of littoral transport; seawalls are structures separating land and water areas, and are designed to prevent erosion due to wave action--thus, they are essentially bulkheads on the land side and breakwaters on the sea side; and revetments consist of a facing of stone, concrete, or other suitable material constructed to protect a scarp, embankment, or shore structure against erosion by wave action or currents. Some types of construction are better suited to structures of a particular purpose; for example, breakwaters and jetties are often rubble-mound structures. On the other hand, groins may be of almost any type of construction, the choice of construction type being determined by the availability of materials and the economics of the particular installation. In addition to coastal structures, concrete armor units have been used in several non-Corps earth-fill dams for protection against wave erosion on the reservoir side of the dam and, in one instance, against ocean wave attack on the seaward side of the dam.

Factors Influencing Design

5. The design of coastal structures to insure their stability at a minimum of cost is complicated because they are subjected to the forces of wave action. The action of short-period waves with periods up to about 20 sec and heights up to about 50 ft is considered in this

report. The magnitude and distribution of wave forces on rubble-mound type structures vary with the geometry of the structure, the deepwater wave characteristics, the depths of water along the seaward toe of the structure, the bottom configuration (depth contours) seaward of the structure, and the specific weight of water in which the structure is situated.

Depths at structure

6. Two of the primary factors influencing wave conditions at a structure site are the water depth and the bottom contours in the general vicinity of the structure. Generally, the depth will determine whether a given structure is subjected to breaking, nonbreaking, or broken waves for a particular design-wave condition. Also, the height of a given deepwater wave after it arrives at the structure site is dependent on the shoaling coefficient (a function of the depth to wavelength ratio at the structure) and the refraction coefficient (a function of the relative depth and the bottom contours in the vicinity of the site). Variations in depth usually result in changes in the wave conditions; consequently, in evaluating the marine environment at a particular site, a range of possible depths may warrant investigation to insure that those depths that produce the most severe conditions are found. The severity of the conditions selected for design purposes may also depend upon the purpose of the structure and the consequences of its failure. Obviously, the design conditions for a structure that cannot be permitted to fail must be more severe than the design conditions for a structure whose failure would not result in extreme economic loss. Variations in depth may be the result of tidal fluctuations, storm surge (termed wind setup in confined bodies of water such as lakes), anticipated scour at a particular location during storm conditions, and seiching. The depth at a particular place and time may be due to any combination of the above components. It should be noted that, in some instances, the critical design condition may not necessarily occur at the greatest total water depth. Variations in depth along the length of a structure must be considered since wave conditions will consequently vary along the structure, being perhaps more critical

at one location than at another. Also, in some instances offshore shoals located a considerable distance from the structure may focus wave energy, thus dramatically increasing wave heights at the structure site.

7. Tidal data for selected coastal locations in the United States can be obtained from "Tide Tables" published annually by the National Ocean Survey, National Oceanographic and Atmospheric Administration (NOAA). Hydrographic charts are also published by the National Ocean Survey and by the U. S. Navy Oceanographic Office. These charts often use limiting assumptions for the sake of navigation which may render them inadequate for use in wave refraction studies. The most detailed bathymetric information is contained in the actual "boat sheets" which can be obtained directly from the NOAA in Washington, D. C. Generally, these sheets should be used in compiling bathymetric maps for wave refraction analyses. Detailed surveys may also be needed for specific areas of interest.

Design wave

8. The physical factors affecting wave conditions at a site are the exposure of the site to waves generated seaward of it and the meteorological conditions in the wave generating area. In some cases wave data may be available at or near a given site, or may be obtained from an appropriate wave measurement program. More often, however, deepwater wave conditions must be determined from synoptic meteorological data or simultaneous synoptic meteorological observations and transformed to conditions at the site by refraction, shoaling, and diffraction analyses. The result of such a study is a statistical description or distribution of wave characteristics generated by the meteorological conditions. The selection of a design-wave height from the distribution of heights obtained from the study depends upon the purpose of the structure and the economic, social, and environmental consequences of damage. The design wave for a rubble-mound type structure can be a lower wave from the wave-height distribution (usually the significant wave) since any failure that may occur due to higher waves in the wave train is progressive and the displacement of several individual armor units will not result in the complete loss of protection. In

fact, some of the displaced units may rekey themselves and provide increased stability. At times, the largest wave that can attack the structure is selected as the design wave. Determination of the maximum wave that can attack a given structure must take into account the limitations due to water depth and the structure itself. When periodic waves advance up an unobstructed sloping bottom, they eventually become unstable and break; and the limiting height and the depth of water at breaking are functions of the beach slope and the deepwater wave steepness. For periodic waves in water of constant depth, the depth and height at breaking are related only to the deepwater wave steepness. For solitary waves in constant depth, the breaking criterion is independent of the deepwater wave steepness, and according to McCowan[1] the maximum height at breaking is

$$H_b = 0.78 d_b \tag{1}$$

According to Keulegan and Patterson,[2]

$$H_b = 0.73 d_b \tag{2}$$

Equation 1 has been used for periodic waves in many instances for water of constant depth, or when the bottom slope is small, and $d/L \leq 0.1$. For periodic waves in water of constant and finite depth, Miche[3] gives the maximum wave steepness, as a function of d/L, as

$$\left(\frac{H}{L}\right)_{max} = 0.143 \tanh \frac{2\pi d}{L} \tag{3}$$

Also, for other than flat or very small slopes, the maximum height at breaking and the breaking depth are functions of the beach slope.[4] However, when the sloping bottom is terminated at a particular depth of water by a barrier, such as a rubble-mound breakwater, the situation is changed; and the maximum wave that can attack the structure is not only a function of the wave steepness and the depths of water seaward of the structure but is also limited to some extent by the shape and absorption-reflection characteristics of the structure itself. Tests have been conducted (Jackson[5]) for rubble-mound breakwaters in which the largest waves that could attack the structure were obtained for quarrystone and tribar armor units as a function of the breakwater slope, the beach slope, and the d/L ratio of the waves at the position of the structure (breakwater slopes of 1 on 1-1/2 and 1 on 3; beach slopes of 1 on 10, 1 on 50, and flat; and d/L ratios from 0.05 to 0.5). Until additional test data are available covering the complete ranges of beach slope, breakwater slope, types of cover layers, and breakwater geometries, it will be necessary to perform individual model studies or to estimate the design-wave height by use of equations 1 and 2 and test data similar to that of Iversen[4] and Jackson.[5]

9. In summary, the factors influencing the choice of a design wave from the forecast wave environment at the site are (a) the type of structure, (b) the purpose of the structure, (c) the level and frequency of damage permissible, (d) the economic and social consequences of failure, and (e) the cost of subsequent repair to the structure.

PART III: HISTORY OF ARMOR UNITS

10. From the earliest Roman times, breakwaters were constructed of masonry with keyed or close-fitting joints; and the construction methods changed but little until the present century. For a large percentage of the early breakwaters, records indicate eventual failure at one time or another. Early in the present century, apparently engineers began to realize, based on experience, that randomly placed blocks, boulders, or quarrystones helped to dissipate wave energy; and it became common practice to construct sloping-faced rubble breakwaters using these materials in the protective cover layers. Rubble-mound breakwaters with the armor units randomly placed in multiple layers have an important advantage over both the sloping-face and vertical-wall types where the armor units are cubes or rectangular blocks placed one layer thick in a regular, tight-fitting manner with a minimum of voids. Considerable damage (displacement of armor units) to the cover layers of rubble-mound breakwaters with randomly placed armor units can occur due to wave action without destruction to the point that the structure must be completely rebuilt. Also, if wave damage occurs of sufficient magnitude to require replacement of armor units, the stability of the structure after repair is usually increased.

11. The primary problem with respect to the design of rubble-mound breakwaters was that, until about the middle of the present century, the phenomena by which waves dislodge armor units from the sloping face of the structure were not understood to the extent that breakwaters could be designed so as to be stable under the attack of waves of given dimensions. Thus, it was not possible to determine the advantages and disadvantages of different shapes of armor units and different placement techniques. It was not until 1933 that the first formula for estimating the required weights of armor units for rubble-mound breakwaters was published. This was the formula derived by the Spanish professor D. Eduardo de Castro.[6] The first formula generally accepted by the engineering profession was developed by another Spanish engineer, Ramon Iribarren Cavanilles, and was published in 1938.[7] The Iribarren

formula contained a shape coefficient and a friction coefficient. The friction coefficient was assumed to have a value of 1.0; values of the shape coefficient, which was said to be independent of the face slope of the structure, were determined by Iribarren for concrete blocks and natural stones. These values were determined by noting the stability characteristics of actual breakwaters that had been attacked by storm-wave action.

12. During the period 1942-1950, a series of small-scale hydraulic model tests of rubble-mound breakwaters were conducted at the U. S. Army Engineer Waterways Experiment Station (WES)[8] in which the Iribarren formula, and a similar formula developed by Epstein and Tyrrell, were investigated. Concrete cubes and simulated quarrystone armor units were used in these tests. It was found that the shape coefficient in these formulas varied appreciably with both the shape of armor unit and the sea-side slope of the structure. In 1951, a comprehensive series of tests on rubble breakwaters was begun at WES for OCE as an item of the Civil Works Investigations (ES 815, Stability of Rubble-Mound Breakwaters). At first, the tests were concerned with the evaluation of the shape coefficient in a revised and more general form of Iribarren's formula for quarrystone armor units. A new shape of concrete armor unit, the tetrapod, was developed at the Laboratoire Dauphinois d'Hydraulique Ets. Neyrpic, Grenoble, France, in 1950,[9] and it was intended to determine shape coefficients in the revised Iribarren formula for this unit and the cube in addition to the quarrystone armor unit. However, preliminary tests showed that the friction coefficient in Iribarren's formula varied appreciably with the shape of armor unit and the method of placing the armor units in the cover layer; that values of the friction coefficient, as measured by tests using model armor units, varied over a considerable range; and that for steep breakwater slopes, small variations in the friction coefficient cause large variations in the calculated shape coefficient. This latter finding is important because the use of concrete armor units of special shape is more apt to be economically feasible for the steeper breakwater slopes. Thus, the attempt to use Iribarren's formula to correlate the test data was abandoned.

The ES 815, Stability of Rubble-Mound Breakwaters, tests were later resumed, and a new formula similar to that of Iribarren, without a friction coefficient and with cot α (where α is the angle of the breakwater slope, measured from the horizontal) as the angle function, was developed by Hudson.[10,11] This formula, based on the ES 815 test data and a more general approach with respect to the variables involved in the phenomena of waves attacking a rubble breakwater, in conjunction with experimental coefficients determined for quarrystone and different shapes of concrete armor units, has been used by the Corps of Engineers during the last decade as an aid in the design of rubble-mound breakwaters. Since 1950, a considerable number of concrete armor-unit shapes have been developed throughout the world. A majority of these armor-unit shapes are listed in table 1, and figs. 1-27 show the shapes of a majority of the units listed.

PART IV: HYDRAULICS OF COVER-LAYER DESIGN

Development of Stability Formula

13. The stability of rubble-mound breakwaters depends on a protective cover layer of armor units, the required weight of which depends on several variables but primarily on the heights of the largest attacking waves. The armor units may be natural rocks or quarrystones, or concrete units of special shape, as discussed in the preceding paragraphs. There are more than a dozen formulas in the literature for computing the required weights of armor units from known or assumed wave dimensions, specific weights of the armor units and the water, and the seaward slope of the rubble mound. These formulas are not strictly applicable unless the crown elevation of the structure is sufficient to prevent major overtopping of the structure. Most of the formulas now in use have the form

$$W_r = \frac{\gamma_r}{(S_r - 1)^3} \frac{H^3}{K_D f(\alpha)} \tag{4}$$

where

W_r = weight of individual armor units necessary for stability

γ_r = specific weight of the armor unit

S_r = specific gravity of the armor unit relative to the water in which the structure is situated (i.e., $S_r = \gamma_r/\gamma_w$)

γ_w = specific weight of the water in which the structure is situated

H = wave height

K_D = dimensionless stability coefficient representing the effects of all other variables involved in the phenomena that occur when water waves impinge on a rubble-mound structure

α = angle of sea-side slope measured from the horizontal

In the derivation of equation 4, the form of the function $f(\alpha)$ depends on the assumed force diagram for individual armor units at the time of incipient instability. A simple form of equation 4 was derived by Hudson[10,11] by assuming that the drag force on individual armor units is the primary force acting to dislodge the units, and that the buoyant weight of the unit submerged in still water is the primary force tending to prevent movement of the unit. The form of the function $f(\alpha)$ was determined by experiment; the resulting formula is

$$W_r = \frac{\gamma_r H^3}{K_D (S_r - 1)^3 \cot \alpha} \tag{5}$$

Determination of Stability Coefficients

14. A large number of small-scale tests have been conducted on idealized breakwater trunk sections to determine values of K_D in equation 5 for rough and smooth quarrystones and several different shapes of molded armor units.[11,12] Most of these tests were made with large water depths, relative to wave height, using nonbreaking monochromatic waves with no overtopping. Some tests have been made for idealized conical-shaped breakwater heads in relatively deep water and nonbreaking waves. Also, a few such tests have been made in which breakwater trunks in relatively shallow water were subjected to the largest breaking waves that could attack the structure. In addition to K_D values from the above-described tests of idealized breakwater sections, data from which values of K_D can be calculated are available from the results of model studies of existing or proposed full-scale structures (see references 13-17, 19, 40, and 43). Based on the results of tests described above, it has been found that K_D is a function of several variables, the most important of which, for a given rubble-mound breakwater geometry, are (a) shape of armor unit, (b) manner of placing the armor units, (c) portion of breakwater (trunk or head), (d) form of waves (breaking* or nonbreaking), (e) angle of wave incidence, (f) number of layers of armor units, (g) percent damage to the cover layer, i.e. displacement of armor units by the forces of wave action, and

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* When the waves are critically breaking, the exact values of d/L, H/L, and H/d are important.

(h) model scale. The effects of model scale can be made negligible by proper selection of the linear scale of the tests. If some degree of damage can be allowed, larger values of K_D , corresponding to smaller values of W_r , can be used for design. The selected value of K_D , therefore, depends on the degree of risk that the designer can afford to assume. The deliberate selection of a value of K_D such that an estimated amount of cover-layer damage will occur for the selected design wave takes advantage of the fact that the settling and readjustment of the nesting characteristics between armor units, which usually result when moderate cover-layer damage occurs due to wave action, make the remaining cross section more stable than the original section. When the selected value of K_D is larger than that for the no-damage criterion,* consideration should also be given to the possibility of structural damage to individual armor units that may occur due to their movement during periods of severe wave action. A structure designed to resist the forces of wave action corresponding to a moderate storm, but which may suffer some damage without complete destruction during a severe storm, may have a lower total annual cost than one designed to be completely stable for the larger waves. Therefore, damage tests were conducted and the test results have been reported by Hudson[11] and by Jackson.[12] The purpose of the tests was to provide information from which the designer could determine the damage to the cover layer that should be expected if he purposely or inadvertently selected a design-wave height that was smaller than those that actually occurred after construction of the breakwater was completed. These damage tests were conducted in such a way that there was no overtopping of the test structure for the no-damage criterion, i.e., there was no overtopping by the largest wave that would do no damage to the structure. Therefore, the test waves that caused damage to the structure were larger than the test waves that caused no damage and overtopped the structure. This is not the same situation that would occur if it was desired to design

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* The no-damage criterion allows minor displacement of armor units to the extent that the stability of the armor-unit section is not affected.

a structure that would not be overtopped by the selected design wave, but would be damaged by the design wave by an amount that would result in an optimum design based on estimated first costs, plus repair costs, plus interest on the sum of the two. The damage data referred to above can be used to estimate the size armor unit that should be used to obtain a selected amount of damage; but some error will be involved, depending on whether the crown of the structure is such that overtopping occurs in nature to the extent that it occurred in the damage tests.

Physical Characteristics of Armor Units

15. Several of the armor-unit shapes are shown in figs. 1-27. Details of shape, volume, and linear dimensions for some of these armor units are shown in figs. 28-38. The thickness of cover layers and the number of armor units required per unit area of cover layer can be calculated by use of the following formulas when values of the experimentally determined coefficients are available:

$$t = nk_{\Delta}\left(\frac{W_r}{\gamma_r}\right)^{1/3} \tag{6}$$

$$\frac{N_r}{A} = nk_{\Delta}(1 - P)\left(\frac{\gamma_r}{W_r}\right)^{2/3} \tag{7}$$

where

t = the thickness of n layers of armor units of weight W_r and specific weight γ_r

N_r = the required number of armor units for a given surface area A

P = the porosity (void ratio) of the cover layer

Available values of k_{Δ} and P, which have been determined experimentally at WES using small-scale armor units, are listed in table 2. The magnitudes of k_{Δ} and P vary with the shape of armor units and the manner of placing the armor units.

PART V: SELECTION OF ARMOR UNITS AND DESIGN COEFFICIENTS

Selection of Armor Units

16. Based upon the results of tests conducted to date, it is believed that the quarrystone, tetrapod, tribar, and dolos armor units should be considered for use in the design of rubble-mound breakwaters. Although the stability of quarrystone units is not as good as that of the better, special-shaped concrete units, stone is usually more economical when wave conditions are not too severe and when rock of good quality can be obtained locally at reasonable cost. The choice as to whether quarrystone or one of the concrete armor units should be selected depends to a considerable extent upon the availability and cost of the materials, cost of transportation to the breakwater site, and to some extent upon the construction techniques adopted. The major cost items involved in use of quarrystone armor units are the quarrying operations and the selection, transportation, stockpiling, and placing of the units into position on the structure. Similarly, the major costs involved in the use of concrete armor units are the fabrication of the units (concrete material, forms, reinforcement if required, pouring, curing, and stripping of forms), transportation to stockpile, thence to structure site, placing of armor units into position on the structure, and in some cases, depending upon the type of armor unit, the payment of royalties. Should it be necessary to transport the quarrystones considerable distances for a particular project, local casting of concrete armor units may produce significant savings in transportation costs and hence offset the additional costs involved in the production of concrete units. Also, smaller concrete units can be substituted for the larger quarrystones required for comparable protection. Thus, under certain conditions, it is possible that concrete armor units will prove to be the less costly of the two. A suitable basis for comparing quarrystone costs with concrete armor unit costs is the average annual cost of the structure, for equal protection, computed over the selected economic life of the structure.

17. To the extent that the armor units in full-scale structures in the field can be placed in a manner that duplicates the placing techniques used in the small-scale tests, equation 5, used with the appropriate experimental coefficients and cost analyses, can be used to determine (a) when a rubble-mound breakwater can be protected from wave action by quarrystone armor units randomly placed, (b) when cover layers composed of special-shaped concrete armor units are required, and (c) the relative economy, for equal stability, of the types of units tested for which practical placing techniques, which simulate those used in the scale tests, can be developed for the full-scale field units. It is believed that all the placing techniques used in the comprehensive (ES 815) tests can be duplicated satisfactorily in the field except for the uniform placing of tetrapods, quadripods, hexapods, and tetrahedrons. The uniform placing of one layer of tribars can be accomplished satisfactorily above water, but it is difficult and expensive to place these units uniformly below water.

Selection of Stability Coefficients

18. The values of K_D (table 3) for the no-damage criterion, i.e., damage to the cover layer equal to or less than about 5 percent (ratio of amount of material removed from the armor-unit layers to the amount of material originally placed in the cover layer, in percent), were determined by Jackson[12] from small-scale tests on idealized breakwater trunk sections using two layers of armor units (n = 2), nonbreaking waves approaching the test structure at an angle of incidence of 90 degrees, with no overtopping.

19. Table 4 shows the results of damage or safety-factor tests conducted in connection with the tests described in the preceding paragraph. Quarrystone, quadripod, and tribar armor units were tested in the manner described in paragraph 14. Although tetrapods were not tested for the damage conditions, the data obtained for quadripods can also be used for tetrapods since the shape and stability characteristics for these two armor units are nearly identical. In table 4, K_D is

the experimentally determined coefficient in the stability formula (equation 5) corresponding to the different amounts of damage to the cover layers; H_{DW} is the selected design-wave height for the proposed structure; H is the height of the wave that attacks the structure after it is constructed; and D is damage, in percent, as described in paragraph 18. Damage in the amount of about 5 percent or less can be used to design for the no-damage criterion when no overtopping of the structure is expected.

20. There has been little testing in the ES 815 program in which values of K_D were obtained for either breakwater trunks or heads exposed to the attack of large, critically breaking waves (partially breaking waves do not cause forces on rubble-mound structures appreciably larger than those caused by nonbreaking waves). That portion of the ES 815 program concerning breakwater heads subjected to nonbreaking waves is also incomplete. However, tests either are in progress (1973) or are planned to determine values of K_D for the no-damage criteria using rough quarrystone, tetrapod, tribar, and dolos armor units, both breakwater trunks and heads, and critically breaking waves. A considerable number of model studies have been conducted in recent years to determine design data for repair of existing damaged structures, or for the design of proposed new structures, that would be exposed to either nonbreaking or critically breaking waves. For those conditions in which it is considered safe to make final designs without the conduct of hydraulic model studies, the values of K_D listed in tables 5, 6, and 7 may be used.

PART VI: STRUCTURAL BEHAVIOR OF CONCRETE ARMOR UNITS

21. Ideally, concrete armor units should be designed to withstand the stresses that occur during their manufacture, transportation, and placement onto the structure, and those that occur after placement due to movement of armor units during periods of attack by storm waves. It is important that the structural integrity of individual units be maintained because the stability of the structure during wave attack depends primarily on the armor-unit weight, shape, and nesting characteristics. Thus, if armor units are broken due to the forces caused by wave action, the ability of the armor units to resist the wave forces may be reduced, and damage to the cover layer may occur. Presently, however, there is a nearly complete lack of knowledge concerning the forces, and the resulting stresses, that armor units must be designed to withstand; and it is impossible to design the units based on rigid structural analyses of the various static and dynamic loading conditions to which they will be subjected throughout the economic life of the structure.

22. There is no question as to the importance of using a dense, watertight (low permeability), high-quality, high-strength concrete. According to Tyler,[21] the main causes of deterioration in concrete marine structures, in addition to damage due to forces on the individual armor units of rubble-mound structures as mentioned in the preceding paragraph, are freezing-and-thawing attack on the concrete, corrosion of reinforcing steel, and chemical attack on the concrete by sea water. Deterioration from these causes can be greatly minimized or eliminated if the concrete is watertight. Thus, the selection of aggregates, water-cement ratio, and admixtures and the attention given to the procedures of mixing, placing, stripping of forms, and curing of the concrete are extremely important. If possible, the strength of the concrete should not be less than about 5000 psi at 28 days.

23. One of the major considerations in the structural design of concrete armor units is the question of whether they should be reinforced. The results of systematic drop tests of tetrapods to obtain

data concerning their strength characteristics were reported by Danel, Chapus, and Dhaille.[9] Both unreinforced and reinforced tetrapods fractured when dropped from a height of a little over 4 ft onto a concrete slab, but were not damaged by such a fall with a water cushion over the concrete to a depth of about 1/6 to 1/5 the height of the unit. No damage occurred when the tetrapods were dropped from heights up to 10 ft onto a crushed-rock bed.

24. Very little information is available as to the extent of fracturing and breakage that has occurred in the placing of armor units. However, in repairing the Nawiliwili breakwater in 1959, 13 of the 351 unreinforced, 17.8-ton tribars were broken during placement. These units had been placed on scows by crane and were placed on the breakwater from a floating crane operating on the seaward side of the breakwater. Difficulty in placing was encountered because of the wave conditions, which at times caused the end of the boom to rise and fall as much as 5 to 10 ft. Thus, collision of units suspended on the boom sling with those already in place occurred frequently with considerable force. Breakage of about 3.5 percent of 10- and 19-ton, unreinforced tribars occurred at Nassau Harbor in 1968. No breakage was reported of 35-ton, reinforced tribars placed on the head of the same structure. These armor units were also placed by floating plants. Several unreinforced, 25-ton concrete tetrapods were broken at the Crescent City breakwater during a severe storm in February 1960. The units were apparently broken by impact of other objects such as armor stones and pieces of broken tetrapods. They were also severely abraded, apparently by rolling. Several 33-ton, unreinforced tetrapods were broken when they were dislodged from the Kahului breakwater in 1958 by waves about 25 ft in height.

25. As discussed above, some breakage of armor units can be expected if they are placed by floating plant in exposed locations. Since sufficient information is lacking, concerning the forces to which armor units are subjected, to allow accurate stress analyses, and because adequate data from which the percent of armor units that are broken for

different conditions of wave attack are not available, it is not possible at this time to make rational judgments concerning the necessity for armor-unit reinforcement.

PART VII: ADVANTAGES OF ARMOR UNITS WITH HIGH SPECIFIC WEIGHT

26. In addition to the structural advantages of using high-density concrete in the fabrication of special armor-unit shapes, as mentioned in paragraph 22, there is a considerable increase in the stability of any armor unit with an increase in specific weight (see equations 4 and 5). From these equations, and assuming constant values of H , $\cot \alpha$, γ_w , and K_D ,

$$\frac{(W_r)_1}{(W_r)_2} = \frac{(\gamma_r)_1}{(\gamma_r)_2} \frac{\left[(\gamma_r)_2 - \gamma_w\right]^3}{\left[(\gamma_r)_1 - \gamma_w\right]^3} \tag{8}$$

An illustration of this effect is shown in fig. 39 for values of $(\gamma_r)_1$ and (γ_w) of 150 and 64 pcf, respectively, and values of $(\gamma_r)_2$ from 120 to 176.5 pcf. $(V_r)_1$ and $(V_r)_2$ refer to the volumes of the two armor units. The curve of fig. 39 shows that if the specific weight of concrete is increased from 150 to 160 pcf, the required weight for stability of the armor unit under the attack of a given design wave is decreased about 23 percent. The corresponding decrease in the armor-unit volume would be about 29 percent.

27. The effects of inertial forces were investigated by Brandtzaeg[18] who found that the effects of specific weight of the armor unit cannot be ignored when S_r is either unusually large or small. His studies indicated that the specific weight term of equations 4 and 5 should be modified to include a variable term in the denominator. With this modification, equation 5 becomes

$$W_r = \frac{\gamma_r H^3}{K_D (S_r - \phi)^3 \cot \alpha} \tag{9}$$

The results of Brandtzaeg's investigation were not conclusive, and he recommended that further studies be made. His test results indicated, however, that ϕ could vary from about 0.37 to 1.05, and theoretical considerations led him to believe that values of ϕ exceeding those found in the experiments may occur. Until a comprehensive investigation has been conducted to determine the effects of specific weight of armor unit on the stability of rubble breakwaters, the Hudson formula (equation 5) should be used. The range of values of S_r, i.e. values of $(\gamma_r)_2 / (\gamma_w)$ from 1.88 to 2.76 as indicated in fig. 39, is not believed to be sufficient to affect appreciably the accuracy of armor-unit weight determinations.

PART VIII: CASTING OF ARMOR UNITS

28. The casting of concrete armor units is a mass production effort that requires careful design and control of the concrete used in the armor unit. Appropriate Corps of Engineers specifications for concrete for use in sea water should be used. The Pacific Ocean and South Pacific Divisions are experienced in armor-unit casting, curing, and stockpiling and will provide examples of plans and specifications for this work upon request. A decision must be made as to the ownership of the armor-unit forms. In some instances the forms have become the property of the Government, e.g., those used for the 25-ton tetrapods in breakwater construction at Crescent City, Harbor, Calif. Also, in some instances the forms become the property of the contractor, e.g., those used for the 25-ton quadripods at Santa Cruz Harbor, Calif., the 35- and 50-ton tribar forms used during the repair of the breakwater head at Kahului Harbor, Hawaii, and the 42-ton dolos forms used during repair of the Humboldt Bay, Calif., jetties.

29. Consideration must be given to appropriate curing of armor units prior to the removal of forms. The early removal of forms may cause cracks in the concrete both from the stripping operations and subsequent handling of the armor units. A common practice is to use a sufficiently greater number of bottom forms than top forms so that the top form can be removed while the units remain in the lower form for additional curing. Curing compounds are generally used to prevent loss of moisture, and curing duration is an important consideration. The units should not be moved until sufficient strength of concrete has developed to avoid cracking during handling and stockpiling operations.

PART IX: DESIGN OF RUBBLE-MOUND STRUCTURES

Example of Design Procedures

30. The ideal final design of a breakwater is a cross section that will meet the requirements of the structure at a minimum of costs. Thus, for optimum design, the total estimated costs must be used. The total costs include those for design and construction, and the capitalized costs of repair of estimated damages to the structure due to waves larger than the selected design wave. The total estimated costs should also include the repair of vessels and harbor structures, and any slowdown costs due to ship-surge problems that may occur as the result of the estimated damages to the breakwater of such severity as to reduce its ability to protect the harbor from storm-wave action. The problems involved in the design of an optimum breakwater section are truly formidable. For relatively small, inexpensive structures the design and cost-estimate studies are usually made using all available information in the literature. For large, important, and expensive structures it is common practice to conduct (a) a three-dimensional, harbor wave-action model study to determine the optimum length and orientation of the proposed breakwaters and the optimum width and orientation of the navigation opening, and (b) a two-dimensional, breakwater stability model study to determine the optimum cross section of the structure. For very large waves and unusual and complicated shapes of breakwater sections, it is sometimes necessary to conduct three-dimensional, stability model studies of the proposed breakwater. This situation is more apt to occur for breakwater heads. For this example, a breakwater trunk section is selected and it is assumed that the problem does not require a model study, that a rubble-mound type of breakwater is judged to be the most practical, that the scarcity of suitable quarrystone indicates the need of a concrete armor unit, and that the structure would be designed for the no-overtopping and no-damage criteria.

Design conditions

31. Bottom material is sand; bottom slope seaward is about 1:50;

waves approach the structure at an angle of incidence of about 90 degrees; the selected design wave is 30 ft in height with wave periods from 11 to 13 sec, i.e., in the equation $H_{DW} = K_s K_r K_{fp} H_o$ (where H_{DW} is the design-wave height in depth d at the structure site, H_o the wave height in deep water, and K_s, K_r, K_{fp} are the shoaling, refraction, and friction-percolation coefficients, respectively) the storm conditions and the bottom contours seaward of the proposed structure are such that $H_{DW} = 30$ ft, if this size wave can reach the structure after it is constructed; water depth at structure site is 50 ft referred to mean sea level (msl); spring tide is +5 ft msl; mean low water is -3 ft msl; the design hurricane surge is 7 ft above the astronomical tide existing at time of the hurricane; the reflection of waves from the seaward side of the proposed breakwater is not critical with respect to navigation, i.e., the structure does not need to be designed as a wave absorber on the seaward side; the armor units will be molded of concrete with a specific weight of 155 pcf; the structure will be situated in sea water with a specific weight of 64 pcf; and the specific weight of the underlayer stone is 165 pcf.

<u>Design procedures and calculations</u>

32. <u>Selection of armor-unit shape.</u> The Corps of Engineers has used cubes, tetrapods, quadripods, tribars, and dolosse for armor units when good quality rock was not available locally or when the design wave was so large that stone of sufficient size could not be obtained from available quarries. The tribar is slightly more stable than the tetrapod and quadripod (K_D of 10.4 versus 8.3 for $n = 2$) and is royalty-free for the Corps of Engineers. The dolos unit is also royalty-free and the stability coefficient for $n = 2$ is considerably larger than that of the tribar. The dolos unit has not been tested by the Corps of Engineers as thoroughly as some of the other armor units, and additional testing is necessary before final recommendations for the selection of K_D can be given. However, it is believed that sufficient testing has been done to show that the dolos is the superior unit. Thus, the dolos unit is selected.

33. <u>Selection of $\cot \alpha$ for the sea-side slope.</u> Since the size

of armor unit and the wave runup increase as $\cot \alpha$ decreases, and volume of material required to construct a breakwater increases as $\cot \alpha$ increases, the optimum slope must be determined by successive trial calculations. However, the steepest slope selected should not have a value of $\cot \alpha$ less than that which will not fail by a landslide type failure. Adequate data are not available at this time for all of the better types of concrete armor units; but those test data that are available indicate that the sea-side slopes of rubble-mound structures using randomly placed tribars with $n = 2$ should not be steeper than about 1 on 1.5, and that such slopes using dolosse should not be steeper than about 1 on 2. Thus, for this example a sea-side slope with $\cot \alpha = 2$ is selected.

34. Selection of $\cot \alpha$ for the harbor-side slope. Comprehensive tests to determine the optimum harbor-side slope have not, as yet, been conducted. However, it is common practice to use slopes from $\cot \alpha =$ 1.25 to 1.5 . The angle of repose for dumped stones is about 1 on 1.25, and this slope is used when the structure is constructed by the end-dump method, and there is only moderate wave action and minor overtopping. When the structure is to be designed for large waves and moderate overtopping, a harbor-side slope of from 1 on 1.33 to 1 on 1.5 is usually used. For large amounts of overtopping it is necessary to design the upper portion of the structure so as to obtain a trajectory of the overtopping water such that the resulting water jets will not impinge on the rear slope without a water cushion. For this situation the steepest rear slope that will be stable is preferred. When the crown and rear slope of a rubble-mound structure will be subjected to large waves, and when the crown elevation is to be such that considerable overtopping will occur, it is usually best to obtain the optimum design by use of a hydraulic model investigation. For the present example, waves are relatively large, the overtopping is negligible, and the end-dump method of construction will not be used. Thus, a rear slope of 1 on 1-1/3 is selected.

35. Selection of K_D . The presently recommended values of K_D for two layers of dolos armor units randomly placed on a breakwater trunk for the no-damage and no-overtopping conditions, and $\cot \alpha = 2$,

are 22.0 and 25.0 for breaking and nonbreaking waves, respectively. Thus, to select the proper value of K_D, it is necessary to determine whether the waves that attack the structure are breaking or nonbreaking. The preliminary selection of the 30-ft design-wave height (paragraph 31) was done on the basis of the deepwater wave statistics; refraction, shoaling, and friction phenomena; and equation 3. For the given conditions it was determined that the 30-ft waves would be nonbreaking at the position of the structure before the structure was constructed. Jackson's data[5] for tribar armor units can be used to determine whether the waves will be breaking or nonbreaking with the structure in place, and the corresponding maximum wave height that can attack the structure (no such data are available for dolosse, but the tribar data can be used with minor error by interpolation of the data in figs. 40 and 41). This determination is made as follows: since the largest waves that can reach the structure will be at high tide, and since the area is assumed to be subject to hurricanes, the selected water depth will be $d = 50 + 5 + 7 = 62$ ft. The design-wave periods range from 11 to 13 sec; therefore, d/L_o varies from 0.100 to 0.0717, d/L varies from 0.141 to 0.116, and L varies from 440 to 535 ft. From figs. 40 and 41 the largest interpolated values of H/L, for values of d/L of 0.141 and 0.116, are 0.074 and 0.066, respectively. The corresponding largest critically breaking waves that can attack the structure are 32.5 and 35.3 ft, respectively. Thus, since the largest waves available, considering the deepwater wave statistics and the refraction and shoaling phenomena, are 30 ft in height, the selected design-wave height, $H_{DW} = 30$ ft, will result in the attack of waves that are partially breaking; and for this condition the value of K_D corresponding to the nonbreaking wave condition can be used, i.e., $K_D = 25$.

36. Selection of crown elevation. Since this structure will be designed for the no-overtopping condition, which from a practical standpoint means that only minor overtopping can be tolerated for the selected design-wave and still-water level conditions, the crown elevation will be selected such that $R = h$ (R is the height of runup above the design still-water level and h is the elevation of the crown with

respect to the same still-water level). Thus, for this structure the crown elevation would be $h = (12 + R)$ ft above msl. For nonbreaking waves, wave runup on rubble-mound structures is primarily a function of H/L, $\cot \alpha$, and the porosity of the armor-unit cover layer. There is some evidence (tests in progress at WES, May 1973) that runup for critically breaking waves can be considerably larger than for nonbreaking or partially breaking waves, depending on the wave form (H/L and d/L). More tests are needed to determine values of runup for critically breaking waves. Considerable small-scale test data are available for use in estimating wave runup,[11,12,22] but little data are available for the dolos armor unit. Merrifield and Zwamborn[13] found that R/H for dolosse varies from 0.83 to 0.90 for their test conditions ($\cot \alpha = 1.5$ and nonbreaking waves), compared with 1.00, 0.90, and 0.98 for rectangular blocks, tetrapods, and tetrahedrons, respectively. Foster and Gordon[20] found that the value of R/H for dolosse was about 0.85 for values of $\cot \alpha$ from 1.25 to 2.0. However, the authors caution that care should be taken in applying this value for design because of the limited range of wave periods used. The fact that R/H did not vary with $\cot \alpha$ in their tests also makes the runup data obtained by Foster and Gordon suspect. Jackson's data for randomly placed tribars, $n = 2$ (see fig. 31 of reference 12), show that, for these units with $\cot \alpha = 2$, nonbreaking waves, and an $H/L = 0.055$, the value of $R/H = 0.9$. This value should be slightly conservative for dolosse. Based on the above-described considerations, a value of $R/H = 0.9$ was selected. The corresponding value of R is 27 ft for the 30-ft design wave. Thus, the selected crown elevation is $h = (12 + 27) = +39$ ft, referred to msl.

37. <u>Selection of armor-unit weight W_r</u>. The required armor-unit weight is determined using equation 5 for the following conditions as assumed or determined in the above paragraphs 32-36: $H_{DW} = 30$ ft, $\gamma_r = 155$ pcf, $\gamma_w = 64$ pcf, $K_D = 25$, and $\cot \alpha = 2.0$. Then

$$W_r = \frac{155(30)^3}{25\left(\frac{155}{64} - 1\right)^3 2.0} = \frac{155 \times 27,000}{25 \times 2.86 \times 2.0}$$

$$= 29,300 \text{ lb} = 14.65 \text{ tons}$$

Thus, a 15-ton unit is selected.

38. Thickness of armor-unit layer. The thickness of the armor-unit layer can be determined from equation 6 and the shape coefficient for dolosse presented in table 2 ($k_\Delta = 1.15$). Thus, with $n = 2$,

$$t = 2 \times 1.0 \left(\frac{30,000}{155}\right)^{1/3} = 2(193.6)^{1/3}$$

$$= 2 \times 5.8 = 11.6 \text{ ft}$$

and for design of the section a value of 11.6 ft is used.

39. Number of armor units required. The number of armor units per square foot of surface area can be calculated by use of equation 7, together with the values of $k_\Delta = 1.00$ and $P = 0.63$ from table 2, as follows:

$$\frac{N_r}{A} = 2 \times 1.0 \times 0.37 \left(\frac{155}{30,000}\right)^{2/3} = 0.74(193.6)^{-2/3}$$

$$= \frac{0.74}{33.5} = 0.0221$$

Thus, 22 armor units would be required per 1000 sq ft of cover-layer surface area of which about 55 percent should be placed in the first, or lower, layer and about 45 percent in the top layer.

40. Width of crown. Unless additional width is required because of the construction procedure, to insure sufficient width for access of construction machinery, etc., or for other reasons, it is common practice to use a minimum width of crown equal to the equivalent of three armor-unit thicknesses. Thus,

$$w_c = 3 \times 1.0 \times 5.8 = 17.4 \text{ ft}$$

41. Weight of underlayer material. It is customary to use quarrystone for the underlayer system. Although a portion of the core material may be composed of sand or clay, or both, in most cases quarry-run material is used. Two, and sometimes three or four, sizes of underlayer stone are used between the cover layer and the core material. The sizes of stone in each successive underlayer should be such that they will not leach through the voids in the immediate upper layers. Thus, the weight of stones in the first underlayer could be about $W_r/20$ if the armor units are also stones. However, the custom is to use $W_r/10$ for the first underlayer of stones to provide larger voids for better nesting of the stone armor units, and to reduce back pressures on the armor-unit cover layer. The weight of the first underlayer of stones can also be $W_r/10$ for most concrete armor-unit shapes. However, as the value of K_D increases, i.e., as the armor-unit shape becomes more efficient, the size of the armor unit decreases for the same design-wave height; and the ratio between the weight of the underlayer stone and the armor-unit weight should be increased. Otherwise, the size of the underlayer stone would become proportionately too large compared with the armor-unit size. Thus, for dolosse, it is believed that a ratio of about $W_r/5$ should be used for the weight of the first underlayer stone (W_1). A ratio of $W_1/20$ can be used for the second stone underlayer (W_2), and $W_2/20$ for the third stone underlayer (W_3), etc.

42. Gradation of underlayer material. The underlayer material can be graded to some extent. The stone in the first underlayer should be graded the least, and logically the gradation of succeeding layers could be progressively more graded, i.e., be composed of a wider range of stone sizes. The following gradations are suggested for use with dolos armor units:

Layer	Armor-Unit and Underlayer Weight and Gradation	Percent of Total Mixture
Cover layer	W_r	100
Underlayer 1	$0.26\ W_r$	25
	$0.20\ W_r$	50
	$0.14\ W_r$	25

Layer	Armor-Unit and Underlayer Weight and Gradation	Percent of Total Mixture
Underlayer 2	$0.015\ W_r$	25
	$0.010\ W_r$	50
	$0.005\ W_r$	25
Underlayer 3	$0.00085\ W_r$	25
	$0.00050\ W_r$	50
	$0.00015\ W_r$	25
Underlayer 4	Quarry-run stones with a minimum of fines to reduce the loss of material by wave action during construction.	

The corresponding gradations for structures using tetrapod, quadripod, or tribar armor units would be as follows:

Layer	Armor-Unit and Underlayer Weight and Gradation	Percent of Total Mixture
Cover layer	W_r	100
Underlayer 1	$0.13\ W_r$	25
	$0.10\ W_r$	50
	$0.07\ W_r$	25
Underlayer 2	$0.0075\ W_r$	25
	$0.005\ W_r$	50
	$0.0025\ W_r$	25
Underlayer 3	$0.000425\ W_r$	25
	$0.00025\ W_r$	50
	$0.000075\ W_r$	25
Underlayer 4	Quarry-run stones with a minimum of fines to reduce the loss of material by wave action during construction.	

43. Thickness of the underlayers. The thickness of the underlayers can be determined from equation 6 and the shape coefficient for

stones presented in table 2 ($k_\Delta = 1.15$) using the 50 percent weight. Each underlayer is composed of two layers of stones ($n = 2$); thus, for 15-ton dolos armor units

$$t_1 = 2 \times 1.15 \left(\frac{6000}{165}\right)^{1/3} = 2.3(36.4)^{1/3} = 7.6 \text{ ft}$$

$$t_2 = 2 \times 1.15 \left(\frac{300}{165}\right)^{1/3} = 2.3(1.82)^{1/3} = 2.8 \text{ ft}$$

t_3 (not required)

44. Details of cross section. The final details of the design cross section are determined on the drafting board. Ordinary 10 by 10 per inch cross-section paper with a grid size of 16 by 20 in. is very good for this purpose. With the bottom elevation plotted on the sheet, the design is started by laying off the width of the crown at the required elevation (in this case, a width of 17.4 ft and a crown elevation of +39.0 ft msl). The sea-side and harbor-side slopes (1:2 and 1:1-1/3, respectively) are then added, after which the thicknesses of armor-unit cover layer and the succeeding underlayers are drawn until the 50 percent weight of the core material is equal to or greater than the 50 percent weight of the bottom underlayer stone. For nonbreaking waves, the usual practice is to extend the armor units in the cover layer downslope to an elevation on the sea side below a selected low-water datum an amount equal to the height of the selected design wave for the low-water condition. The low-water condition in this example is -3 ft msl, corresponding to a depth of 47 ft. The selected design wave for this condition, determined in the same manner as that for the high-tide conditions using equation 3 and figs. 40 and 41, would be a breaking wave about 29 ft in height. Thus, the dolos armor units would be extended some distance below that mentioned above for nonbreaking waves. Considering the need for a toe-protecting blanket, and the practical difficulties of placing a stone armor-unit section below the dolos units between a -H elevation and the top elevation of the toe blanket and its filter layer, the cross section would be designed

as shown in fig. 42. For purposes of this example, the harbor-side slope design assumed that there would be very little wave action on the harbor side; thus, the back-slope cover layer was terminated 5 ft below the selected low-water level. If the back slope will be exposed to waves of appreciable magnitude, it should be designed accordingly.

Use of Hydraulic Models

45. The use of hydraulic models is a standard procedure in the design of proposed rubble-mound breakwaters and the repair of damaged structures to determine the most economical designs that will provide the required degree of stability. Model studies also afford a means of checking the performance of rubble breakwaters with respect to wave transmission and overtopping. Although model tests should be used to supplement rather than replace proven theory and the good judgment and experience of design engineers, they often indicate design changes that save substantial amounts in construction costs and provide valuable information as to the efficiency of untried design and construction procedures.

46. The stability of rubble-mound structures subjected to short-period wave action, especially when the waves break directly on the face slope and when the structure is to be designed for appreciable overtopping, varies considerably with the shape of the cross section. Changes in the shape and elevation of the crown are critical with respect to the overall stability of the structure, in both deep and shallow water, when appreciable overtopping of the design waves is allowed. When the water is shallow and breaking waves occur, the manner in which the armor units are placed at the toe of the slope is important; and when there is a considerable range of tide, the selected design wave for high-tide conditions may not be satisfactory for use for low-tide conditions. Therefore, whenever the cost of a breakwater or jetty is sufficient to justify a hydraulic model investigation, it is usually desirable to perform a model study to determine details of the optimum shape of cross section for the selected type of armor unit.

47. Model studies should be made in the survey stage when they are needed to determine a logical plan of improvement. When the model study is to determine design details of an evident plan, it should be performed during the design memorandum stage. EM 1110-1-8100 specifies the procedure for obtaining approval for model studies.

PART X: LEGAL ASPECTS OF USING ARMOR UNITS OF SPECIAL SHAPE

48. Most of the armor units for which patents have been obtained, or applied for, are listed in table 1. The Corps of Engineers has used cubes, quadripods, tetrapods, tribars, and dolos armor units. Of these, only the quadripod, tetrapod, and tribar units are covered by patents, and the Corps of Engineers and other agencies of the U. S. Government can use the tribar unit royalty-free. The Chief of Engineers has negotiated Department of the Army Patent License and Release Contract No. DA-49-129-CIVENG-65-21, dated 3 January 1966, with ESTABLISSEMENTS NEYRPIC. Copies of this contract can be obtained from OCE, DAEN-CWE. The contract is controlling of any uses by or for the Department of the Army of artificial (cast-in-concrete) blocks in configurations identifiable as TETRAPOD and QUADRIPOD. The license agreement includes two patents: (a) U. S. Patent No. 2,766,592, issued 16 October 1956, and replaced by Reissue Patent No. 24,632, dated 14 April 1959; and (b) U. S. Patent No. 2,900,699, issued 25 August 1959. Patent Re 24,632 covers the tetrapod (or quadripod) shape per se; Patent 2,900,699 covers certain types of four-section mold forms for casting tetrapods wherein the four sections are identically shaped and interchangeable. Patent Re 24,632 on the tetrapod shape expired on 16 October 1973. However, the total agreement will not expire until 25 August 1976, the date of expiration of Patent 2,900,699 on Neyrpic's specific types of mold form. Therefore, for any projects contemplating the casting of either tetrapods or quadripods prior to 25 August 1976, OCE should be consulted regarding the applicability of the license agreement, ATTN: DAEN-CWE-H and DAEN-GCP. Paragraph 1 of Schedule A of the Patent License and Release Contract, and EM 1110-2-2904, Appendix III, plates 22 and 24, can be consulted for further description of the tetrapod and quadripod armor units. Paragraph 2 of the contract's Schedule A should be particularly noted for compliance whenever the use of tetrapod or quadripod units within your Division is contemplated. The information and technical data required by subparagraphs a and b of paragraph 2 are to

be timely submitted to the contractor by the District Engineer involved to allow for the contractor's review and rendering of advisory comment upon the suitability of any and all such intended uses. Information copies of all such letters of submittal to the contractor (letter only) should be sent to OCE, ATTN: DAEN-CWE and DAEN-GCP. In considering the use of tetrapod or quadripod units for any project, comparative cost estimates should be made for plans utilizing these units, other precast concrete units, and large size stones. Concrete armor units should be used when their costs, including forming and royalty costs, result in the lowest overall project cost. The design should be in accordance with EM 1110-2-2904. Although past use of concrete armor units has been limited primarily to breakwaters and jetties, where wave characteristics and stone cost make concrete armor units economical, they may be used for coastal revetments, earth dams, and embankments.

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Table 1

Types of Concrete Armor Units

Name of Unit	Development of Unit: Country	Development of Unit: Year	U. S. Patent Number	Reference Number
Akmon	Netherlands	1962	None	23
Bipod	Netherlands	1962	None	23
Cob	England	1969	None	24
Cube*	--	--	None	8
Cube (modified)	USA	1959	None	12
Dolos	Rep. So. Africa	1963	None	13
Dom	Mexico	1970	(?)	--
Gassho block	Japan	1967	None	25
Grobbelaar block	Rep. So. Africa	1957	None	26
Hexaleg block	Japan	(?)	None	27
Hexapod	USA	1959	None	12
Hollow square	Japan	1960	3,176,468	25,28
Hollow tetrahedron	Japan	1959	None	25,14,29
Interlocking H-block	USA	1958	None	30
N-shaped block	Japan	1960	3,176,468	25,28
Pelican stool	USA	1960	None	15
Quadripod	USA	1959	None**	12
Rectangular block*	--	--	None	16
Stabit	England	1961	None	17
Stabilopod	Rumania	1965	None	31
Sta-Bar	USA	1966	3,636,713	32
Sta-Pod	USA	1966	3,399,535	32
Stolk cube	Netherlands	1965	3,548,600	33
Svee block	Norway	1961	3,210,944	34
Tetrahedron (solid)	USA	1942	None	12
Tetrahedron (perforated)	USA	1959	None	12
Tetrapod	France	1950	2,766,592	9,12
Toskane	Rep. So. Africa	1966	None	26
Tribar	USA	1958	2,909,037†	12,35
Trigon	USA	1962	(?)	--
Tri-long	USA	1968	None	36
Tripod	Netherlands	1962	None	23

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

** Patent for tetrapods applies also to quadripods.

† Royalty free to agencies of U. S. Government.

The underscored units have been tested, some extensively, at WES.

Table 2

Experimental Values of k_{Δ} and P

Armor Unit	Method of Placing	n	k_{Δ}	P
Smooth quarrystone	Random	2	1.02	0.38
Rough quarrystone	Random	2	1.15	0.37
Quadripod	Random	2	0.95	0.49
Tetrapod	Random	2	1.04	0.50
Tetrapod	Uniform	2	1.05	0.43
Tribar	Random	2	1.02	0.54
Tribar	Uniform	1	1.13	0.47
Modified cube	Random	2	1.10	0.47
Modified cube*	Uniform	1	1.12	0.27
Hexapod	Random	2	1.15	0.47
Hexapod	Uniform	1	1.29	0.43
Perforated tetrahedron	Uniform	2	0.98	0.43
Dolos	Random	2	1.00	0.63
Tri-long	Random	2	0.94	0.40

* Placed in cover layer with the legs parallel to the slope.

Table 3

Values of K_D Obtained From Small-Scale Tests

No-Damage Criterion, Nonbreaking Waves, No Overtopping

Type of Armor Unit	Method of Placing	Number of Layers	K_D Average	K_D Lower Limit
Smooth quarrystone	Random	2	2.8	2.4
Rough quarrystone	Random	2	4.6	4.0
Tetrapod	Random	2	9.4	8.3
Tetrapod	Uniform	2	9.4	8.3
Quadripod	Random	2	9.4	8.3
Quadripod	Uniform	2	9.4	8.3
Tribar	Random	2	11.5	10.4
Tribar	Uniform	1	28.1	25.2
Modified cube	Random	2	11.2	7.8
Modified cube*	Uniform	1	--	11.6
Hexapod	Random	2	10.4	9.5
Hexapod	Uniform	1	--	22.2
Modified tetrahedron	Uniform	2	4.1	2.7
Dolos	Random	2	(Under study at WES)	

* Placed in the cover layer with the leg parallel to the slope.

Table 4

Values of K_D as a Function of H/H_{DW}, D,

and Type of Armor Unit

H/H_{DW}	Range of D, Percent	K_D
	Smooth Quarrystone	
1.00	1-5	2.6
1.08	5-10	3.3
1.19	10-20	4.3
1.29	20-30	5.5
1.41	30-40	7.2
1.54	40-50	9.4
	Rough Quarrystone	
1.00	1-5	4.6
1.08	5-10	5.6
1.19	10-15	7.5
1.27	15-20	9.3
1.37	20-30	11.5
1.47	30-40	14.2
	Quadripod*	
1.00	1-5	8.3
1.09	5-10	10.8
1.21	10-20	14.5
1.32	20-30	19.2
1.41	30-40	23.4
1.50	40-50	27.8
	Tribar	
1.00	1-5	12.0
1.11	5-10	16.4
1.23	10-15	22.4
1.36	15-20	30.1
1.50	20-30	40.7
1.59	30-40	48.2

* Data can also be used for tetrapod armor unit.

Table 5

Recommended* Values of K_D for Design of Structure Trunk

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

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

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

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

Table 6

Recommended* Values of K_D for Design of Structure Head

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

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

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

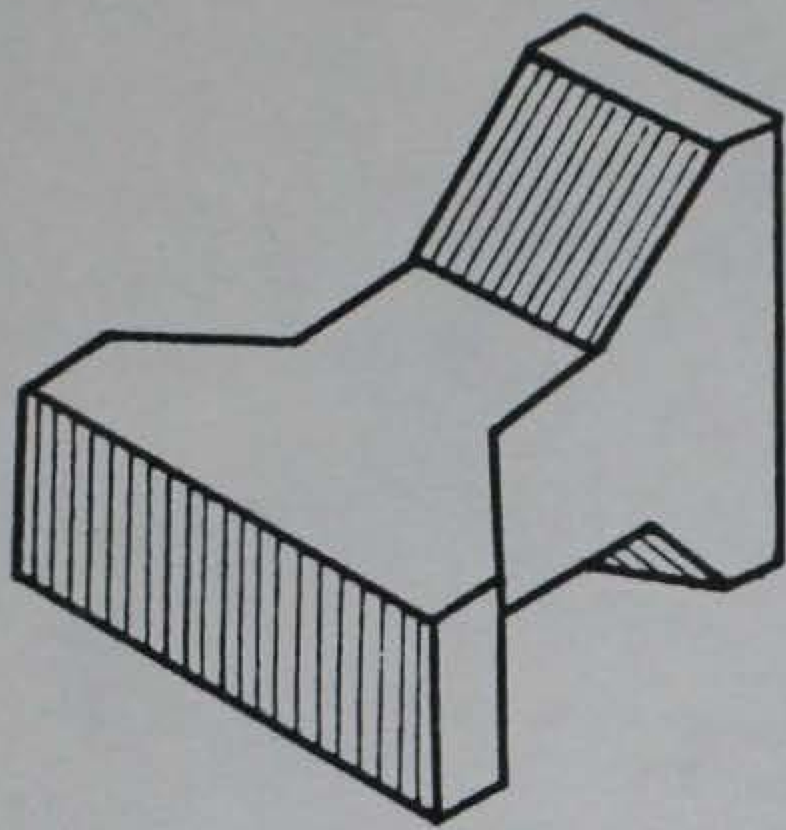
** No data presently available for other armor units.

Table 7

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

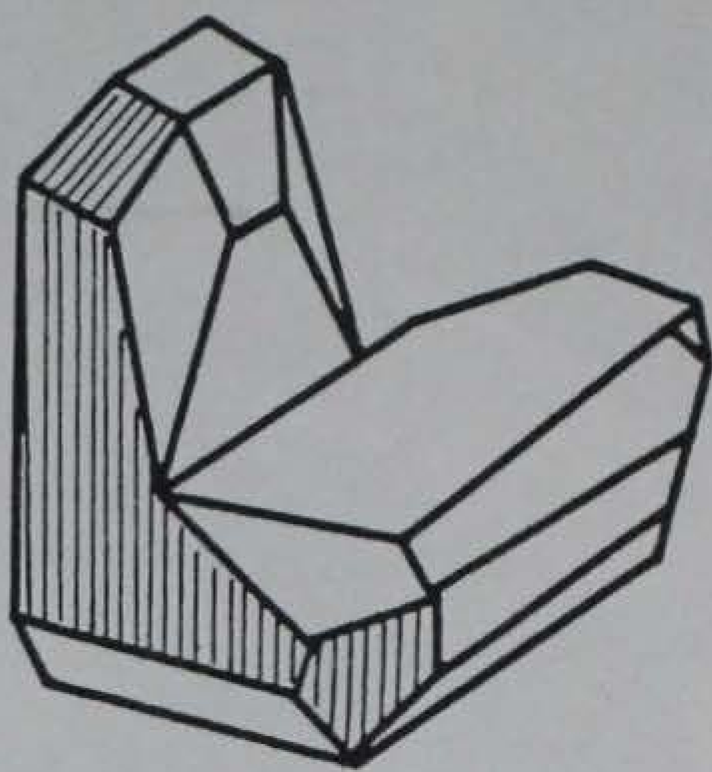
Unit	D, Percent					
	0-5	5-10	10-20	20-30	30-40	40-50
Smooth quarrystone						
H/H_{DW}	1.00	1.08	1.19	1.29	1.41	1.54
K_D	2.4	3.0	4.0	5.1	6.7	8.7
Rough quarrystone						
H/H_{DW}	1.00	1.08	1.23	1.37	1.47	
K_D	4.0	4.9	7.3	10.0	12.4	
Quadripod and tetrapod						
H/H_{DW}	1.00	1.09	1.21	1.32	1.41	1.50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

* See paragraph 14.



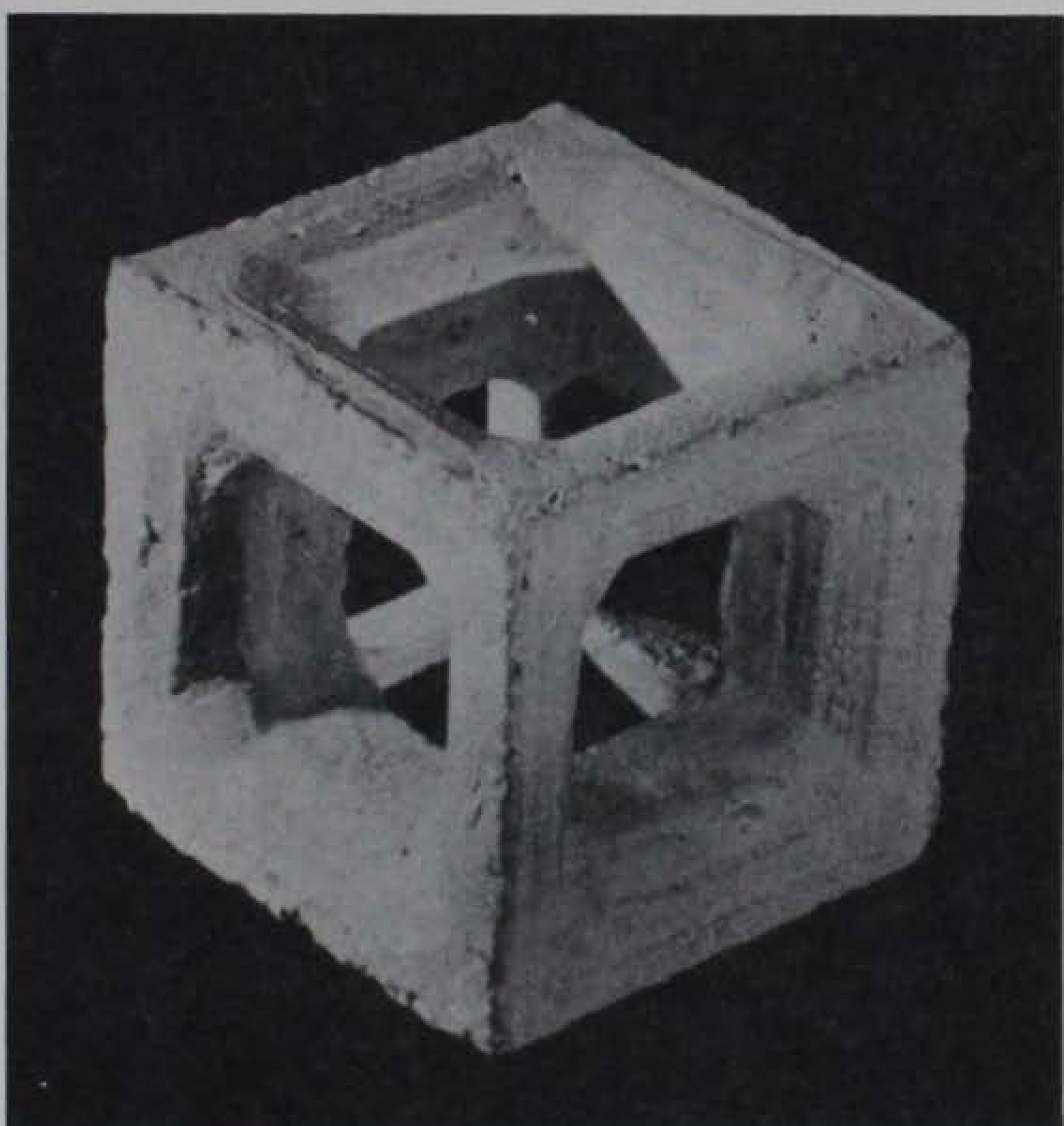
(From Paape and Walther, 1963[23])

Fig. 1. Akmon



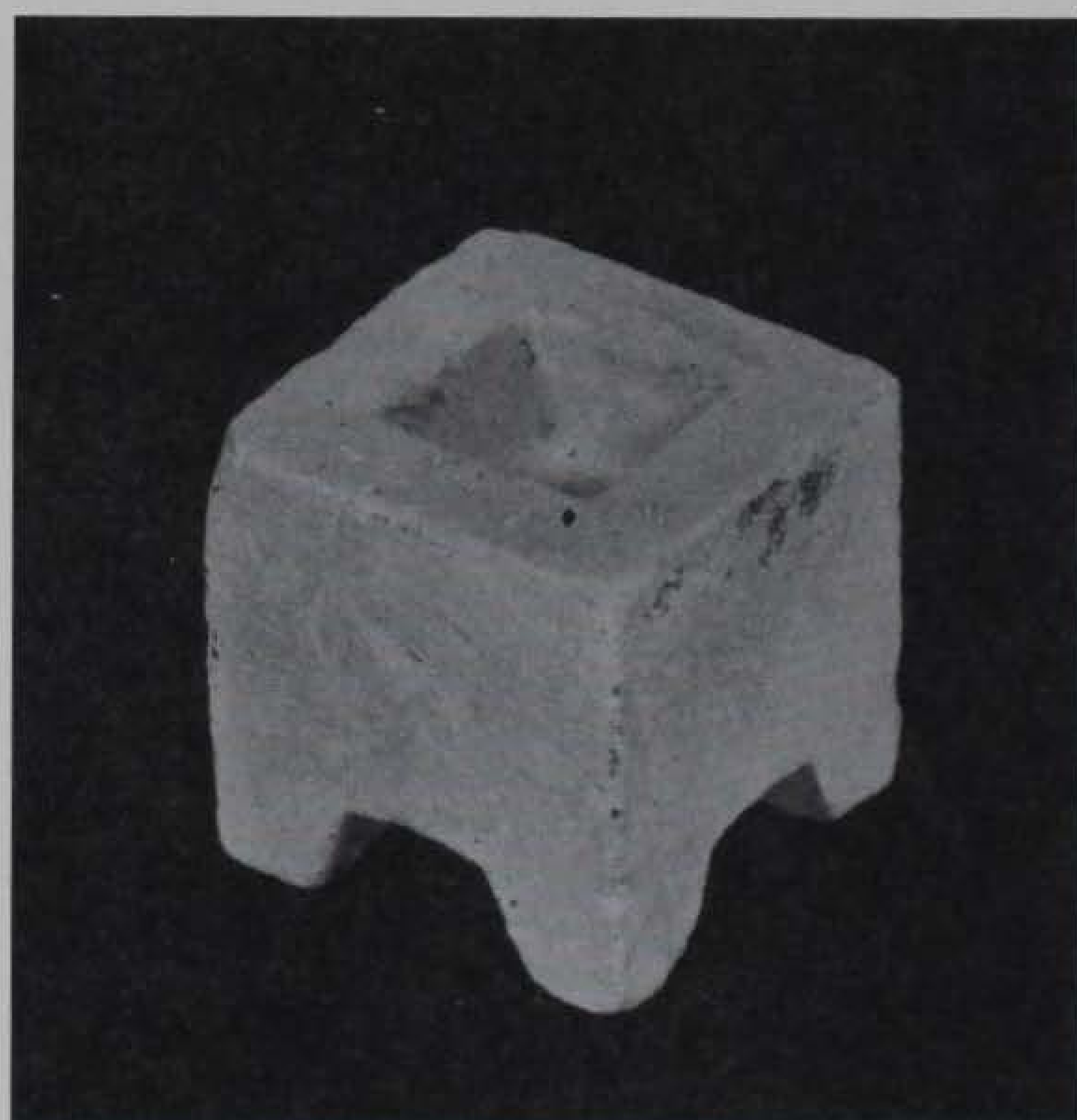
(From Paape and Walther, 1963[23])

Fig. 2. Bipod



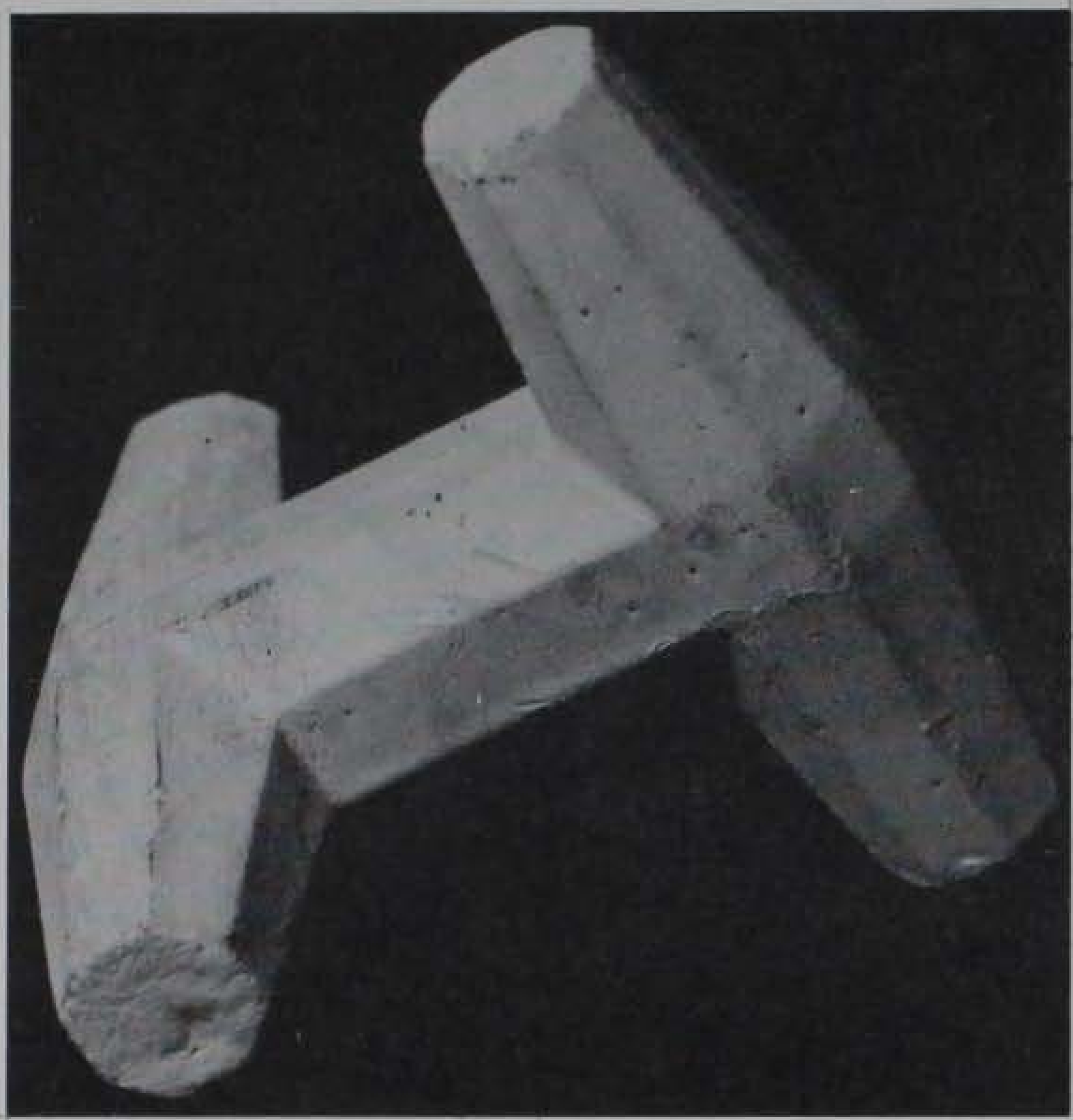
(Courtesy of Coode and Partners, Consulting Engineers, 2 Victoria St., London, S.W. 1)

Fig. 3. Cob



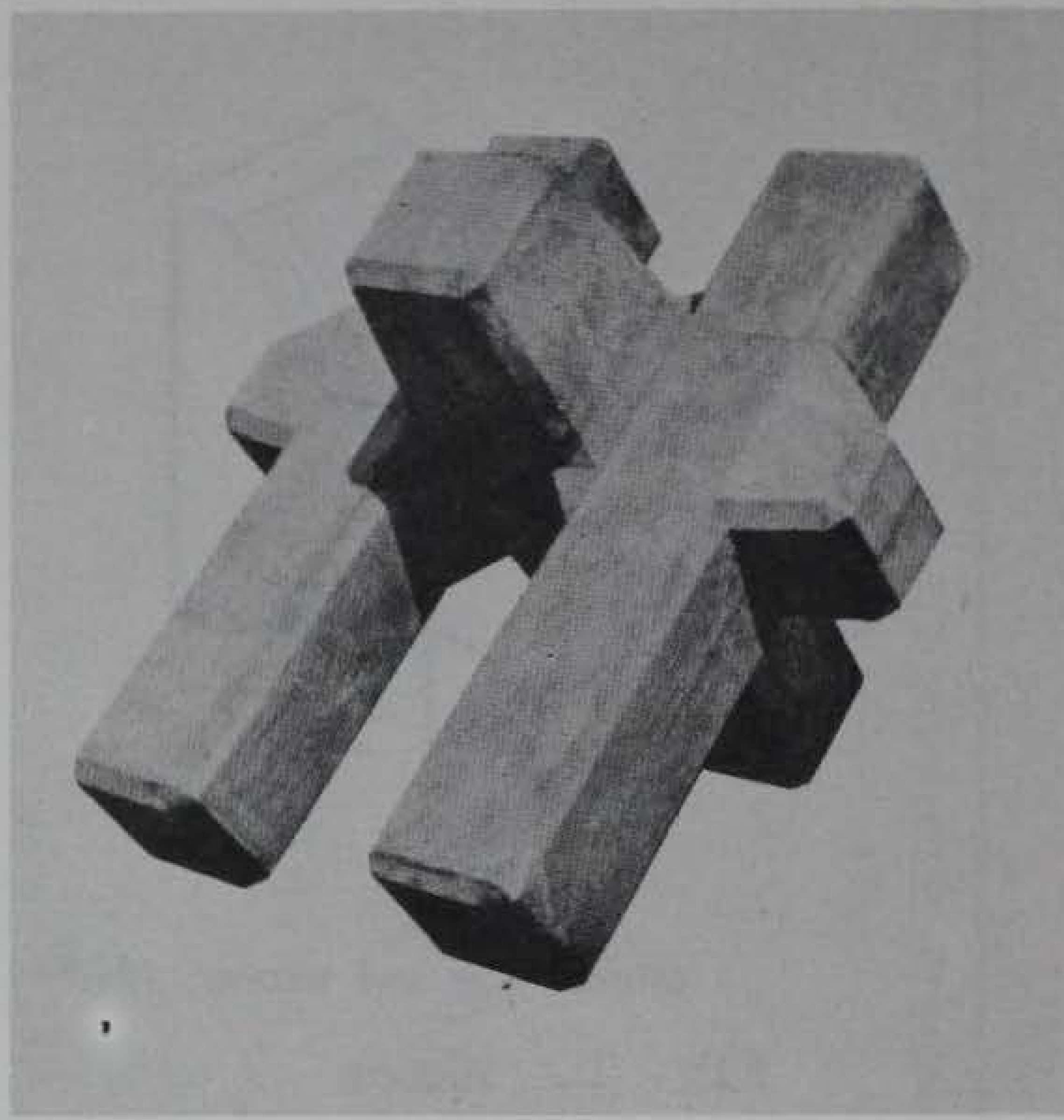
(After Jackson, 1968[12])

Fig. 4. Cube (modified)



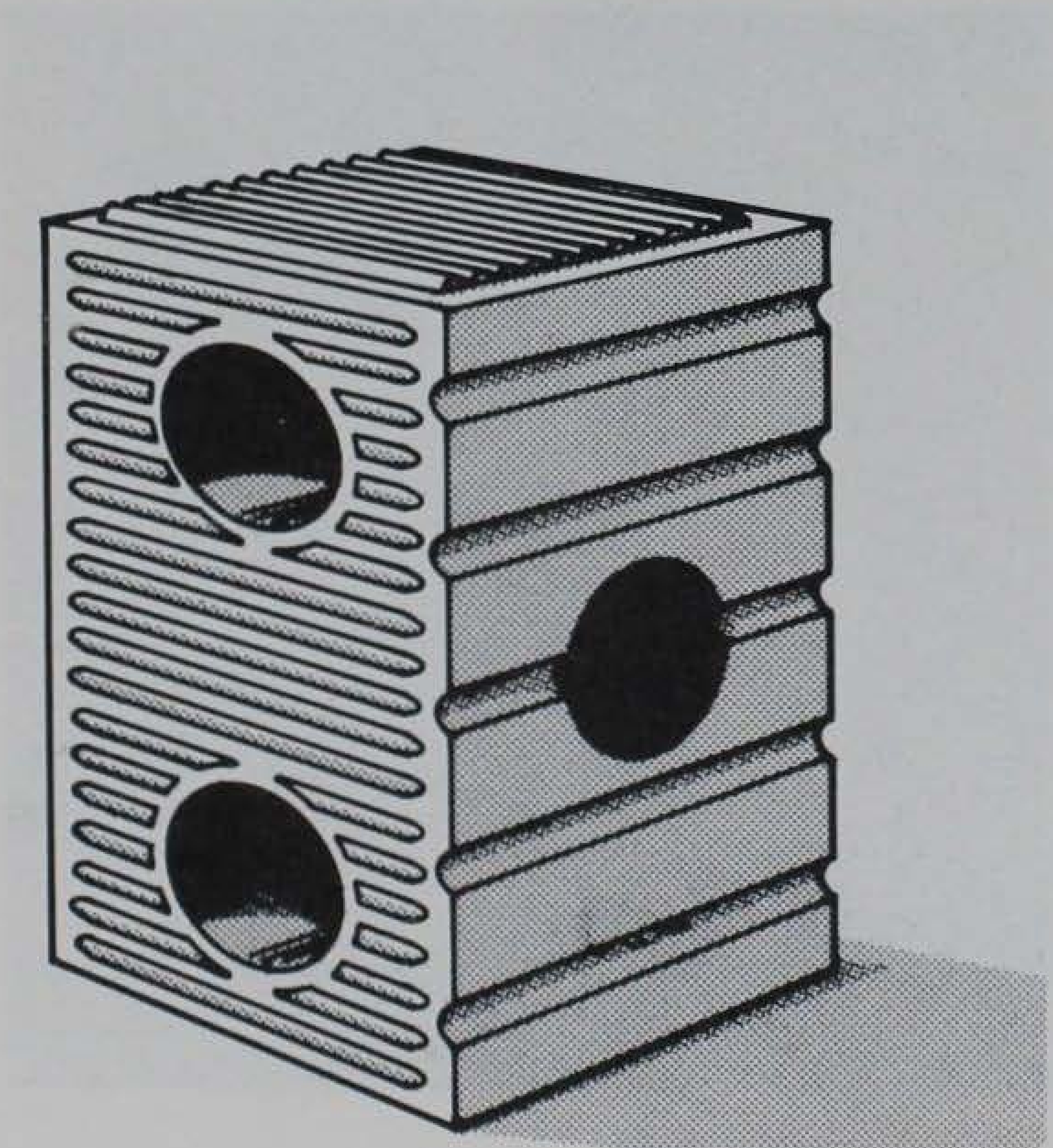
(Courtesy of E. M. Merrifield, Harbor Engineer, Port of East London, Republic of South Africa)

Fig. 5. Dolos



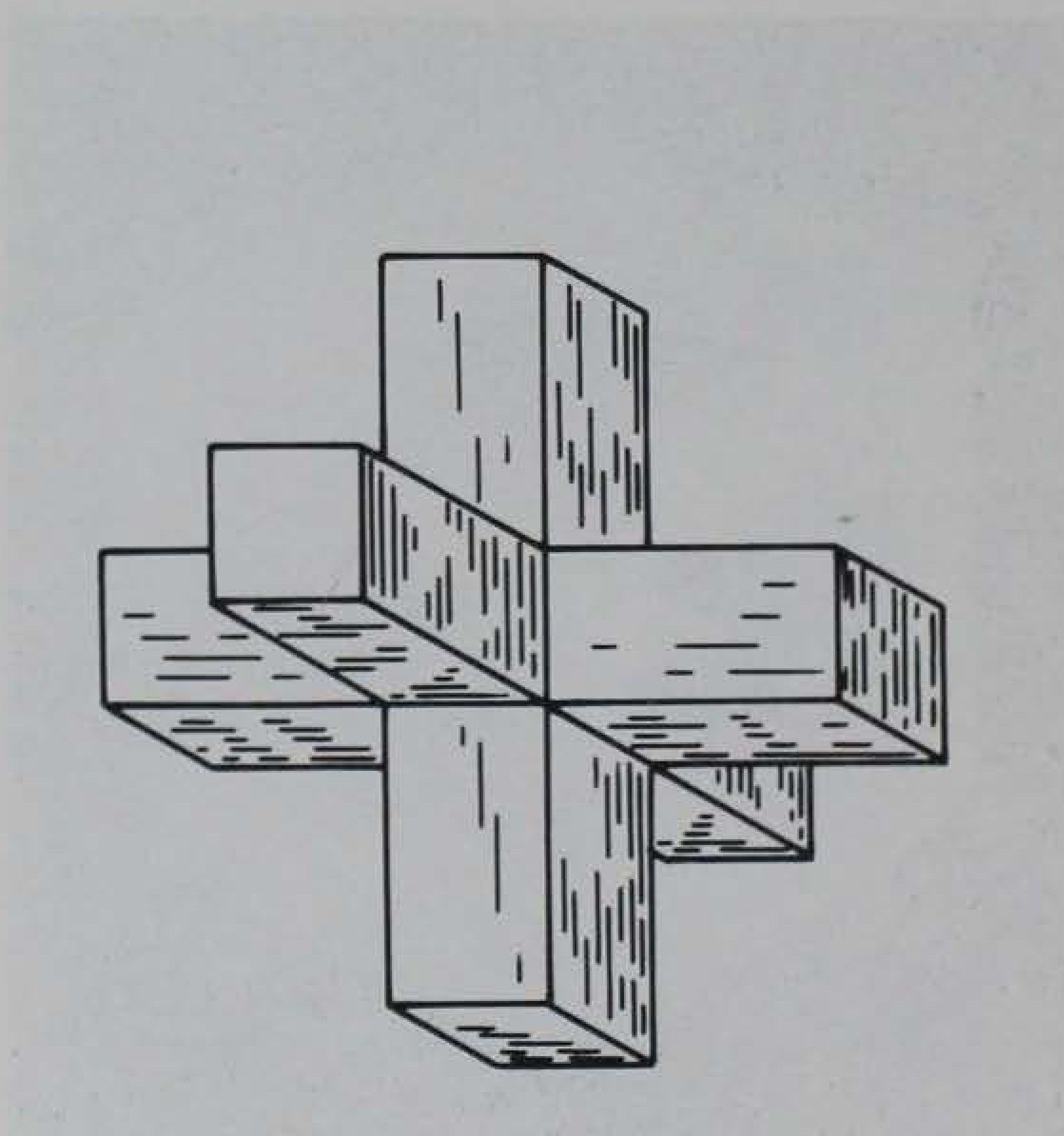
(Courtesy of S. Nagai, Osaka City University, Sugimoto-Cho, Sumiyoshi-Ku, Osaka, Japan)

Fig. 6. Gassho block



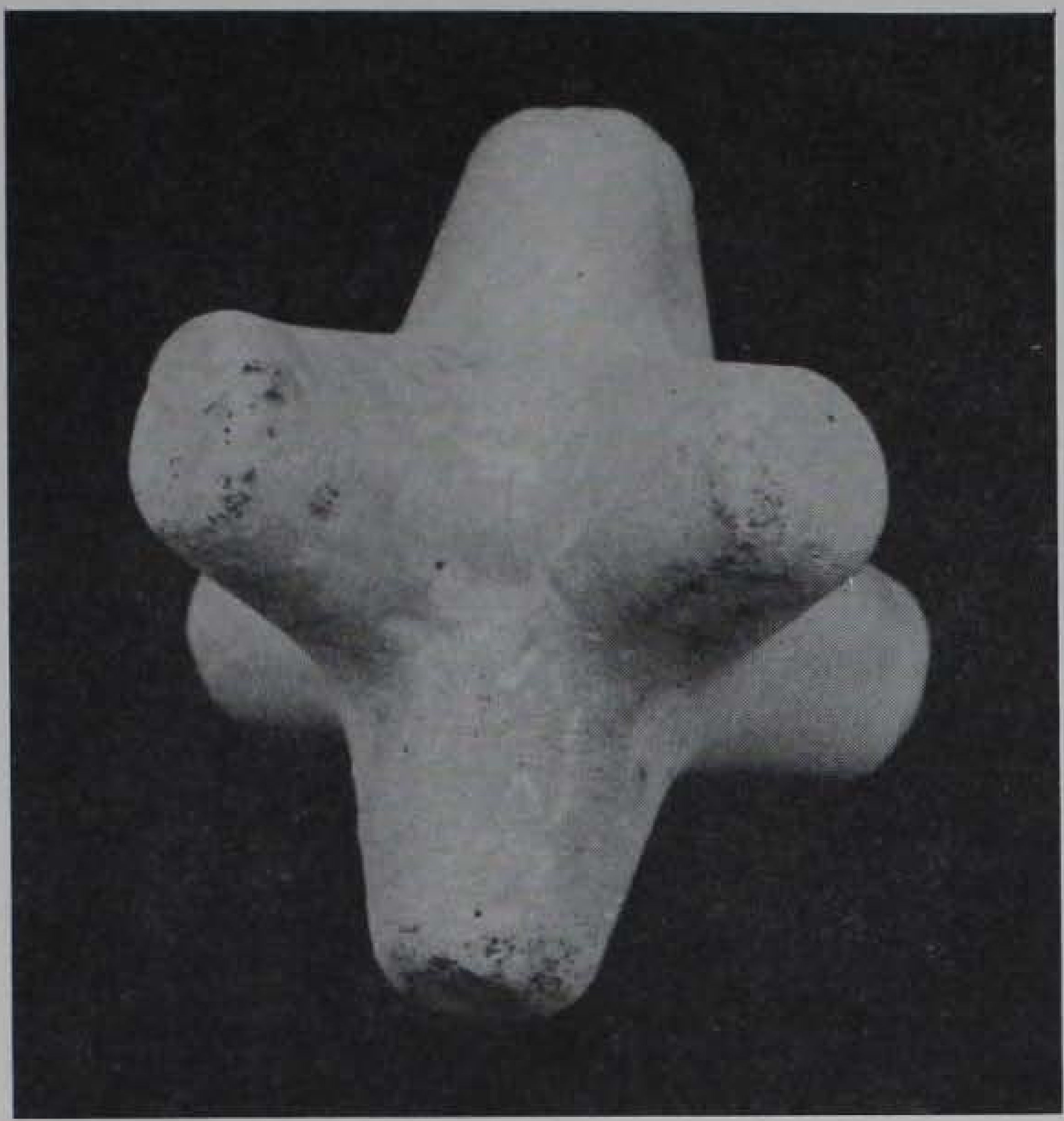
(Courtesy of P. Grobbelaar, 1971[26])

Fig. 7. Grobbelaar block



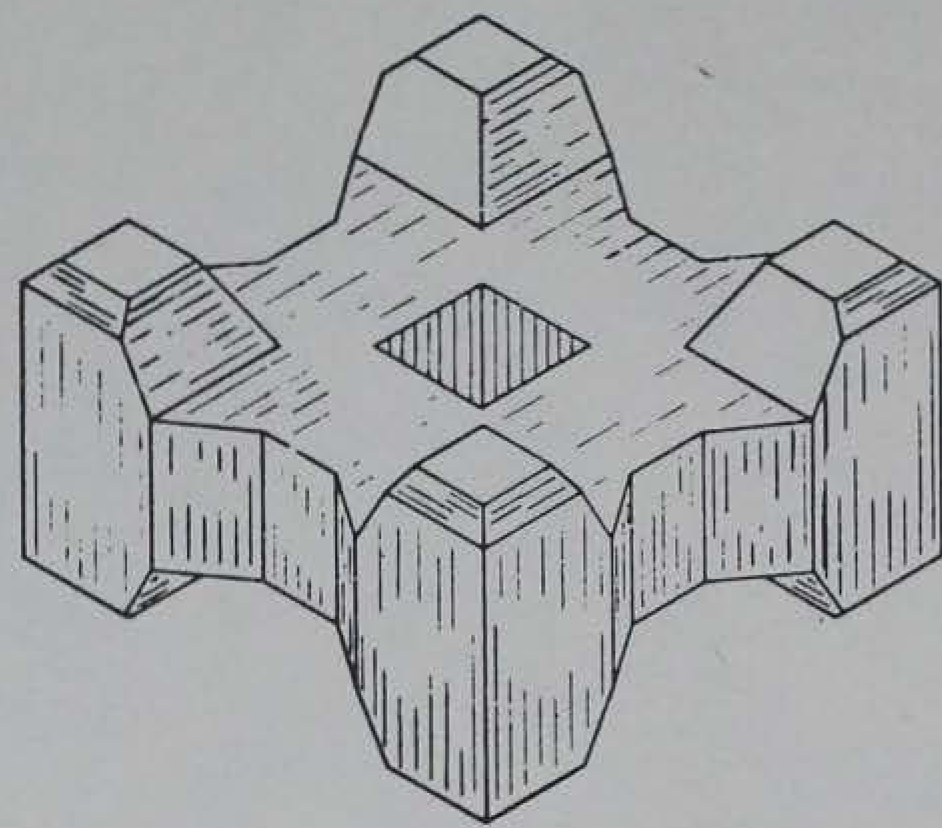
(From Hexaleg Block Works[27])

Fig. 8. Hexaleg block



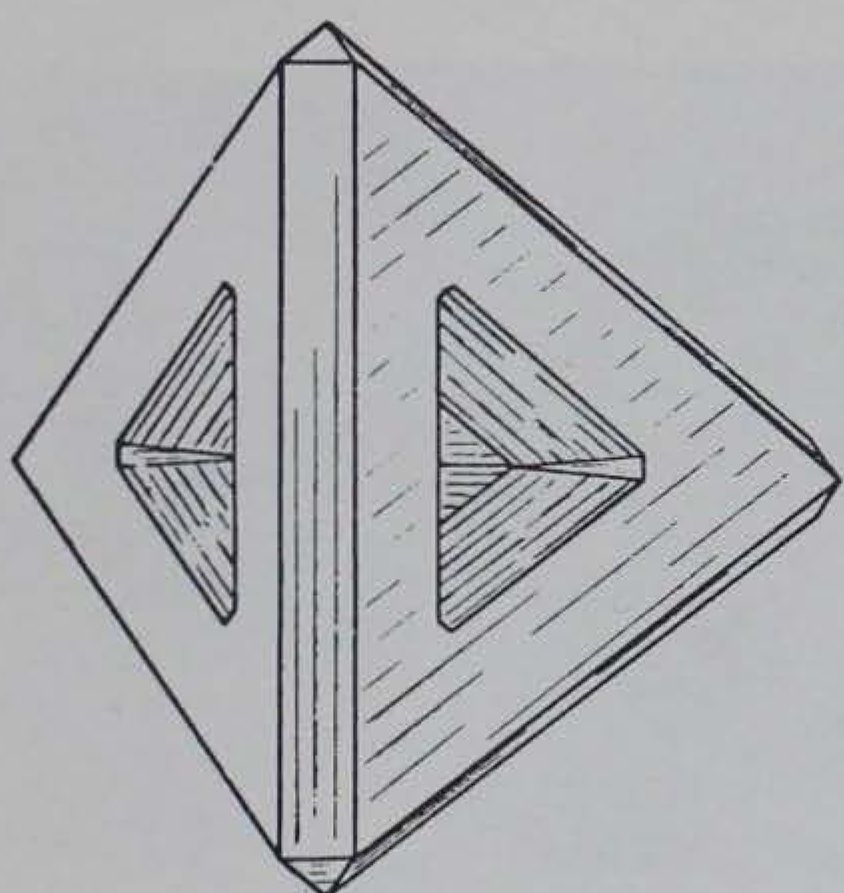
(After Jackson, 1968[12])

Fig. 9. Hexapod



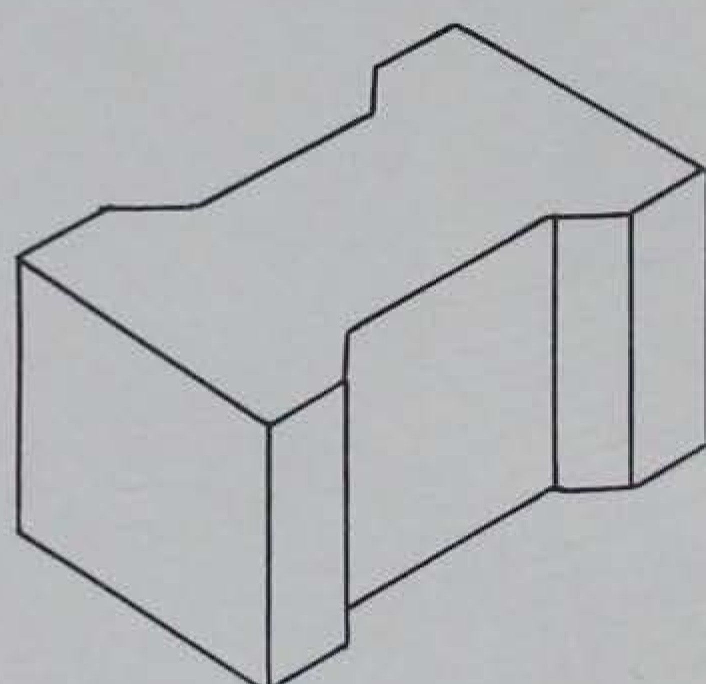
(After Nagai, 1962[28])

Fig. 10. Hollow square



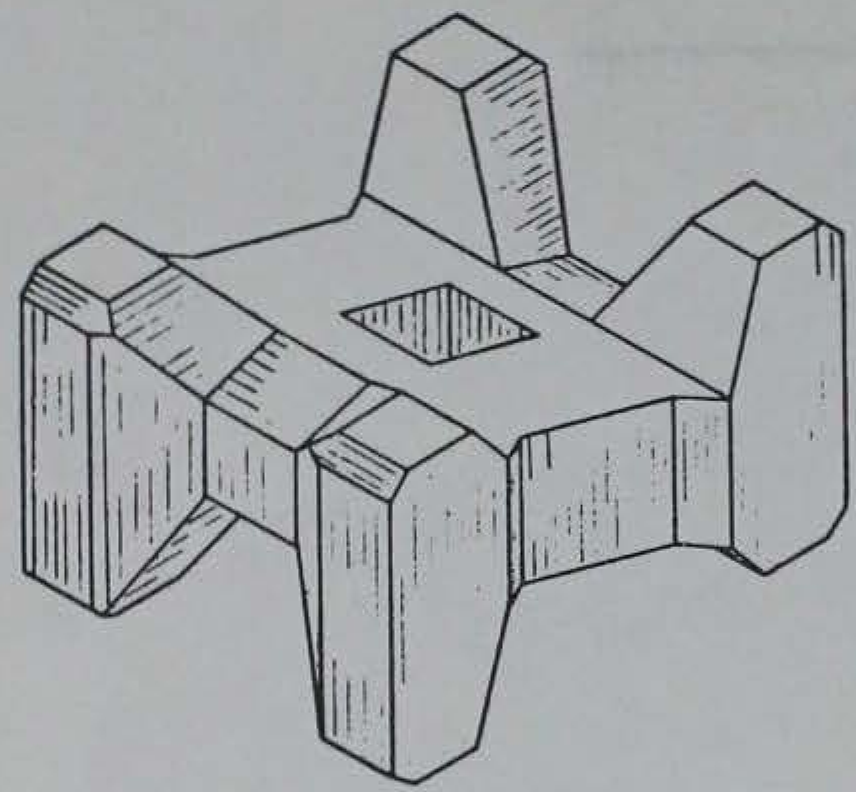
(After Nagai, 1961[14])

Fig. 11. Hollow tetrahedron



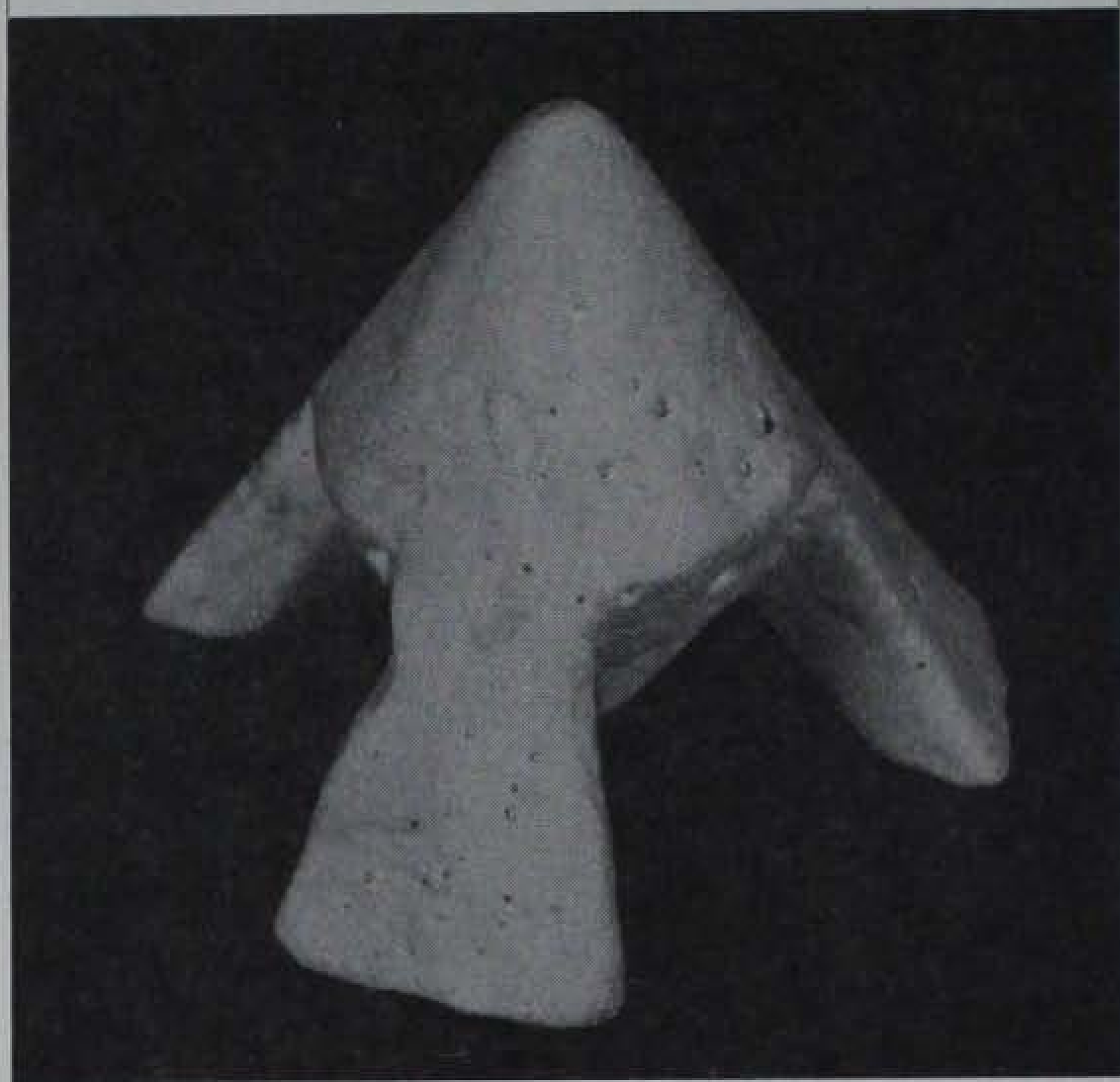
(Courtesy of U. S. Army Engineer District, Galveston, 1972[30])

Fig. 12. Interlocking H-block



(After Nagai, 1962[28])

Fig. 13. N-shaped block



(After Jackson, 1961[15])

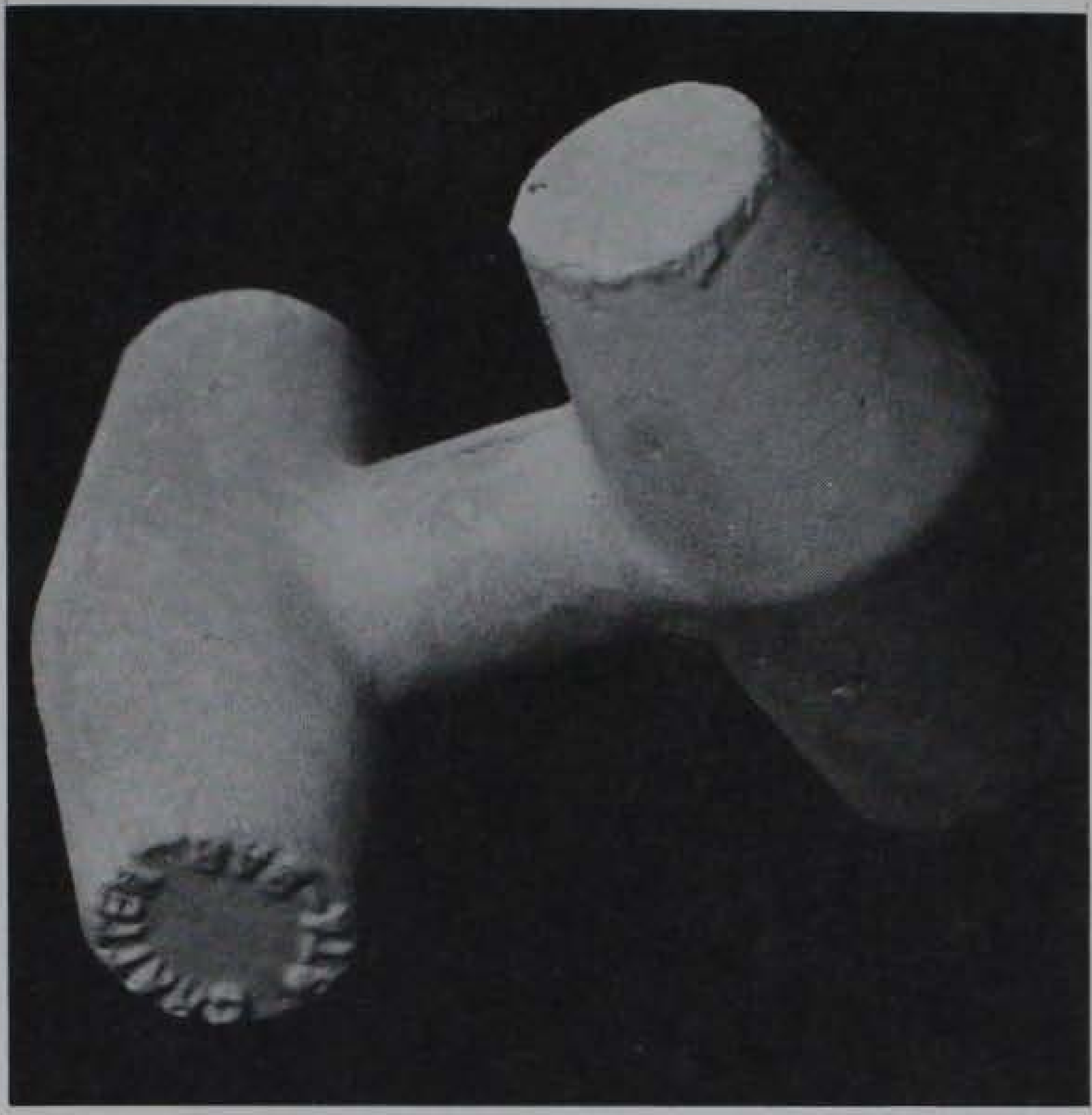
Fig. 14. Pelican stool

(After Jackson, 1968[12])

Fig. 15. Quadripod

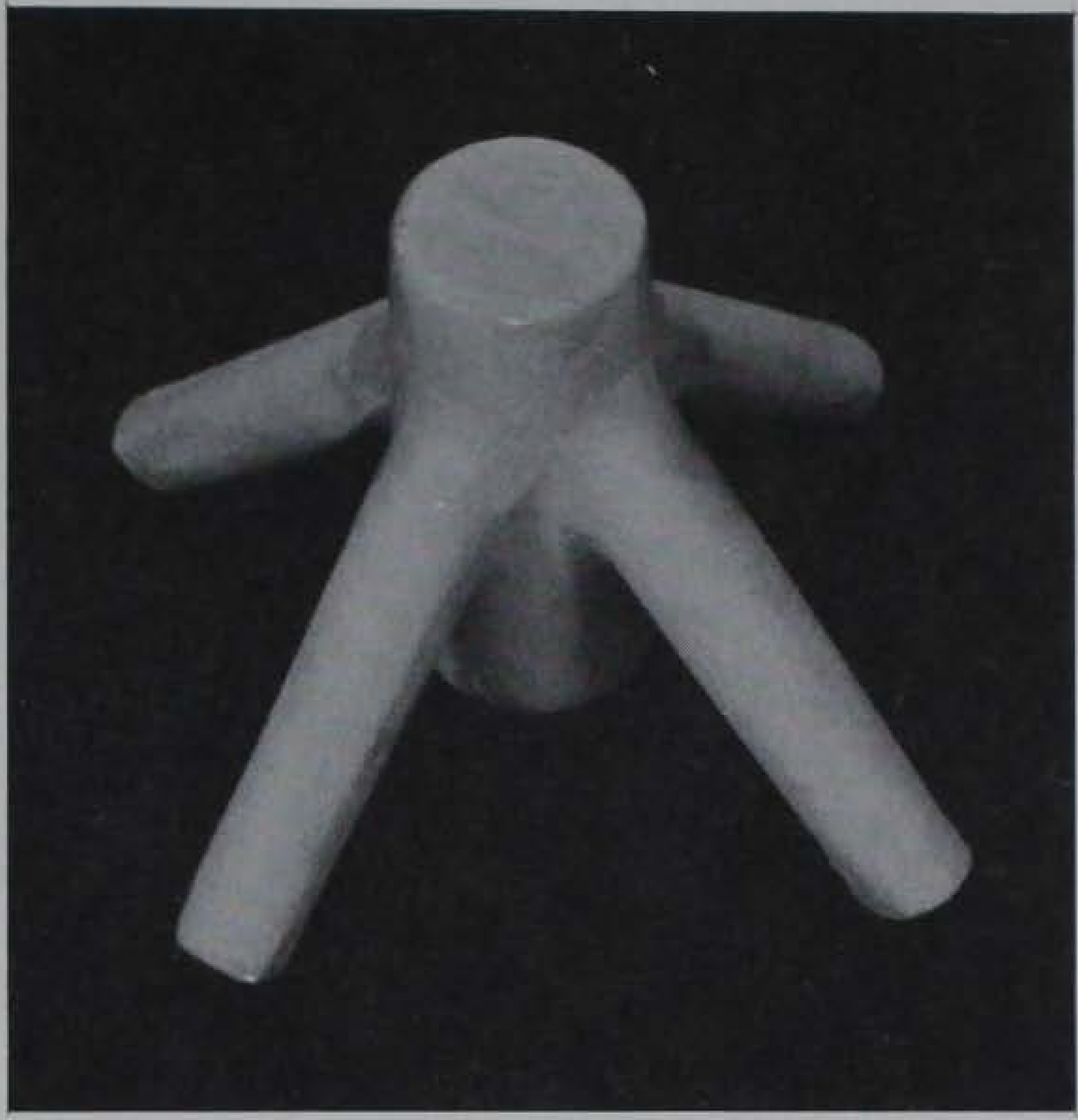
(Courtesy of Stabits Ltd., Sardinia House, 52 Lincoln's Inn Fields, London, W.C. 2)

Fig. 16. Stabit



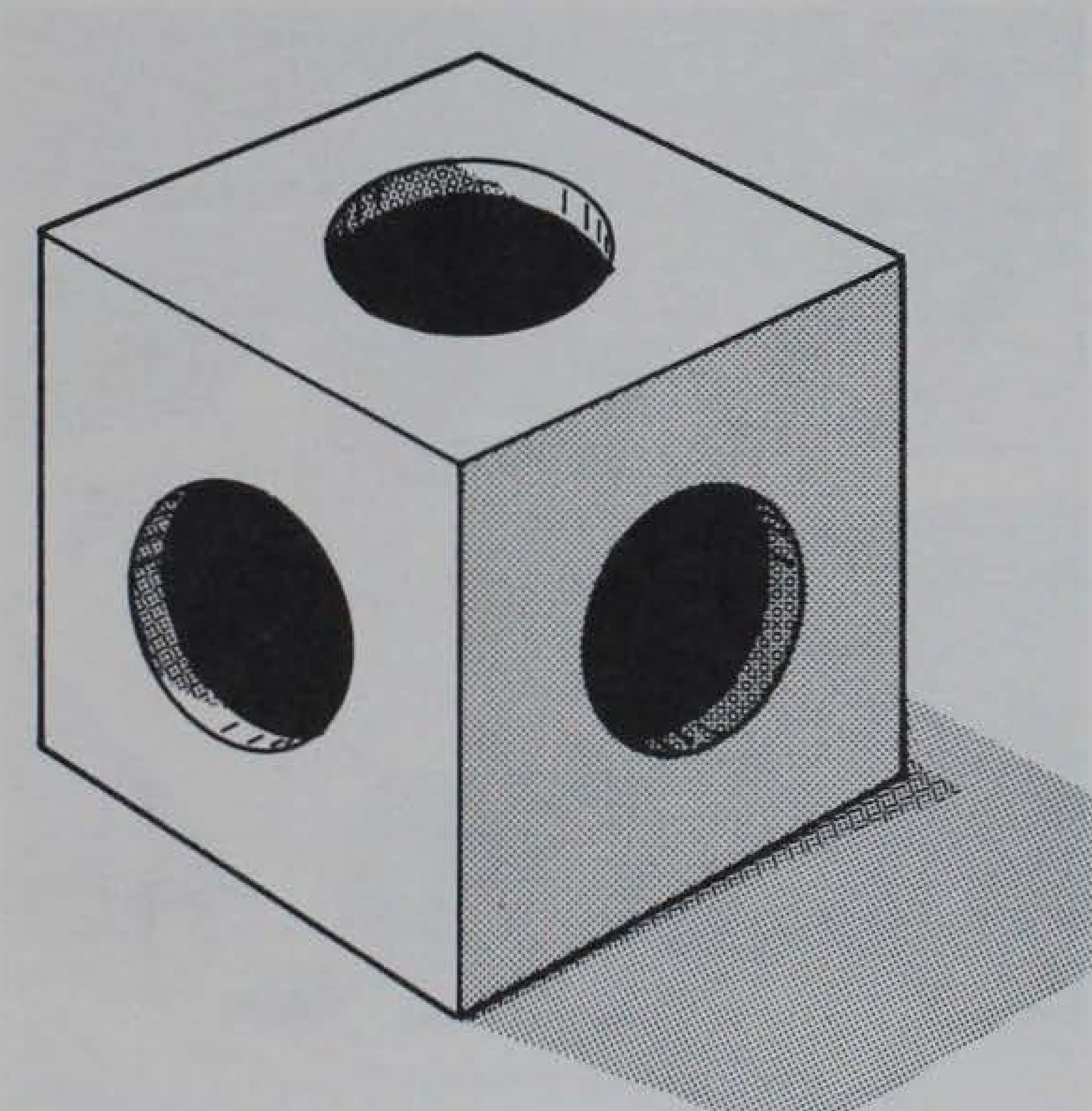
(Courtesy of R. J. O'Neill, Marine Modules, Inc., 475 Tuckahoe Road, Yonkers, N. Y. 10710)

Fig. 17. Sta-Bar



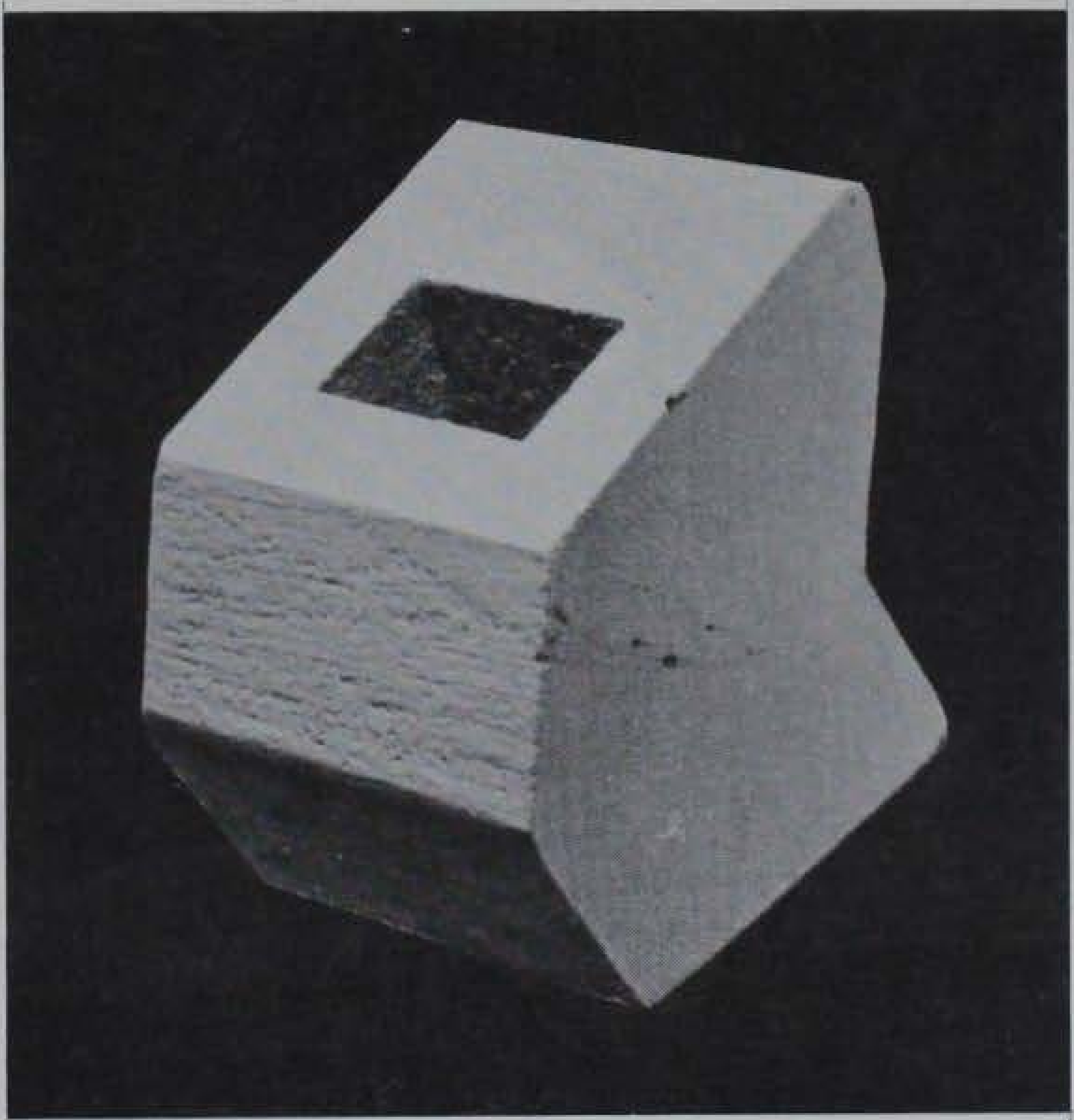
(Courtesy of R. J. O'Neill, Marine Modules, Inc., 475 Tuckahoe Road, Yonkers, N. Y. 10710)

Fig. 18. Sta-Pod



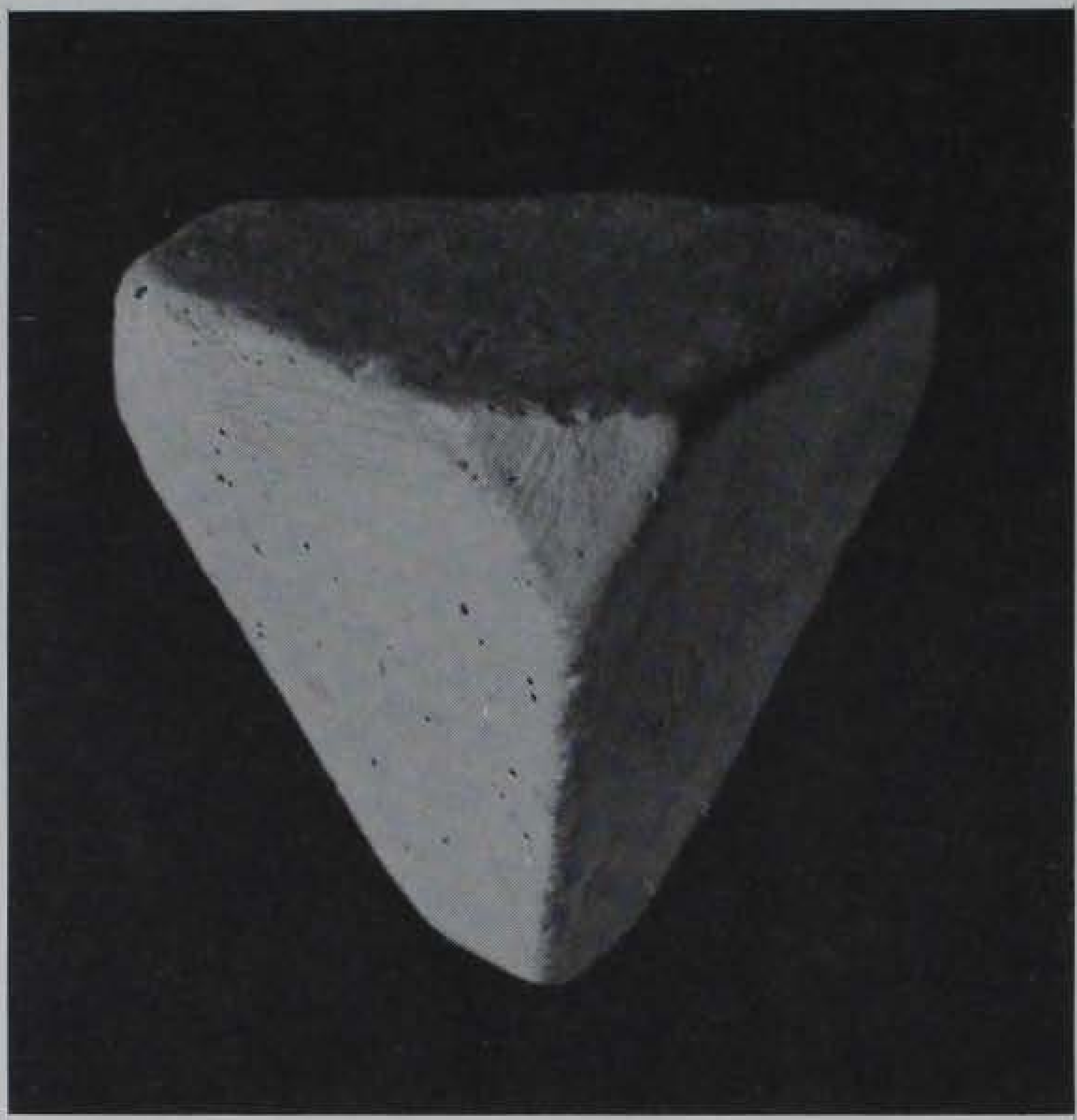
(Courtesy of B. Hakkeling, Ing, Merellaan 269, Maassluis, Netherlands)

Fig. 19. Stolk cube



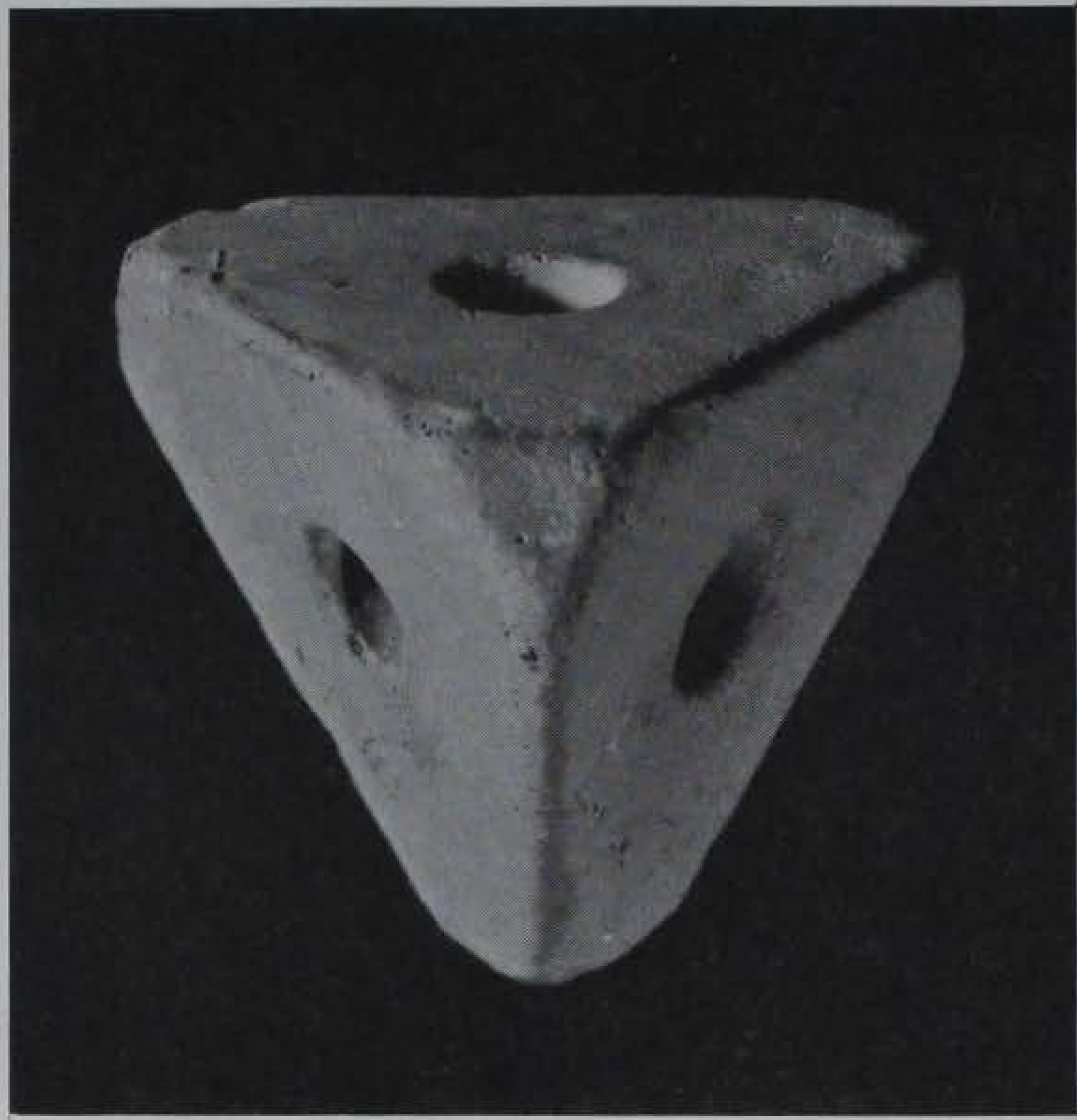
(Courtesy of Noreno, Cort Adlers Gate 16, Oslo, Norway)

Fig. 20. Svee block



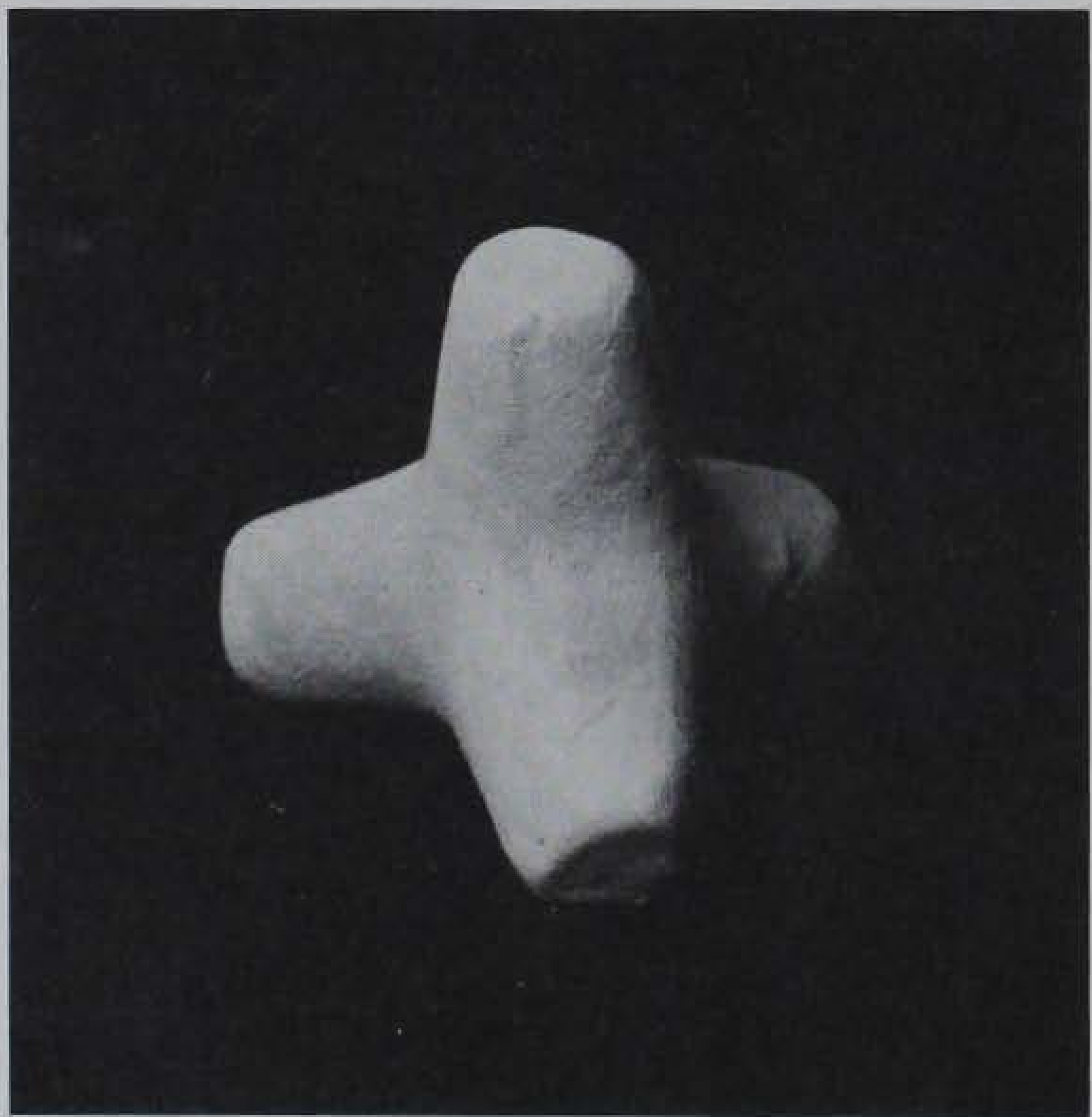
(After Jackson, 1968[12])

Fig. 21. Tetrahedron (solid)



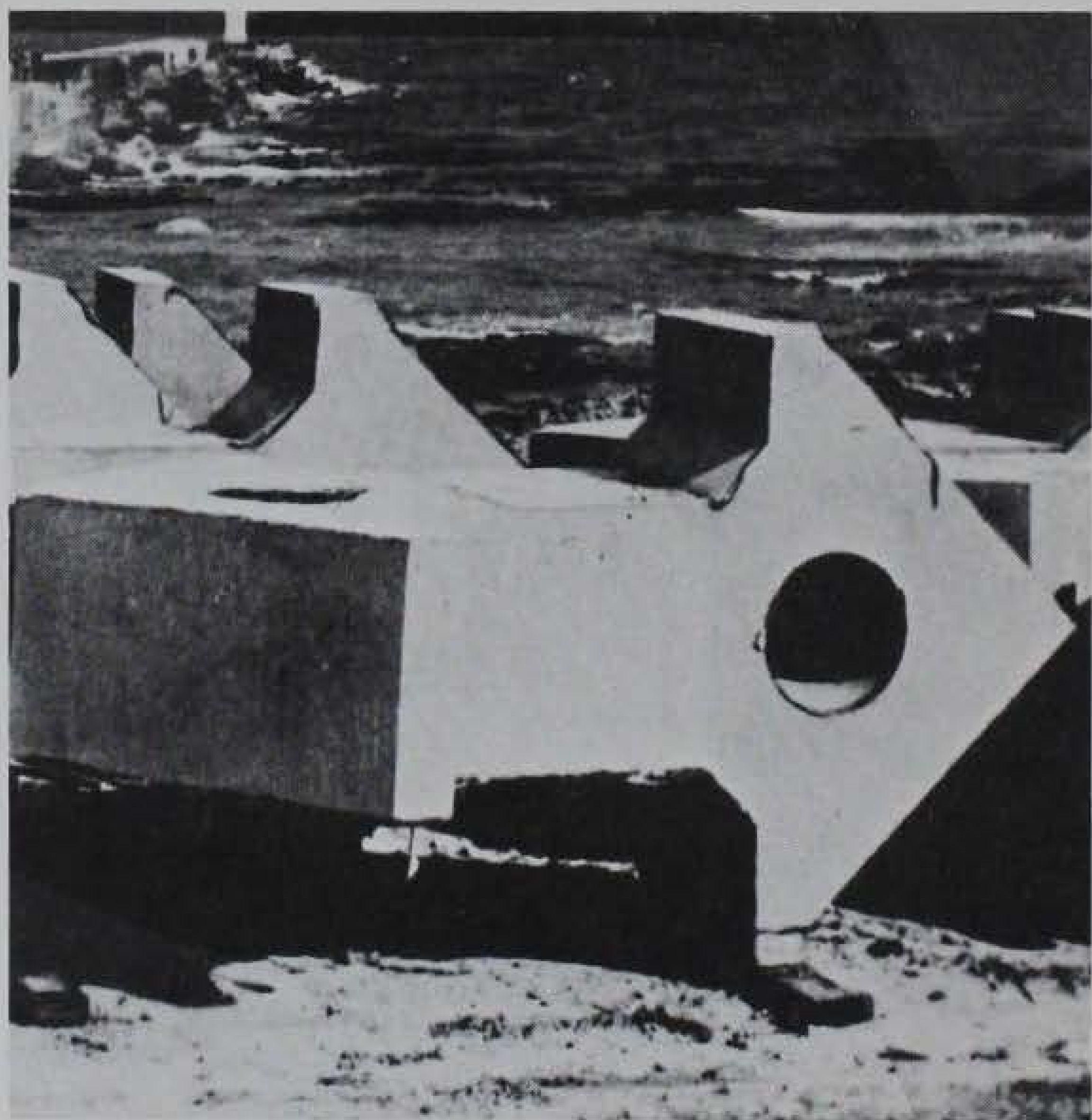
(After Jackson, 1968[12])

Fig. 22. Tetrahedron (perforated)



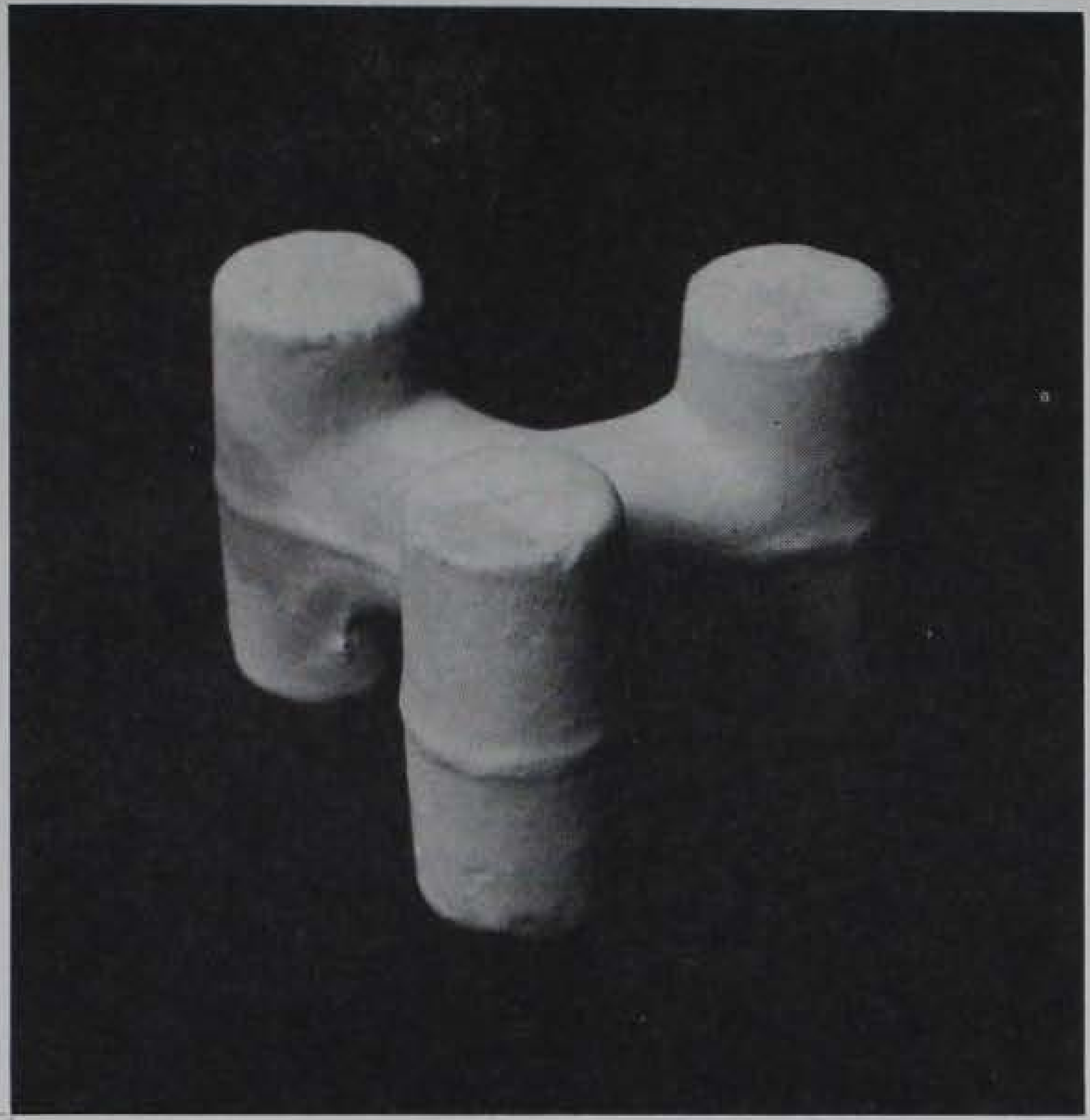
(After Tetrapods [Technical Note and Applications][37])

Fig. 23. Tetrapod



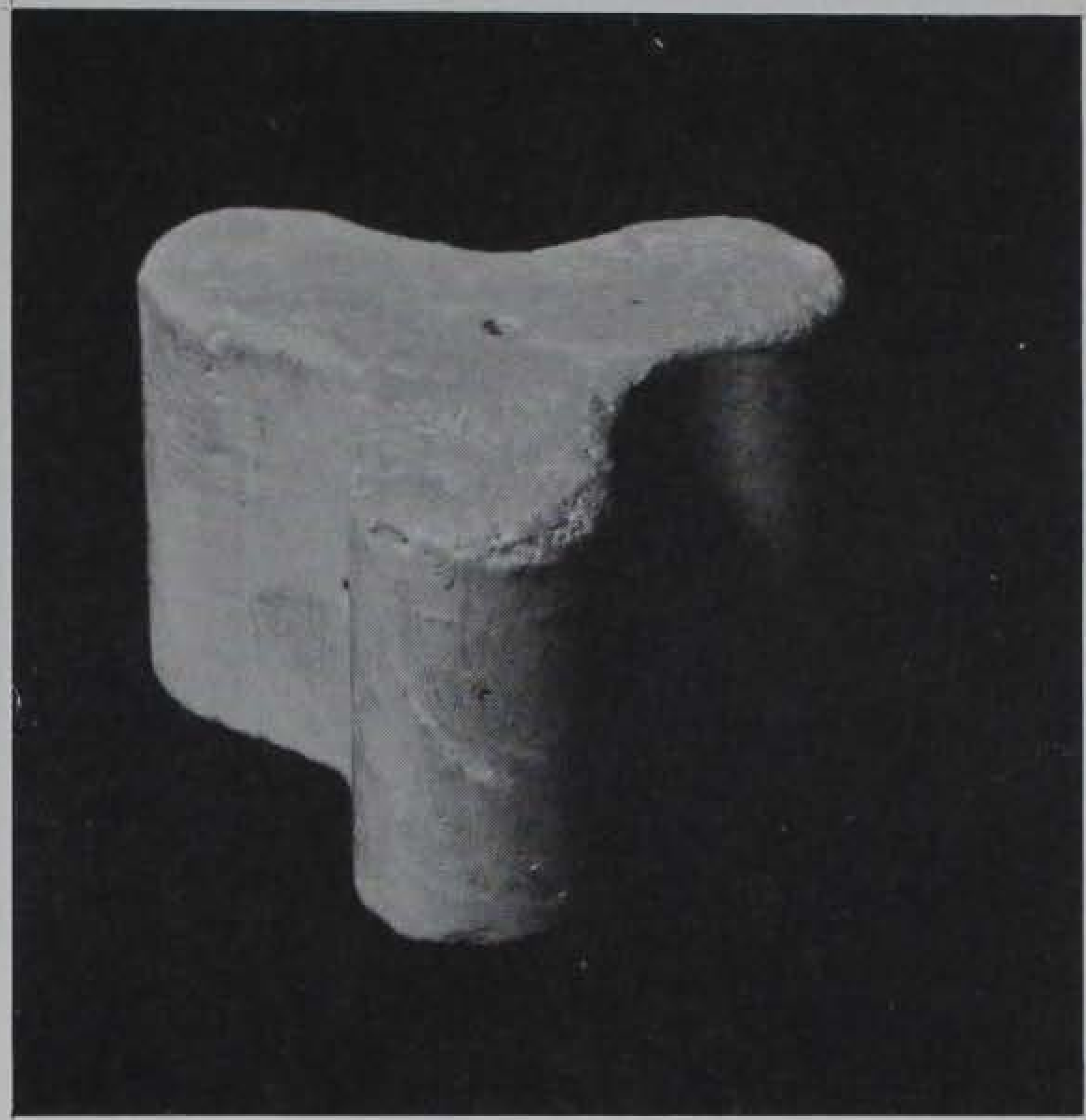
(Courtesy of P. Grobbelaar, 1971[26])

Fig. 24. Toskane



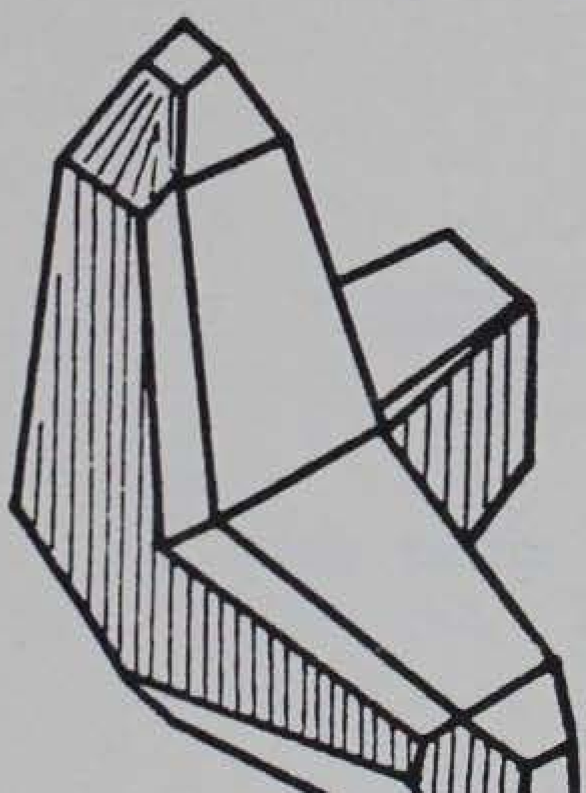
(After Jackson, 1968[12])

Fig. 25. Tribar



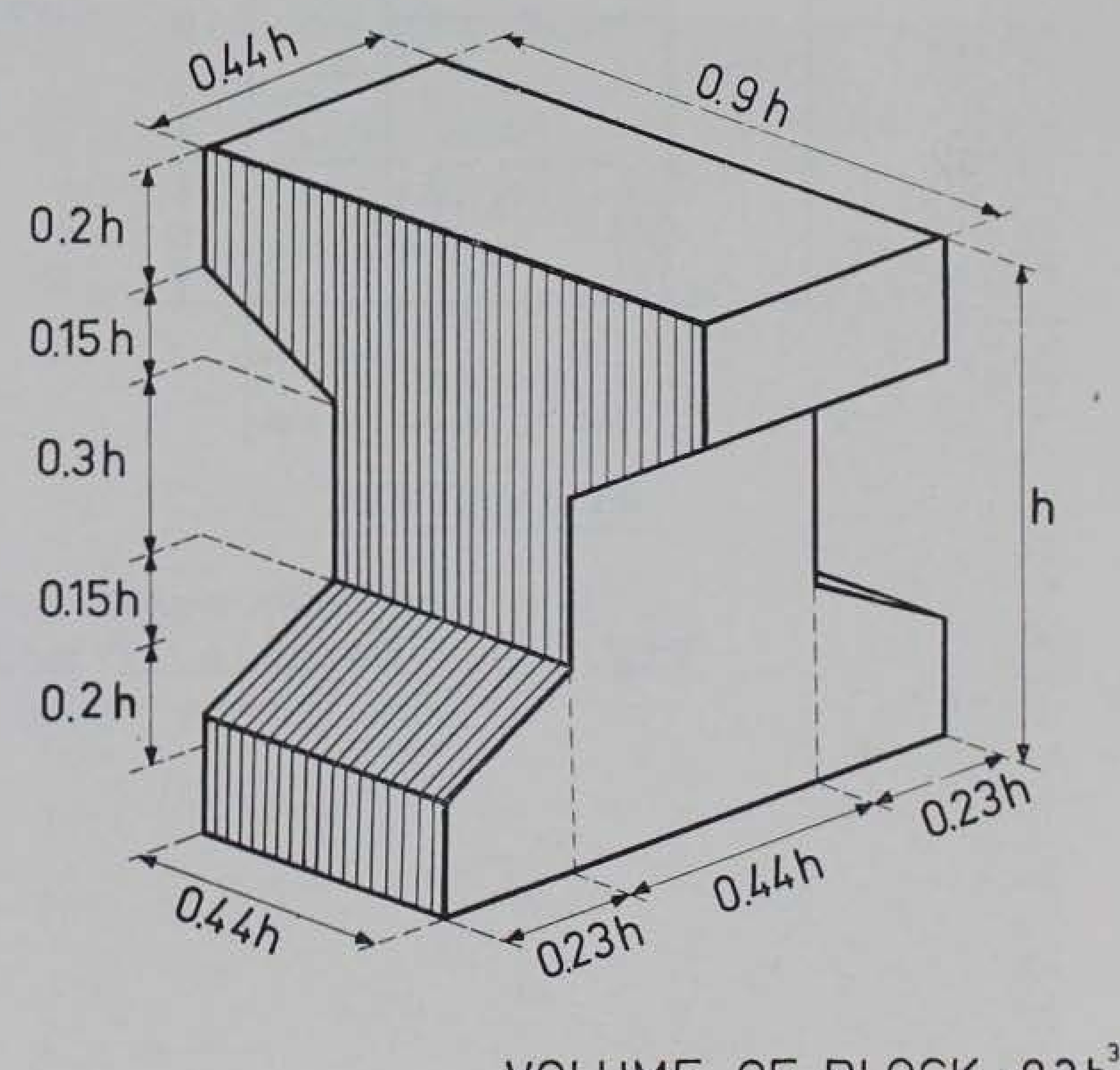
(After Davidson, 1971[36])

Fig. 26. Tri-long



(From Paape and Walther, 1963[23])

Fig. 27. Tripod



(From Paape and Walther, 1963[23])

Fig. 28. Details of Akmon armor unit

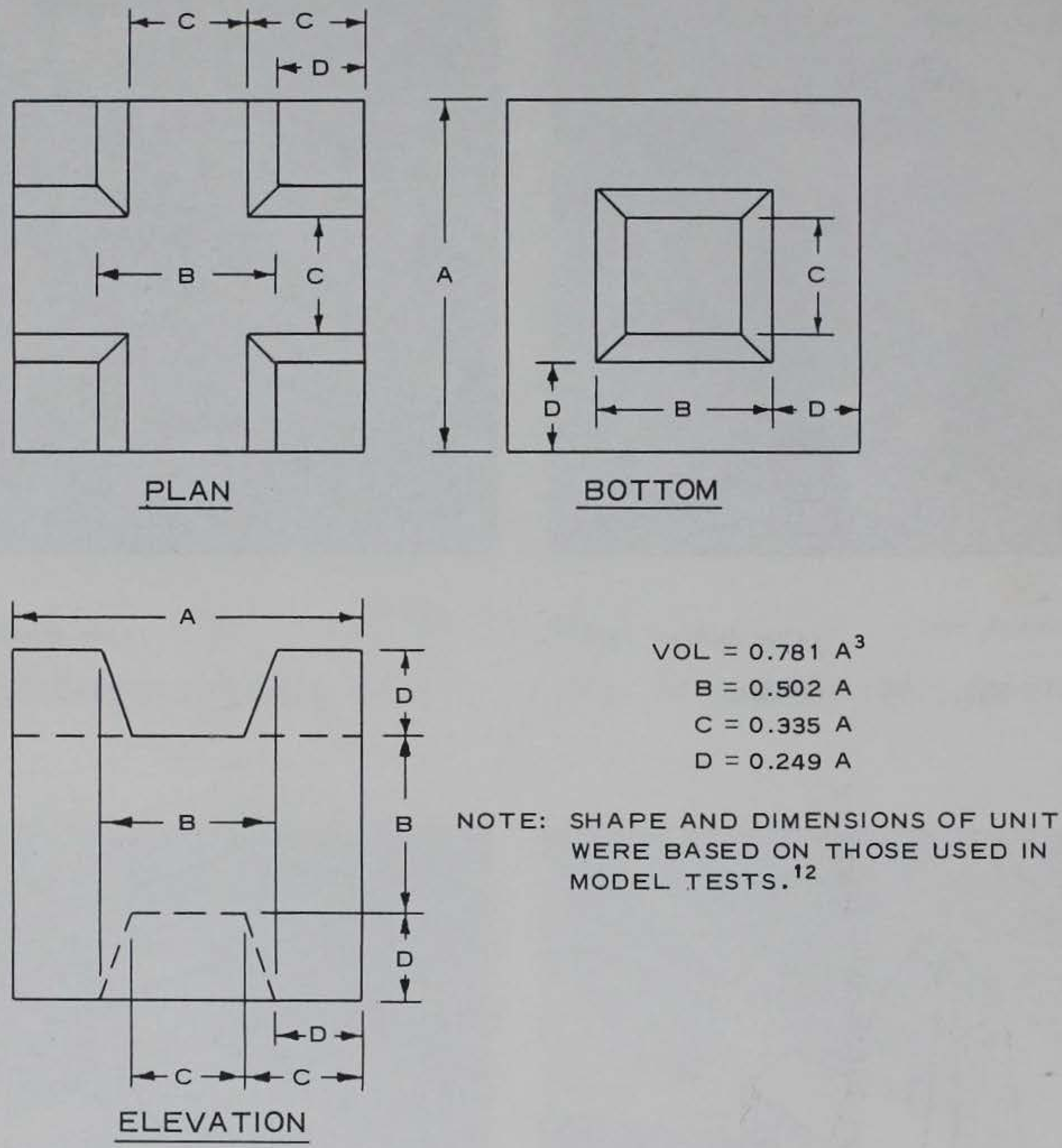
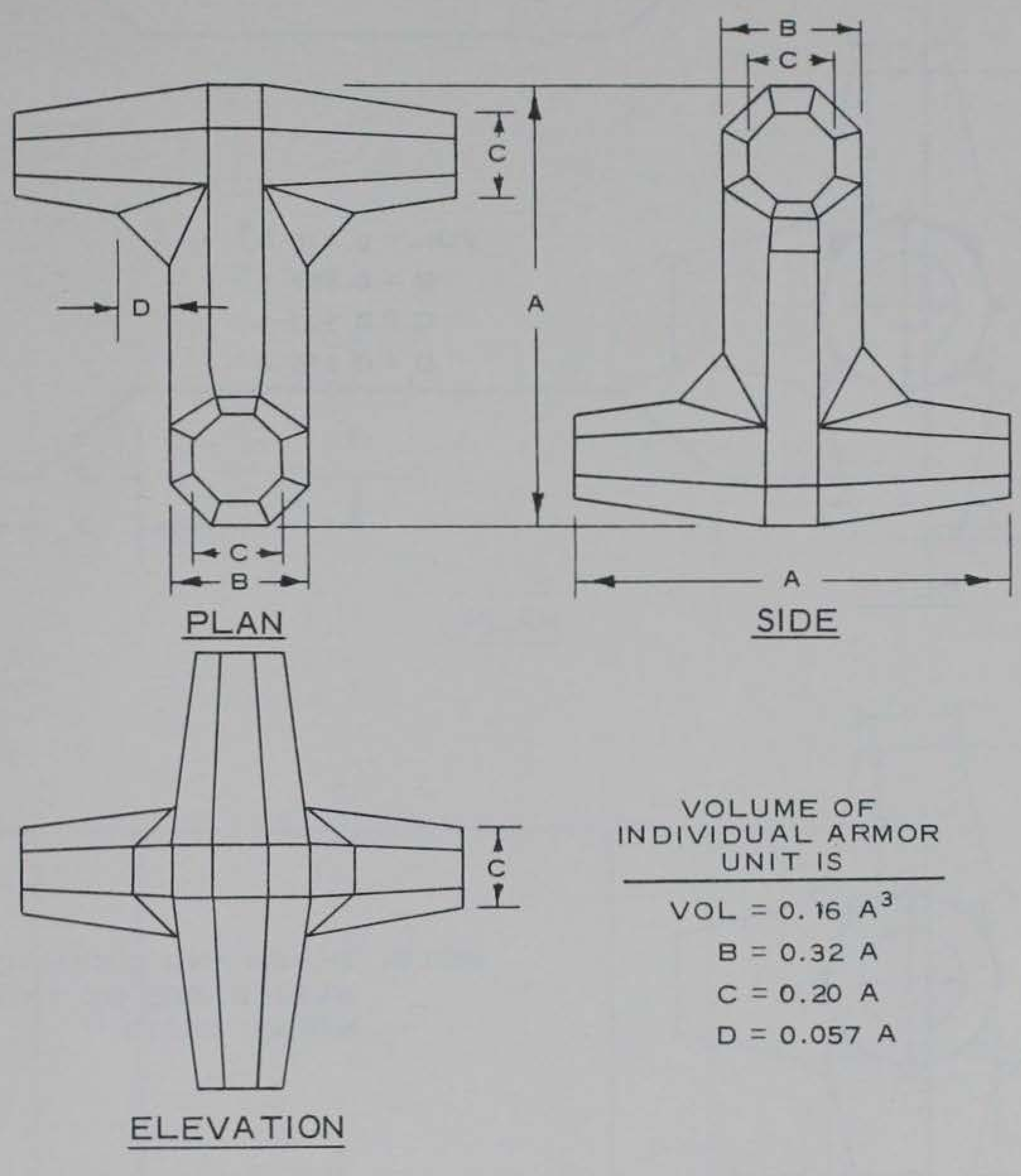
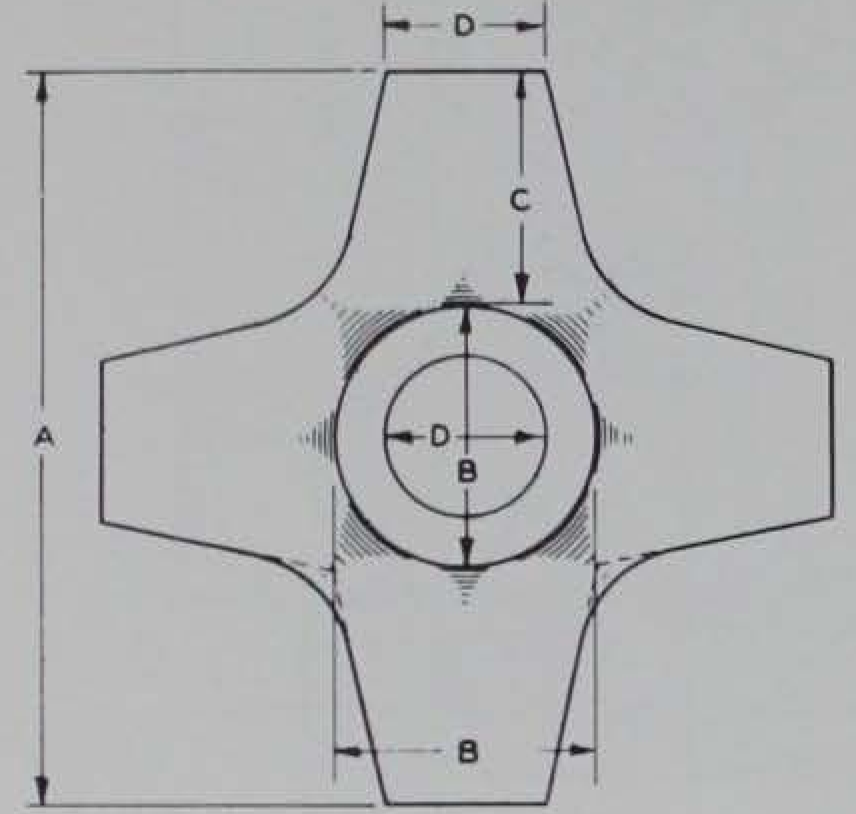


Fig. 29. Details of modified cube armor unit

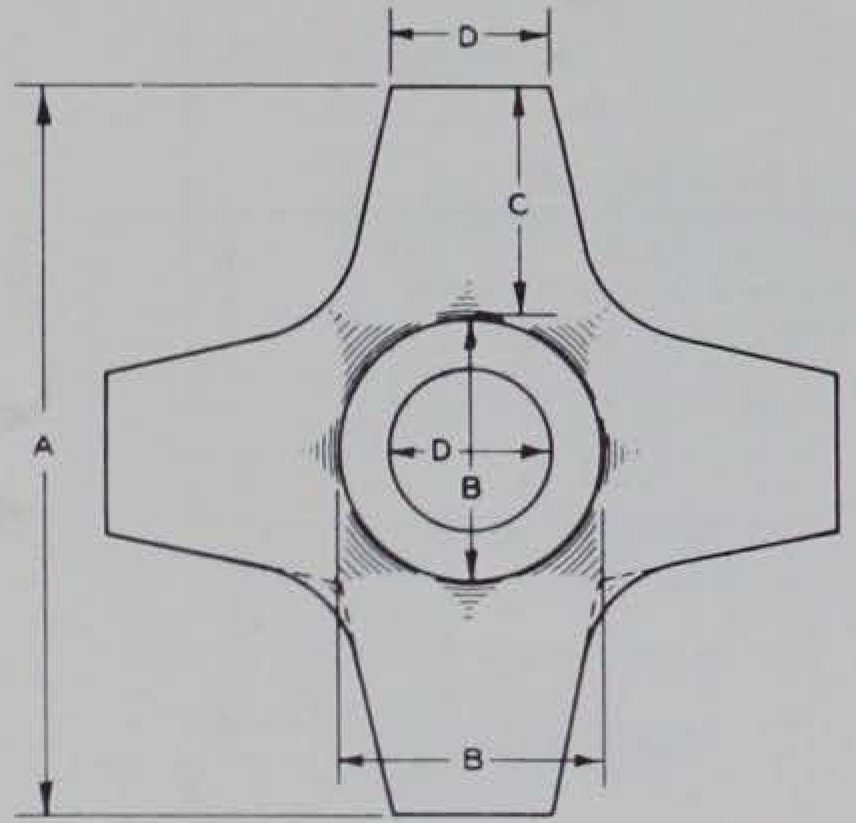


(From Magoon and Shimizu, 1971[38])

Fig. 30. Details of dolos armor unit



PLAN

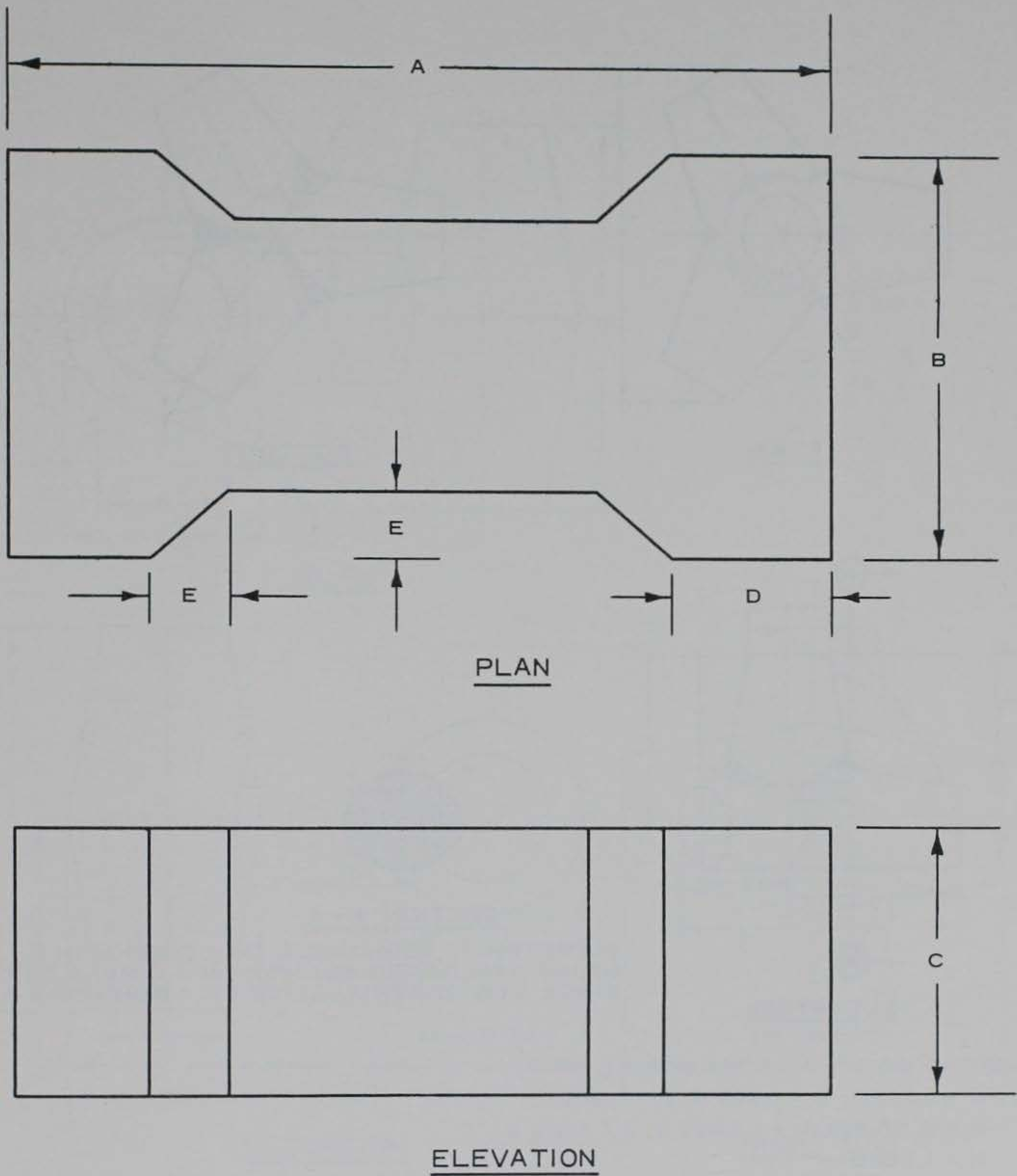


ELEVATION

VOL = 0.176 A^3
B = 0.357 A
C = 0.322 A
D = 0.215 A

NOTE: SHAPE AND DIMENSIONS OF UNIT WERE BASED ON THOSE USED IN MODEL TESTS.[12]

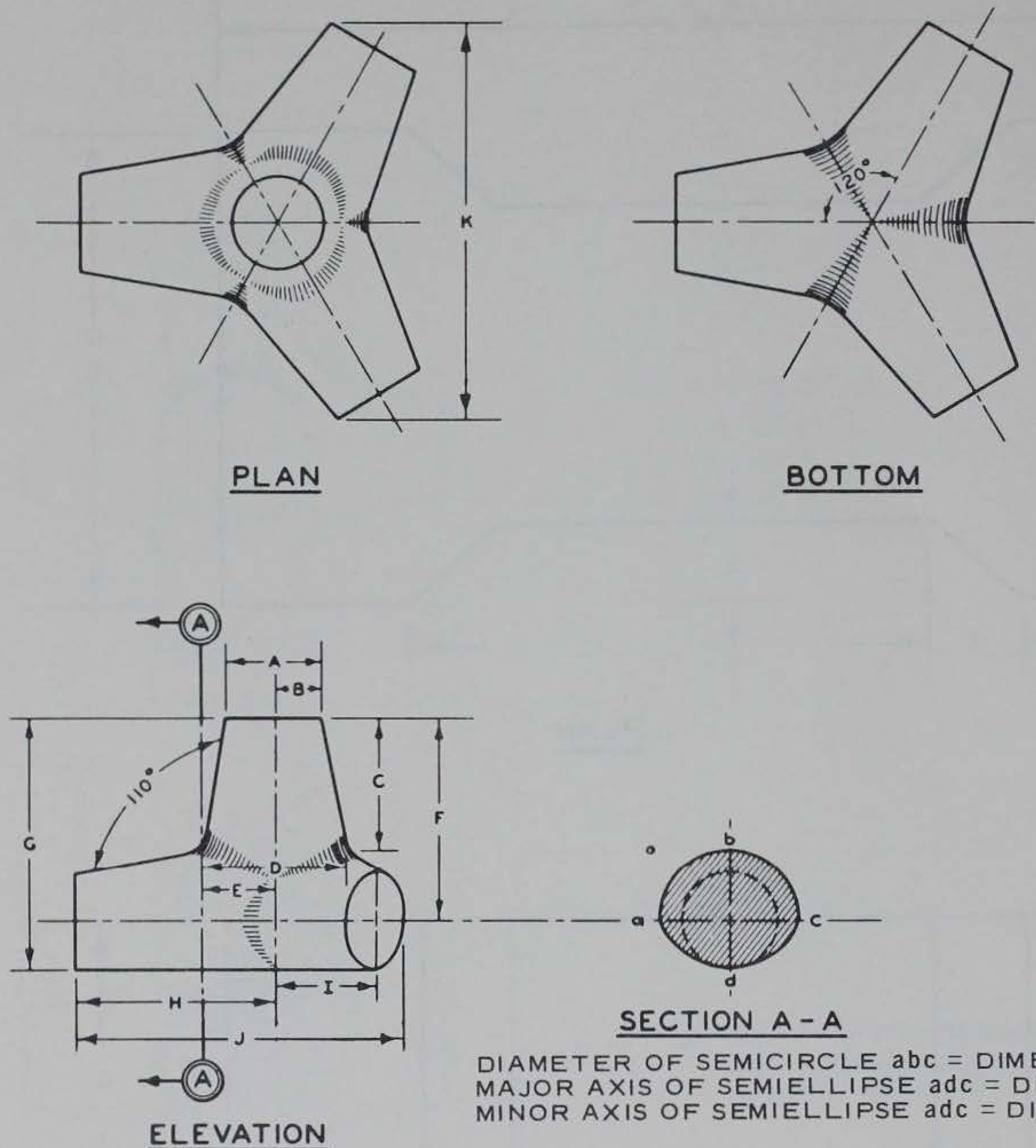
Fig. 31. Details of hexapod armor unit



V = ABC-2 CE (A-2D-E)
B = 0.745 A; C = 0.507 A
D = 0.197 A; E = 0.0845 A

(Courtesy of U. S. Army Engineer District, Galveston, 1972[30])

Fig. 32. Details of interlocking H-block

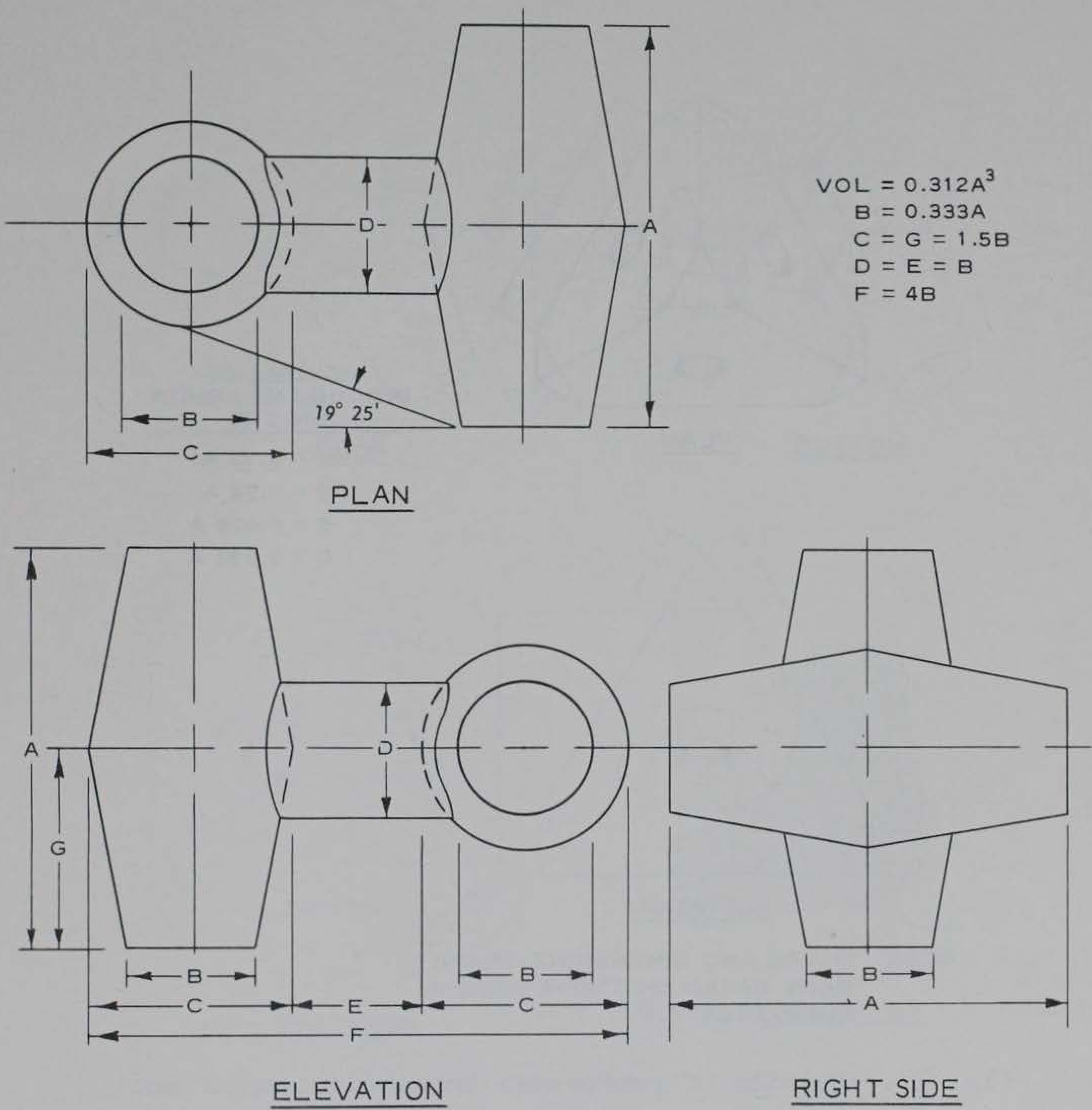


DIAMETER OF SEMICIRCLE abc = DIMENSION D
MAJOR AXIS OF SEMIELLIPSE adc = DIMENSION D
MINOR AXIS OF SEMIELLIPSE adc = DIMENSION A

VOL = 0.495 G^3; A = 0.382 G; B = 0.191 G;
C = 0.526 G; D = 0.566 G; E = 0.283 G;
F = H = 0.809 G; I = 0.405 G; J = 1.379 G;
K = 1.592 G

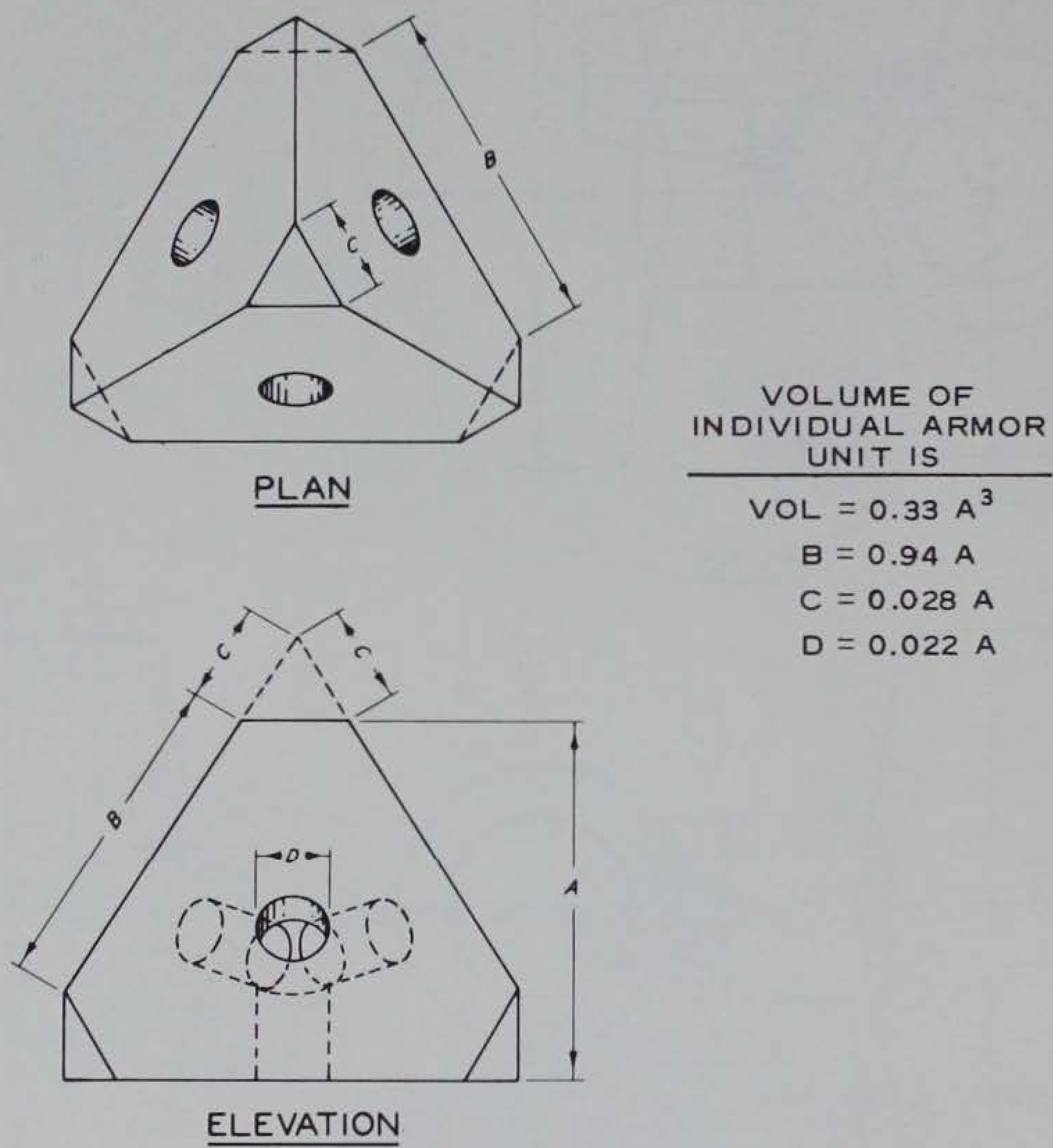
NOTE: SHAPE AND DIMENSIONS OF UNIT WERE BASED ON THOSE USED IN MODEL TESTS.[12]

Fig. 33. Details of quadripod armor unit



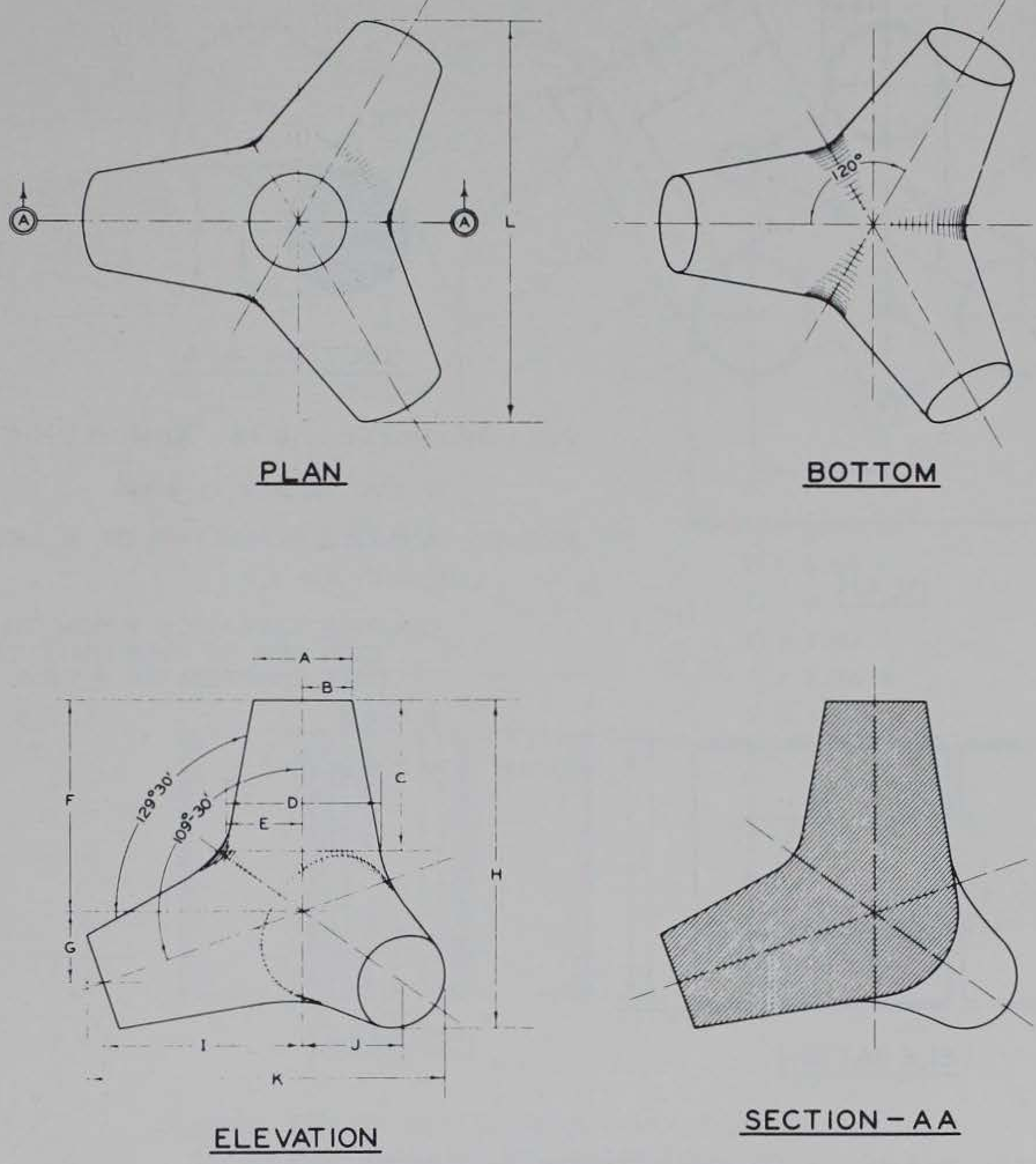
(Courtesy of R. J. O'Neill, Marine Modules, Inc., 475 Tuckahoe Road, Yonkers, N. Y. 10710)

Fig. 34. Details of Sta-Bar armor unit



NOTE: SHAPE AND DIMENSIONS OF UNIT WERE BASED ON THOSE USED IN MODEL TESTS.[12]

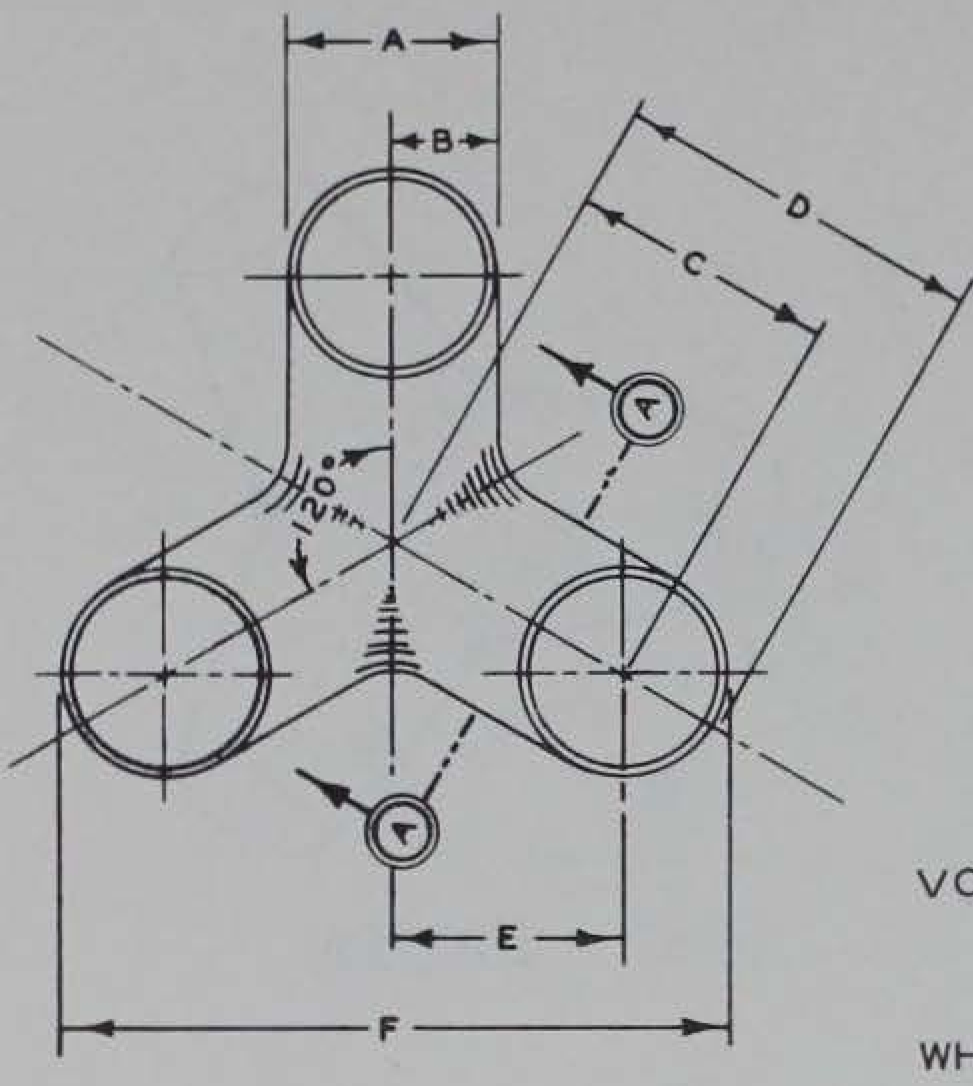
Fig. 35. Details of perforated tetrahedron armor unit



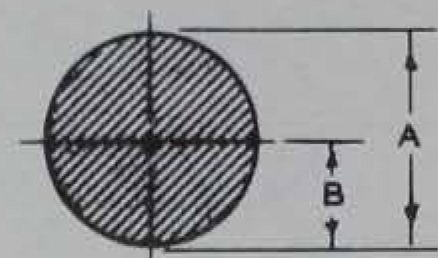
VOL = 0.280 H^3; A = 0.302 H; B = 0.151 H;
C = 0.477 H; D = 0.470 H; E = 0.235 H;
F = 0.644 H; G = 0.215 H; I = 0.606 H;
J = 0.303 H; K = 1.091 H; L = 1.201 H;

NOTE: SHAPE AND DIMENSIONS OF UNIT WERE BASED ON THOSE USED IN MODEL TESTS.[12]

Fig. 36. Details of tetrapod armor unit



PLAN



SECTION A-A

VOLUME OF INDIVIDUAL ARMOR UNIT IS

$$V = A^3 (2.36\, k_v + 3.42)$$

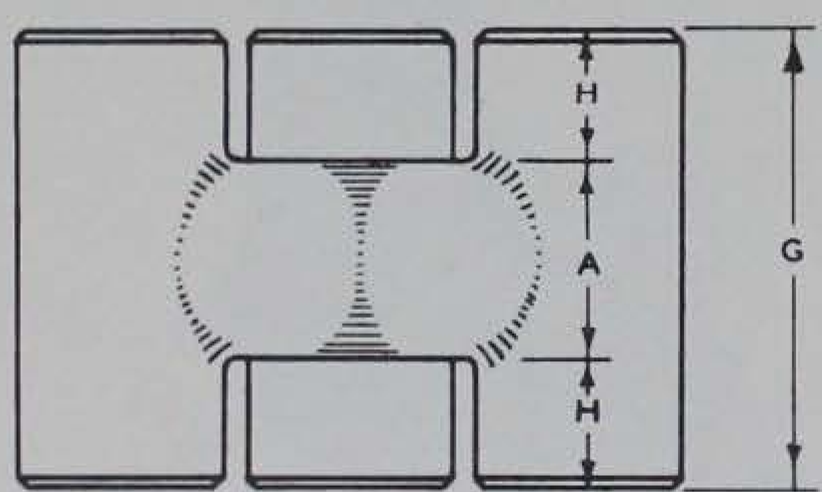
WHERE A = THE DIAMETER OF A LEG

$k_v = C/A = 1.2$

C = THE DISTANCE FROM THE CENTER OF THE UNIT TO THE CENTER OF A LEG

$G = 2\,A$

THUS $V = 6.252\, A^3$



ELEVATION

NOTE: SHAPE AND DIMENSIONS OF UNIT WERE BASED ON THOSE USED IN MODEL TESTS.[12] AT PRESENT TIME PATENTEE RECOMMENDS $C = 1.25\,A$, AND FILLETS AT INTERSECTION OF HORIZONTAL AND VERTICAL MEMBERS WITH A RADIUS EQUAL TO $A/4$. THE EQUATION FOR VOLUME IN CU YD IS THEN, APPROXIMATELY, $V = 0.24^3$. DETAILS OF FORMS SHOULD BE OBTAINED FROM PATENTEE.

Fig. 37. Details of tribar armor unit

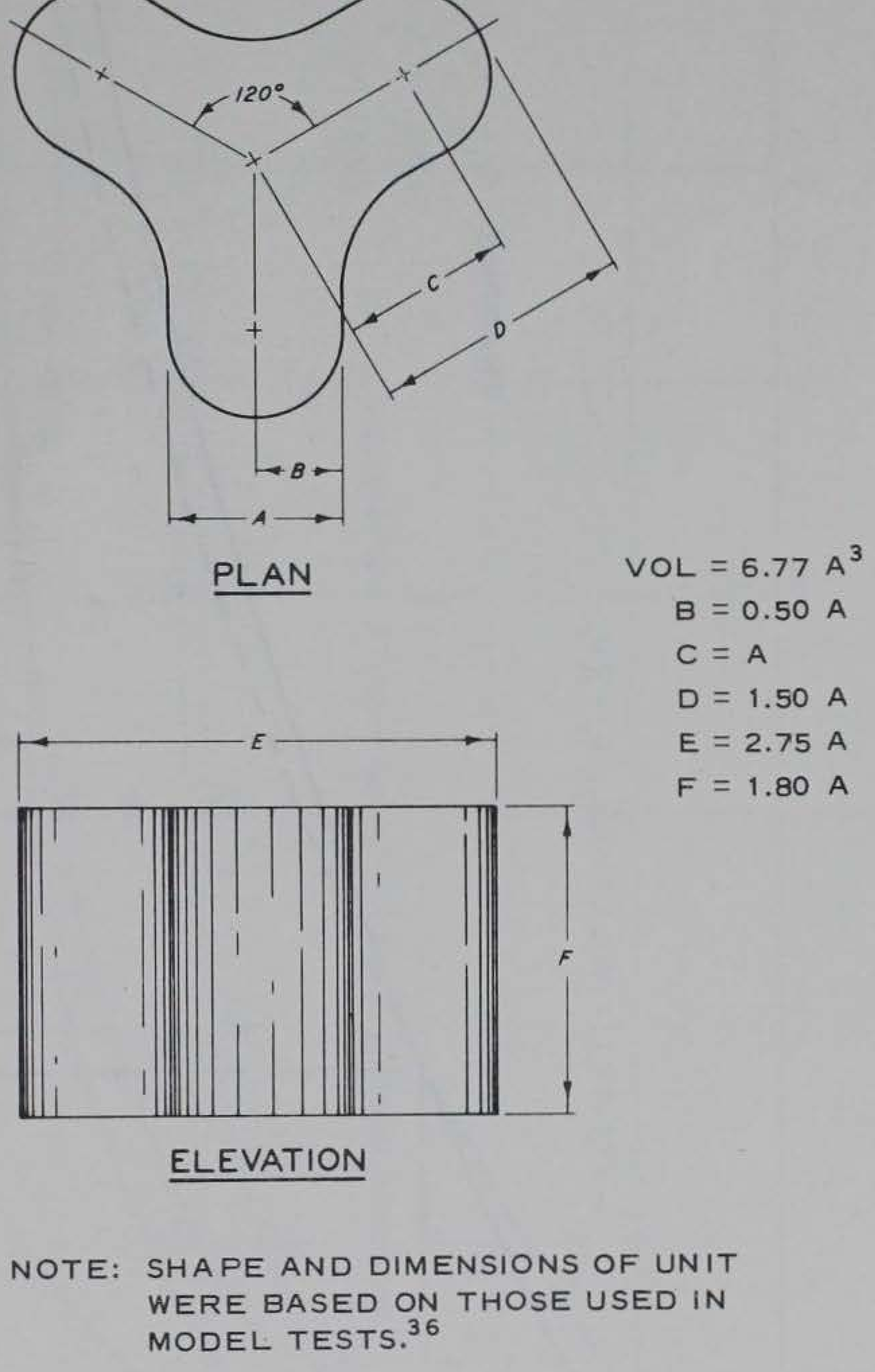


Fig. 38. Details of tri-long armor unit

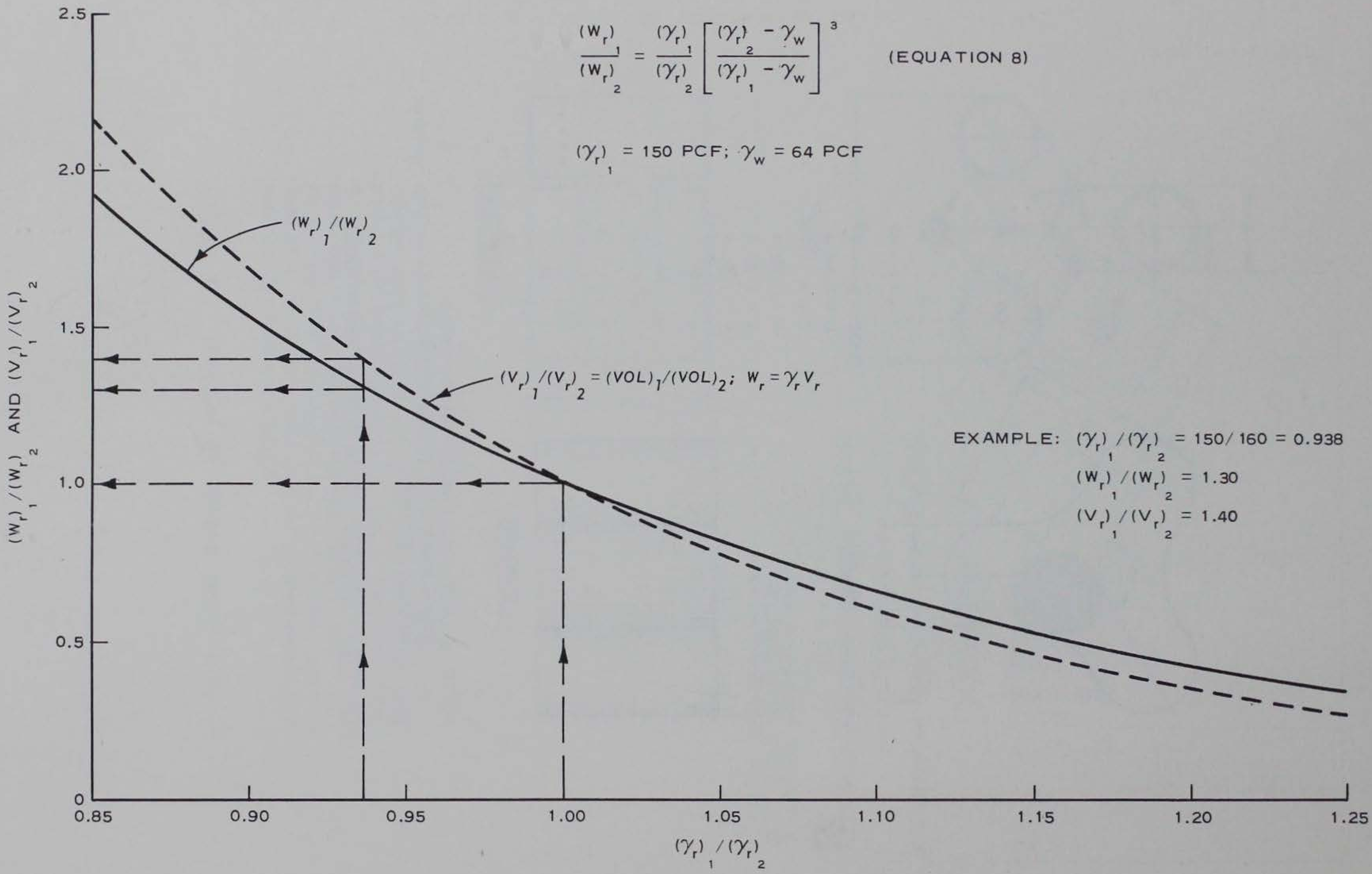
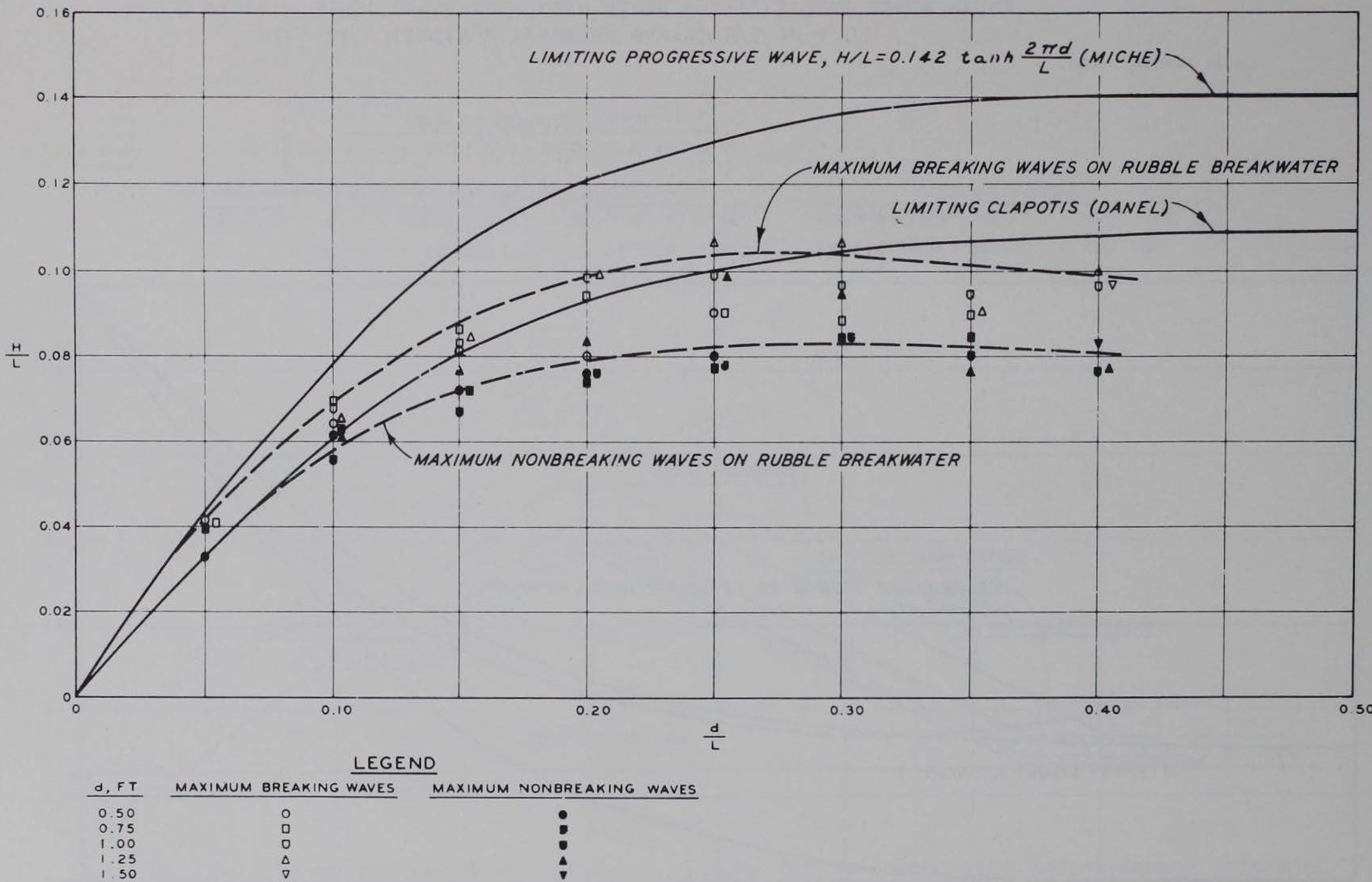
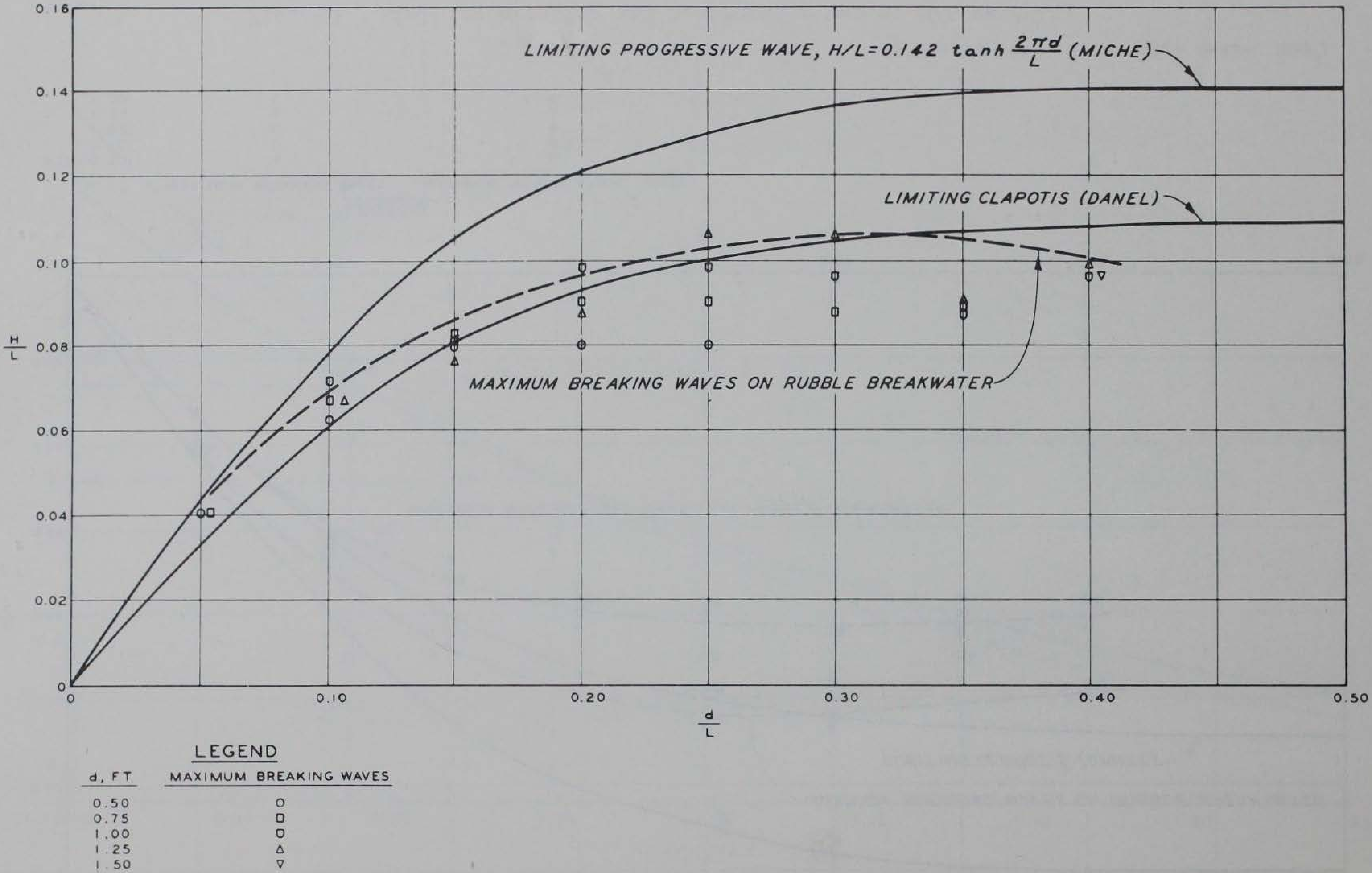


Fig. 39. Effect of specific weight on required weight and volume of armor unit



(From Jackson, 1968[5])

Fig. 40. Limiting breaking and nonbreaking waves, H/L vs d/L ; beach slope = 1:50, breakwater side slope = 1:1.5, tribar armor units



(From Jackson, 1968[5])

Fig. 41. Limiting breaking waves, H/L vs d/L ;
beach slope = 1:50, breakwater side slope = 1:3, tribar armor units

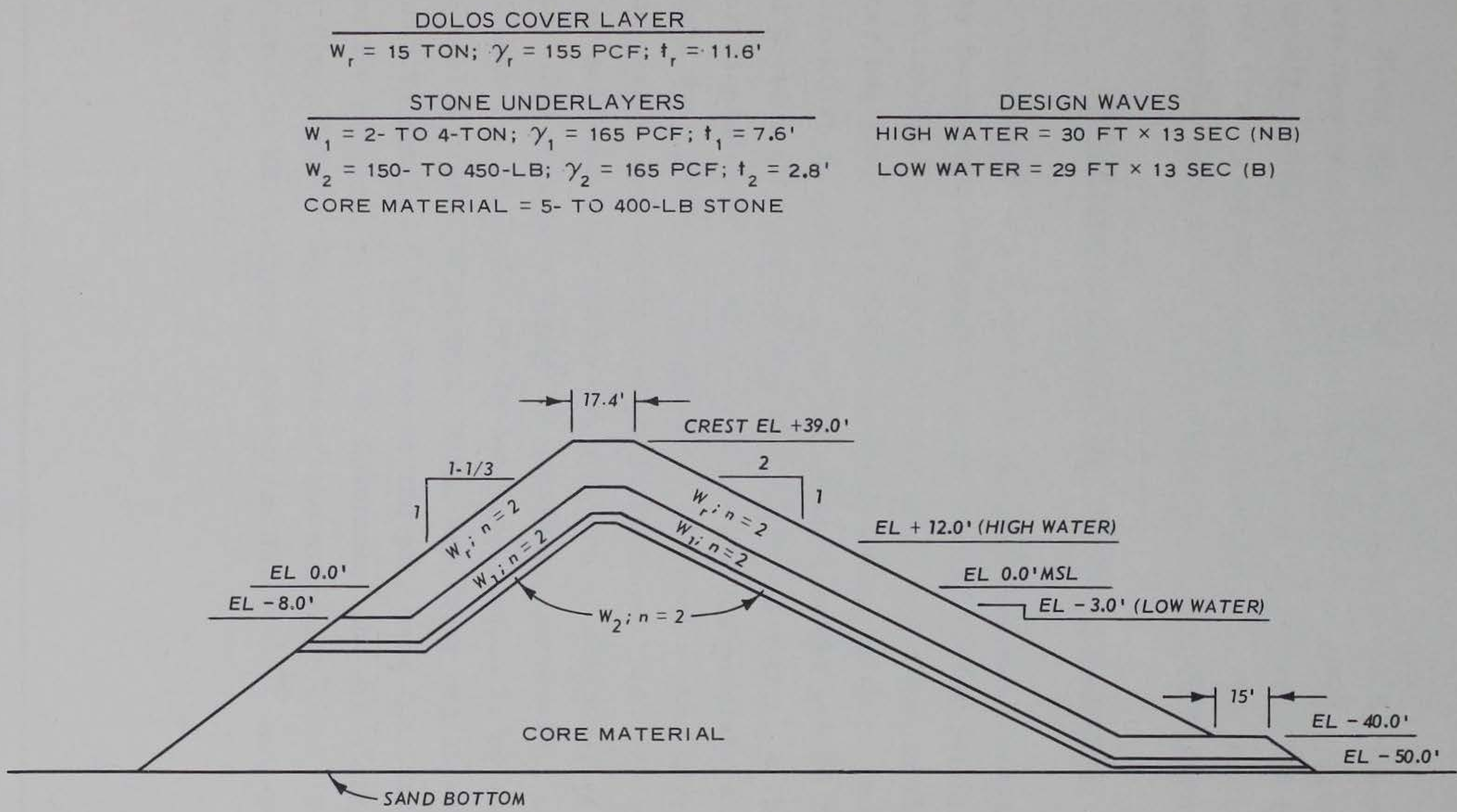


Fig. 42. Details of rubble-mound breakwater section for no-overtopping and no-damage criteria

APPENDIX A: CORPS OF ENGINEERS EXPERIENCE WITH CONCRETE ARMOR UNITS

1. Since the invention of the tetrapod in 1950, concrete armor units have been used extensively throughout the world. Some concrete units, usually cubes or tetrahedrons, were used to protect rubble-mound structures from storm-wave action before 1950.[39*] The following summaries describe some of the Corps experience with such armor units.

Breakwater, Crescent City Harbor, California

2. The outer 560-ft reach of the rubble-mound breakwater at Crescent City Harbor was constructed in 1957 using 25-ton tetrapods. The cross-section details of the structure trunk are shown in fig. A1. This breakwater has been subjected to several storms in which the waves were from 18 to 20 ft in height. The design-wave height was 23 ft. The tetrapod portion of structure was not damaged severely, although there was minor displacement of units on the trunk and moderate displacement on the head due, primarily, to rolling. Several of the unreinforced tetrapods were broken during a storm in February 1960. In 1964 the breakwater head was repaired by the addition of seventy-five 25-ton tetrapods. The structure design is considered successful, and it is believed that the stability of the tetrapod section is adequate to resist the action of future storms. Seven top forms and 35 bottom forms, dimensioned to obtain 25-ton units at 150 pcf, were fabricated. These forms are the property of the U. S. Government and are stockpiled at Crescent City, Calif. The results of the hydraulic model investigations are presented in WES Technical Memorandum No. 2-413[40] and WES Miscellaneous Paper No. 2-171.[41]

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* See Literature Cited at end of main text.

Breakwater, Nawiliwili Harbor, Hawaii

3. Nawiliwili Harbor is located on the southeast coast of the Island of Kauai, Hawaii. The breakwater was authorized in 1919 and construction was completed in 1930. It is a rubble-mound structure about 2150 ft in length,[42] and originally was composed of core stone averaging about 500 lb each with a single layer of armor stones weighing about 8 tons each. The armor stones were keyed and fitted together so well that they withstood the attack of storm waves for a period of about 20 years with only minor damage. However, severe damage occurred in each of the years 1954, 1956, and 1957; and a portion of the breakwater was reconstructed in 1959 as a part of the necessary repair work. A typical cross section of the tribar-protected portion of the structure is shown in fig. A2. The structure was designed for waves 24 ft in height and 18-ton tribars were used. Since that time, the structure has been subjected to hurricane-generated waves 19-21 ft in height and damage was not severe. One post was broken off, one tribar was lost, and other tribars sustained minor damage. Since the structure was repaired in 1959, no maintenance work has been required and the tribar cover-layer repair is considered to be successful. The forms for the 18-ton tribars are the property of the contractor (Hawaiian Dredge and Construction Co.). The results of the hydraulic model investigation are presented in WES Miscellaneous Paper No. 2-377.[43]

Breakwater-Jetty, Santa Cruz Harbor, California

4. The small boat harbor at Santa Cruz was constructed in 1962-63. The west breakwater-jetty was armored with 28-ton quadripods, as shown in Plate 1 of EM 1110-2-2904; a cross section of the structure is shown in fig. A3. The quadripods were designed to be 25 tons; however, due to the form dimensions and the density of the aggregate used, the average weight was 28 tons. Some settlement in short segments has occurred due to scour. However, the structure is in good condition and is considered successful. The forms for the quadripods, 8 top sections and

48 bottom sections, are the property of the Granite Construction Company of Santa Cruz, Calif., and are available for further use. An Engineering Studies surveillance study has been in progress at this site to determine the effectiveness of design, and a report will be prepared upon completion of the study. The results of a four-year study of the stability of the structure are given in a paper by Weymouth and Magoon.[44]

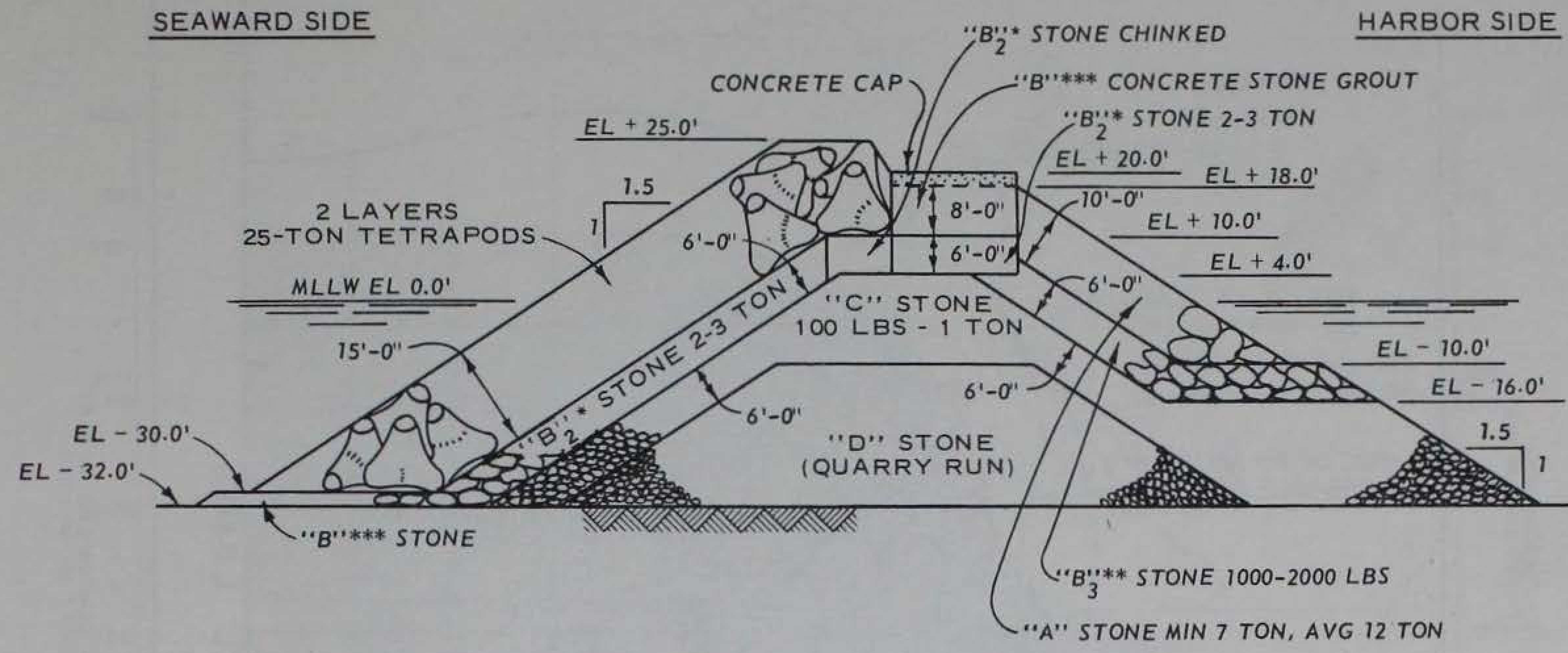
Breakwater, Kahului Harbor, Hawaii

5. Kahului Harbor is located on the northern coast of the Island of Maui, Hawaii. Extension of the existing, privately constructed east breakwater was authorized in 1910; construction of the west breakwater was authorized in 1916; and extensions of both breakwaters were authorized in 1927. Both were rubble-mound structures and were completed in 1931. The east breakwater was 2396 ft long and the west breakwater was 2850 ft long; the side slopes were 1 on 1-1/2 and 8-ton stone armor units were used. After major damage to the seaward ends of the breakwaters and minor damage to the trunk of the east breakwater in 1947, and moderate damage to the seaward ends of both structures in 1952, the structures were repaired using stone armor units. Major damage to seaward ends of both structures occurred again in 1954, and they were repaired in 1957 using 33-ton tetrapod armor units randomly placed two layers thick, using a 1-on-3 slope on the seaward side with a transition to a 1-on-2 slope on the landward side of the breakwater heads. The design wave was 34 ft in height. Another very severe storm occurred in 1958 and waves estimated to be 25 ft in height attacked the Kahului breakwaters.[45,46] According to Palmer,[46] about 30 of the 33-ton tetrapods were rolled away from the inboard quadrant of the west breakwater head, and some of them rolled as far as 100 ft. Three of the units were broken. A hydraulic model study was conducted in 1962[19] to determine plans for repair of the damaged structures. Figs. A4 and A5 show the elements of the recommended plans for the breakwater heads. Thirty-five- and fifty-ton tribar armor units were

used. Reconstruction of the breakwaters has been completed, and a surveillance study is planned to determine the stability of the revised structures. The results of this investigation will be published.

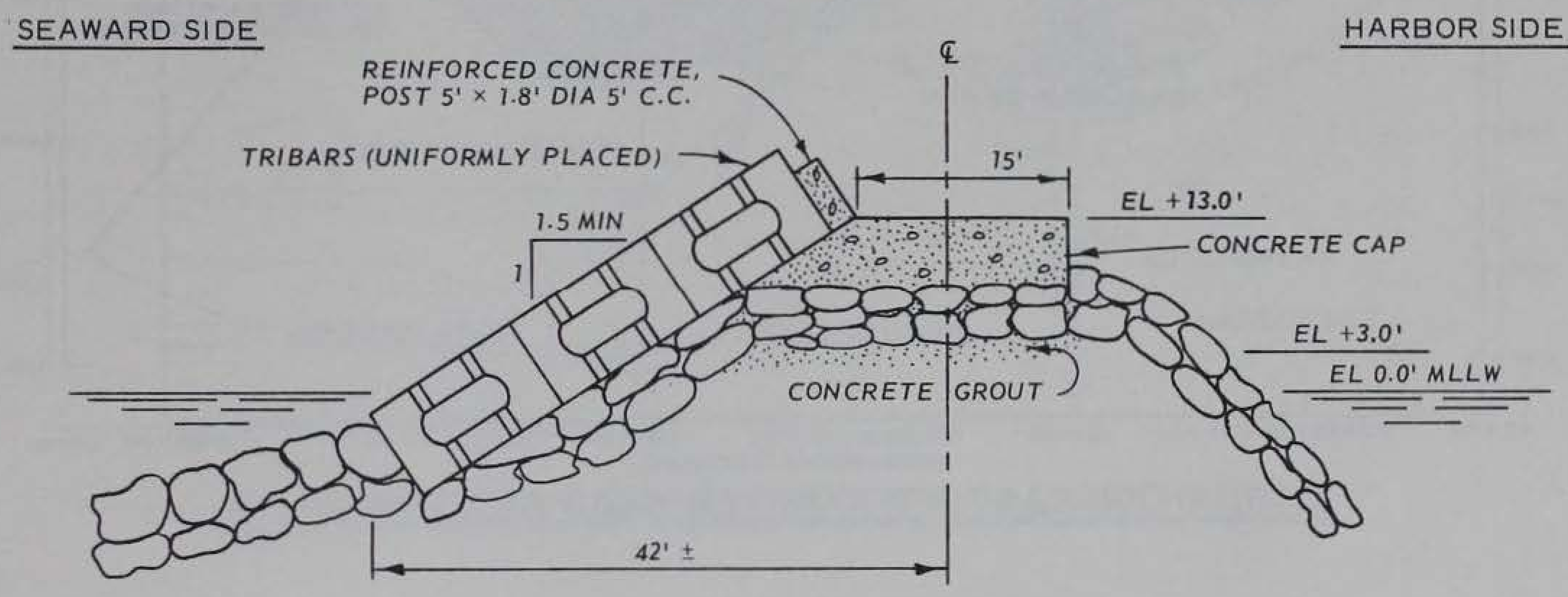
Jetties, Humboldt Bay, California

6. Humboldt Bay is located on the California coast about 280 miles north of San Francisco and about 80 miles south of Crescent City, Calif.[36] The first Corps of Engineers project for the improvement of Humboldt Bay was adopted by the River and Harbor Act of 3 March 1881. Construction of the south jetty began in 1889 and the north jetty was begun in 1891. The original jetties have since been entirely rebuilt. Construction of the authorized project was completed in 1939. The two jetties were of rubble-mound construction. The north jetty was about 4500 ft in length and the south jetty was about 5100 ft in length. Both jetties have been damaged and repaired several times since their original construction. The construction of large concrete monoliths on the heads of the north and south jetties was completed in 1961 and 1963, respectively. The south jetty head was protected by 100-ton concrete cubes. The cubes have been moved downslope by storm waves and both monoliths were severely damaged. A hydraulic model study was conducted during the period 1968-1970[36] to develop plans for repair of the damaged jetty heads. It was determined that, after the monoliths had been rebuilt, they should be protected by two layers of 42-ton, 155-pcf dolosse, randomly placed. The design waves were of the critically breaking type, 40 ft in height at high tide, 31 ft in height at low tide, and 16 sec in period. Fig. A6 shows the section, developed by model testing, that was used as a basis for prototype design.[38] Minor breakage of both reinforced and unreinforced dolosse has occurred at this site.



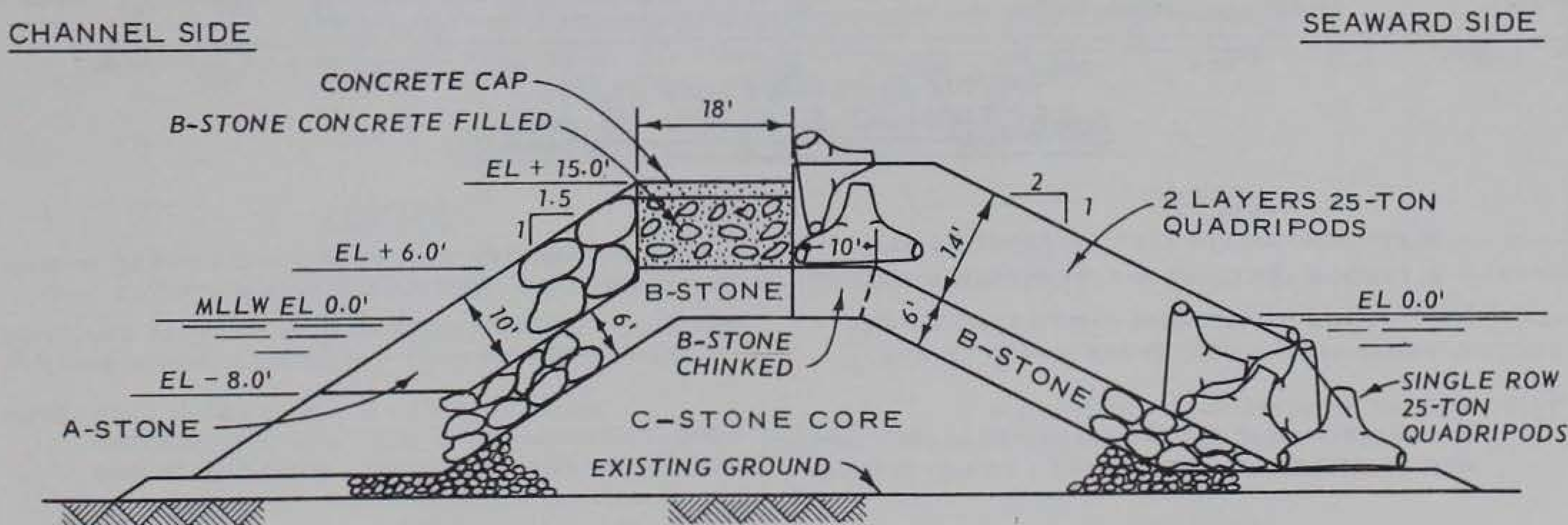
(From Coastal Engineering Research Center, Technical Report 4, 1966[22])

Fig. A1. Crescent City Harbor breakwater



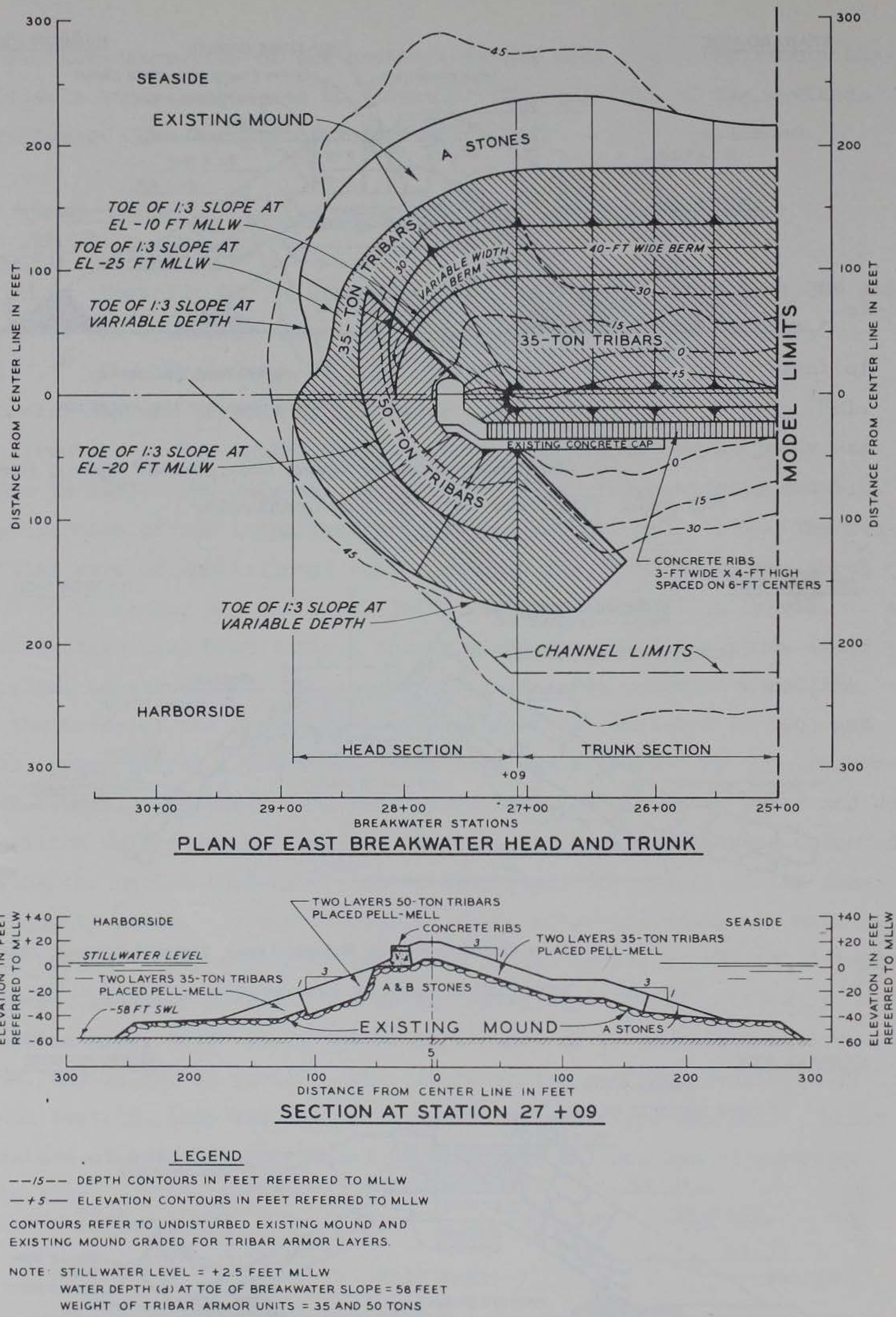
(From Coastal Engineering Research Center, Technical Report 4, 1966[22])

Fig. A2. Nawiliwili Harbor breakwater



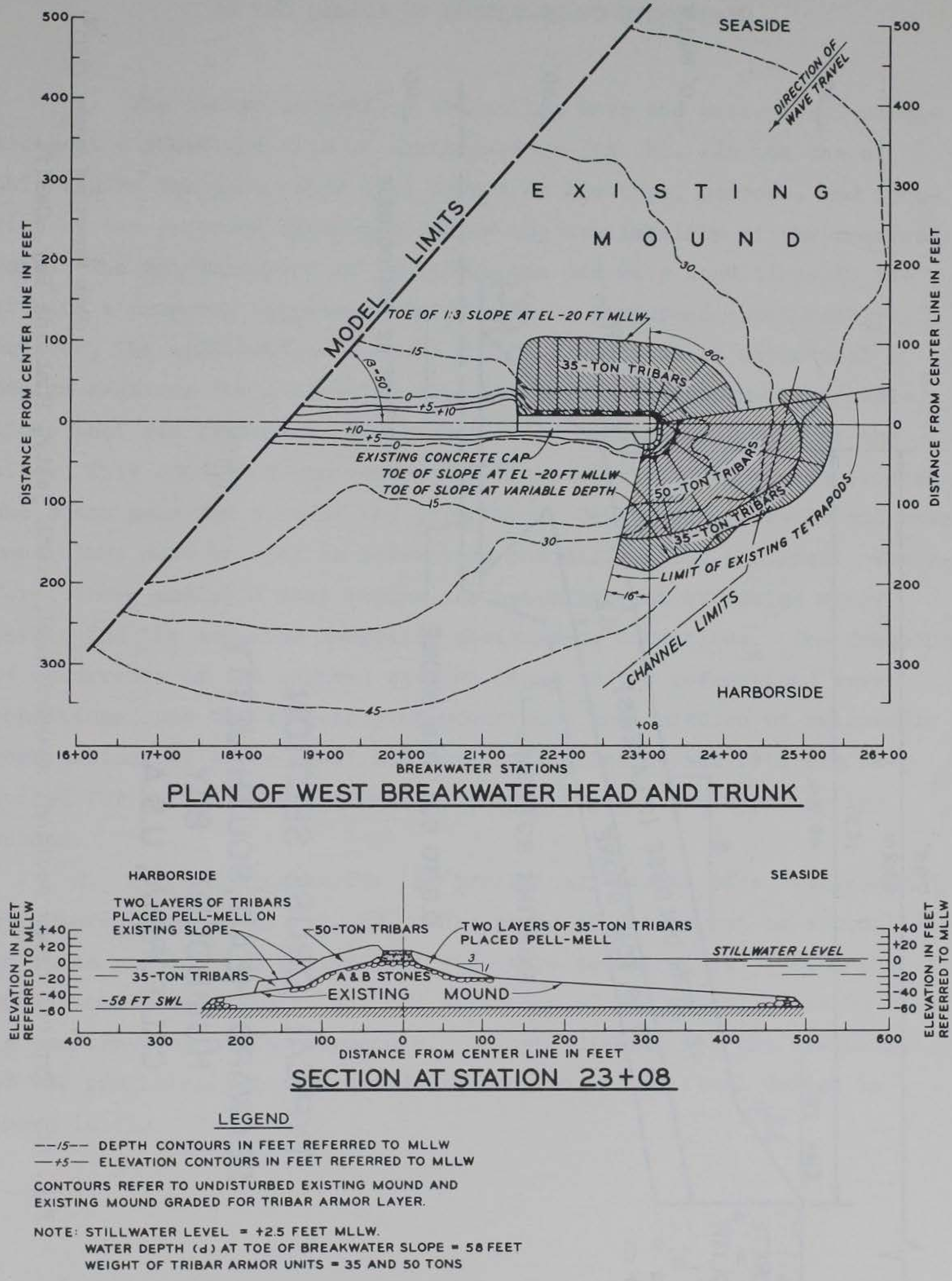
(From Weymouth and Magoon, 1968[44])

Fig. A3. Santa Cruz Harbor breakwater



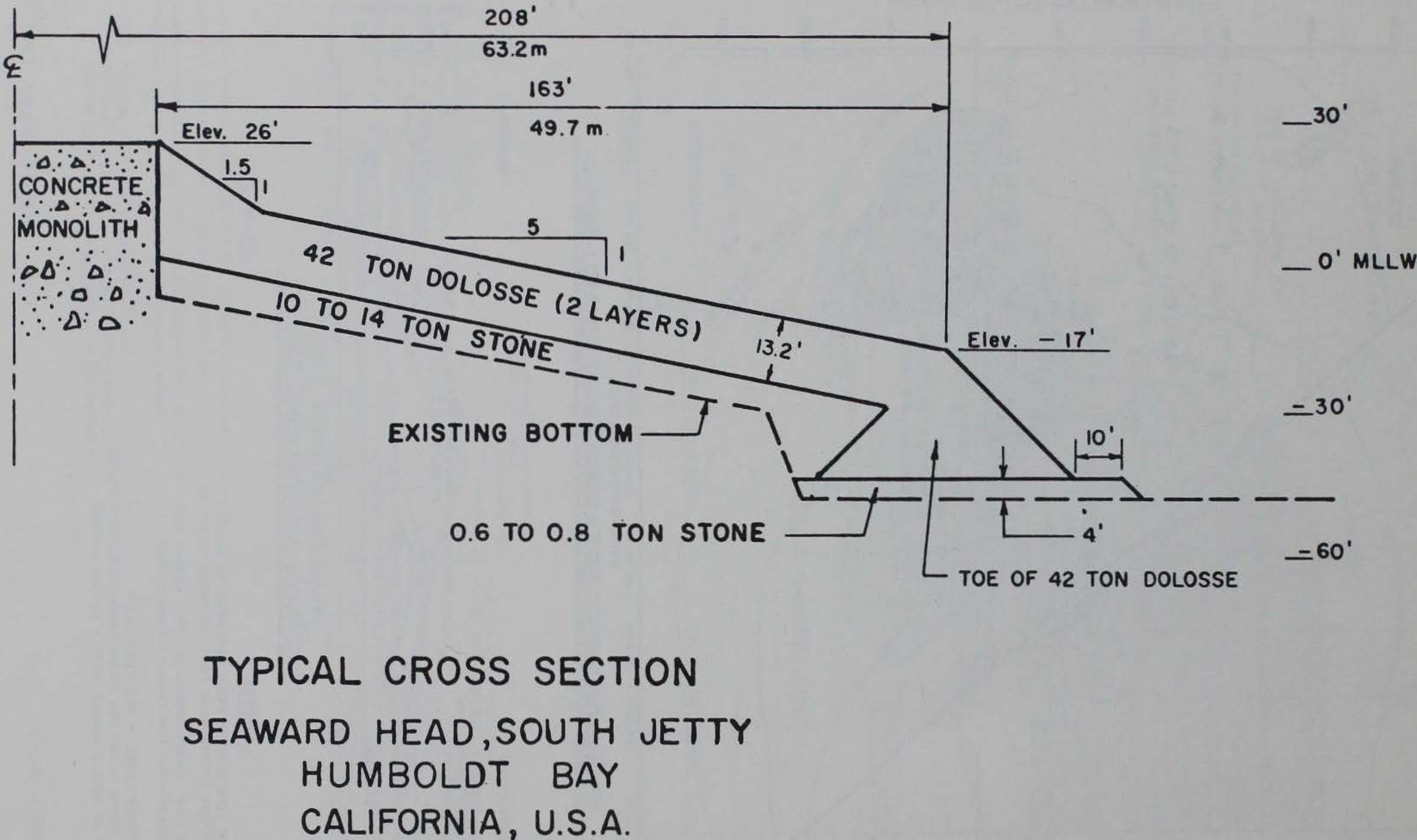
(From Jackson, 1964[19])

Fig. A4. Kahului Harbor; elements of test section, east breakwater head



(From Jackson, 1964[19])

Fig. A5. Kahului Harbor; elements of test section, west breakwater head



(From Magoon and Shimizu, 1971[38])

Fig. A6. Typical cross section, seaward head, south jetty, Humboldt Bay, Calif.

APPENDIX B: FLOW DIAGRAMS FOR DECISION-MAKING IN THE DESIGN OF RUBBLE-MOUND STRUCTURES

1. The design process of evaluating wave and water-level conditions at a structure site is summarized in fig. B1. In the use of this figure the path taken will depend on the type, purpose, and location of the proposed structure and on the availability of the required data. The determination of design depths and wave conditions at the site of a proposed structure can usually be performed concurrently. However, the application of these design conditions to structural design requires the evaluation of water levels and storm-wave conditions that can reasonably be assumed to occur simultaneously at the site. This condition can occur, for example, when a hurricane crosses the coast near the site of the structure. Design water levels and wave conditions must be used in refraction and diffraction analyses. Therefore, these analyses must follow the establishment of design water levels and the selected deepwater design-wave conditions. The frequency of occurrence of the adopted shallow-water (after refraction) wave conditions, and the frequency of occurrence and duration of reasonable combinations of water level and wave conditions at the site are required for an adequate evaluation of any proposed shore-protection scheme.

2. A logic diagram for the preliminary design of a rubble-mound structure is shown in fig. B2. This phase of design can be accomplished in three parts: (a) preliminary structure geometry, (b) evaluation of construction techniques, and (c) selection of the materials to be used in construction of the structure. A logic diagram for the evaluation of the preliminary design and determination of the final design is shown in fig. B3.

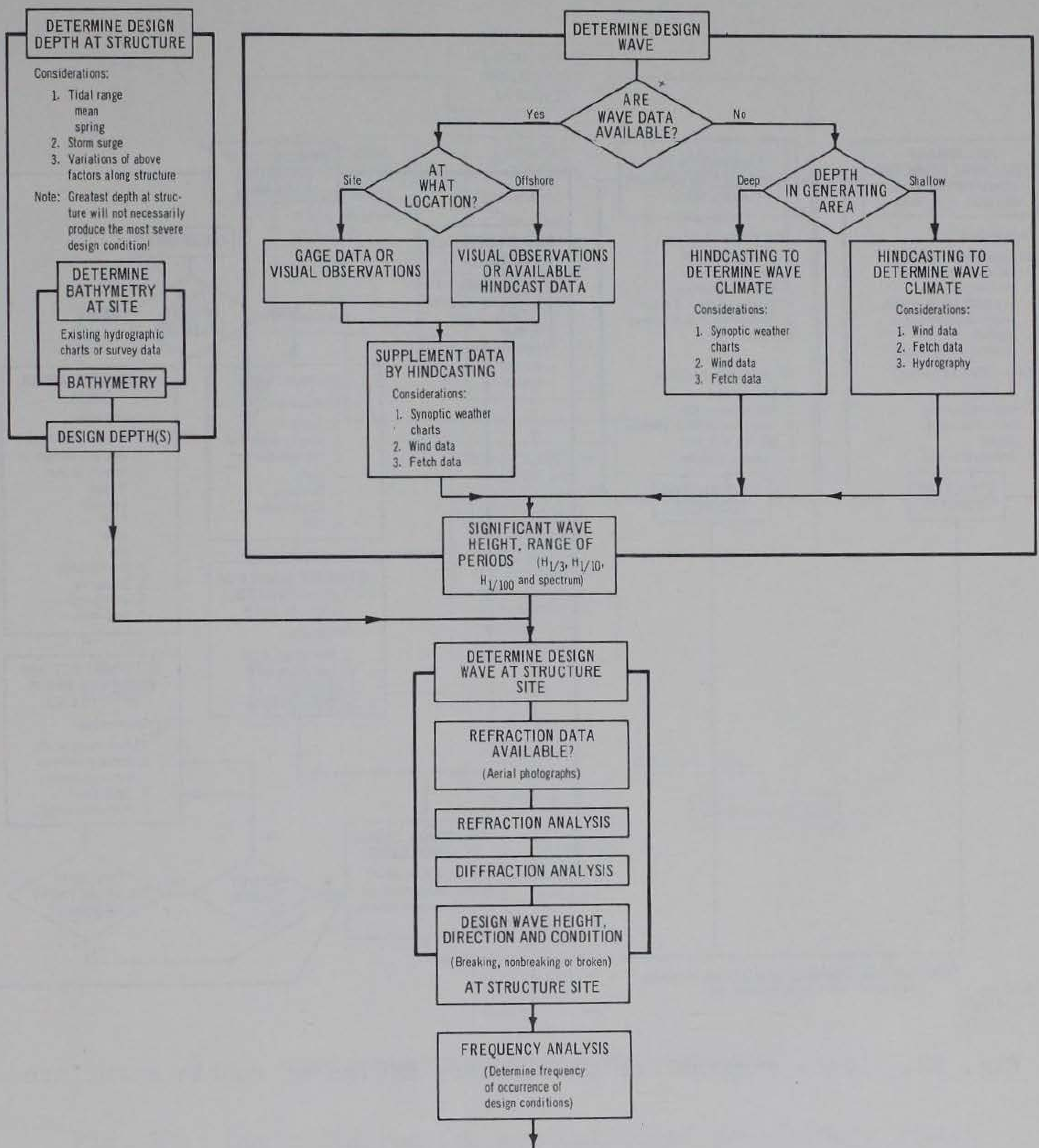


Fig. B1. Logic diagram for evaluation of marine environment

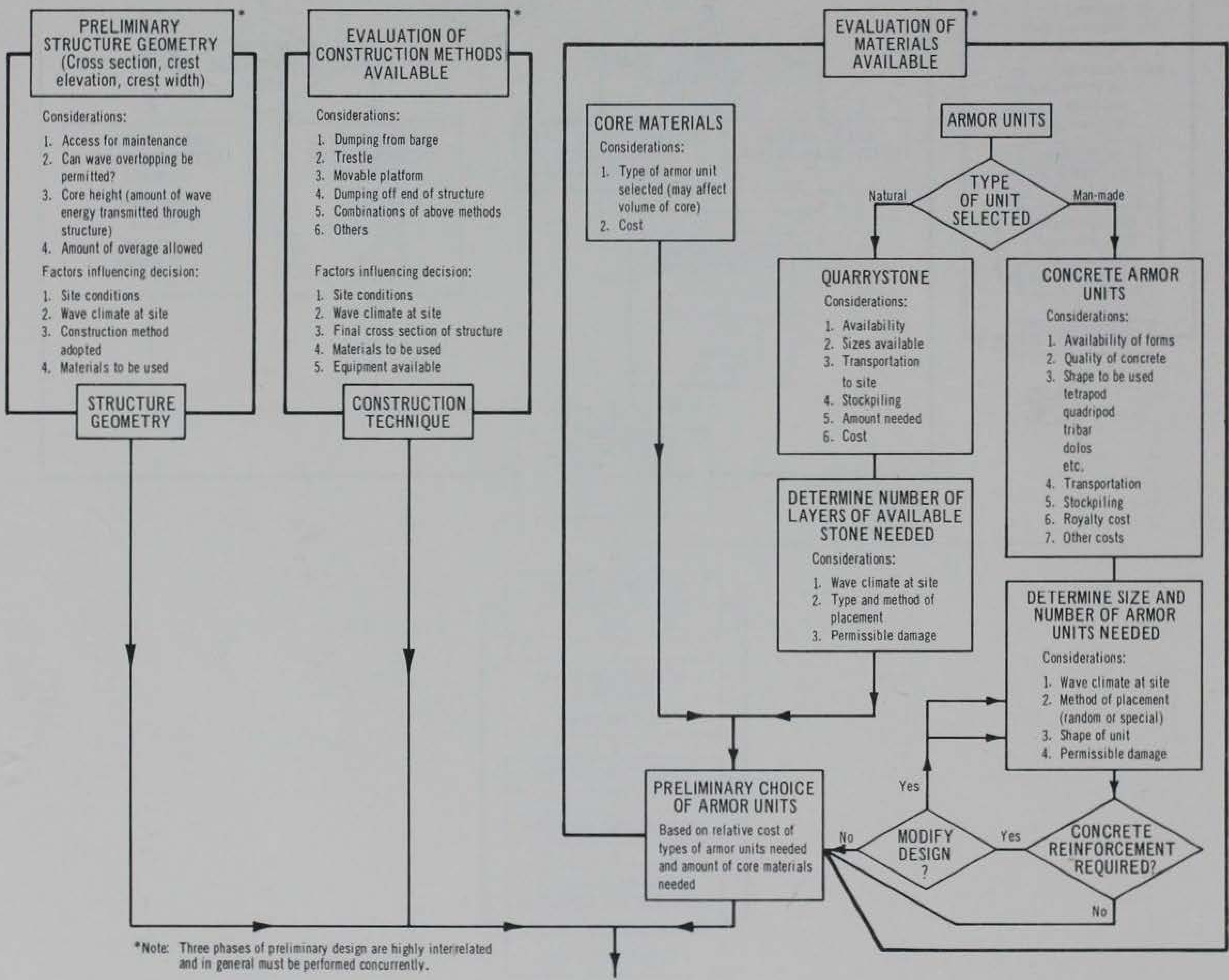
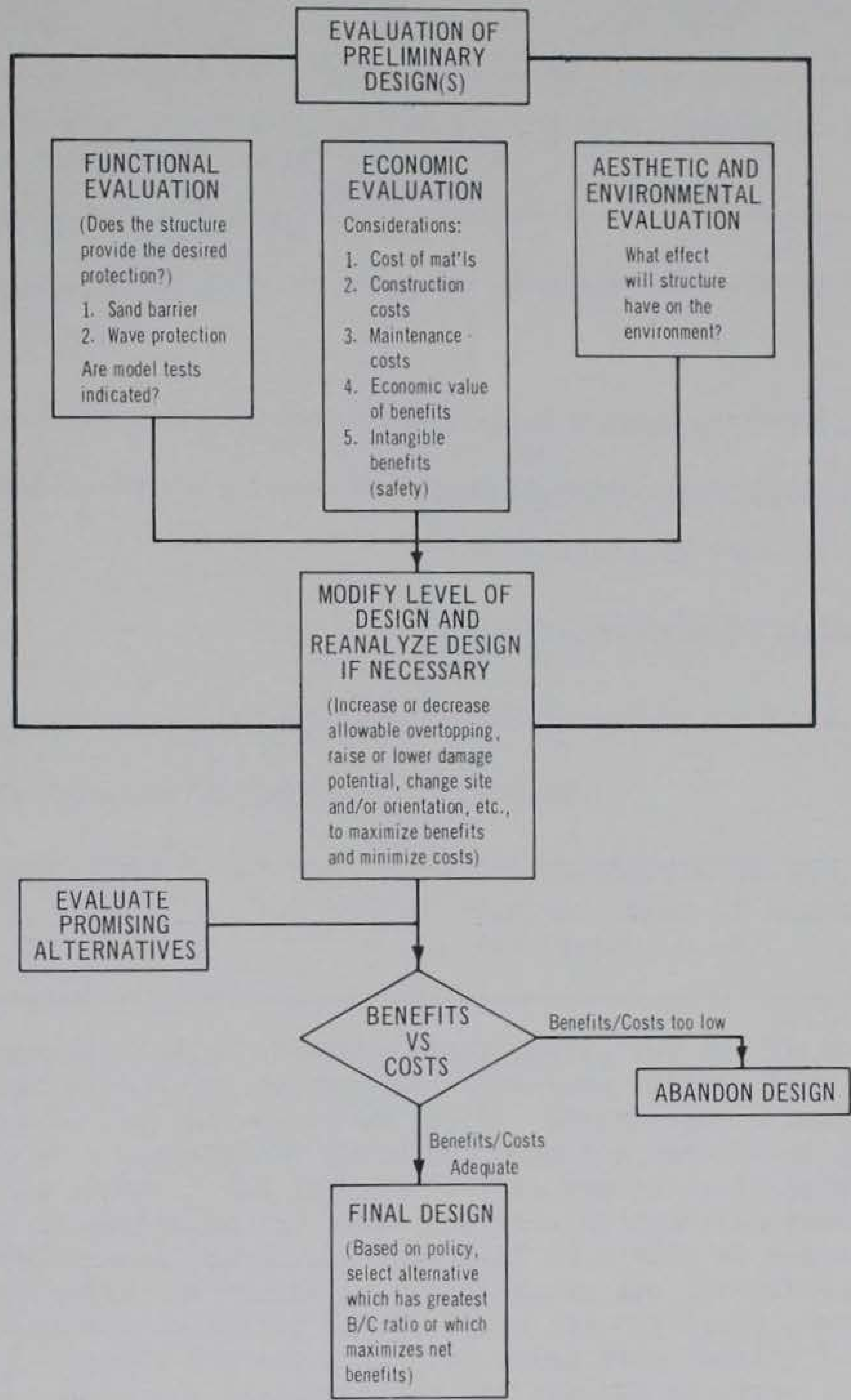


Fig. B2. Logic diagram for preliminary design of rubble structure



LOGIC DIAGRAM FOR EVALUATION OF PRELIMINARY DESIGN

Fig. B3. Logic diagram for evaluation of preliminary design and determination of final design

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13. ABSTRACT

A large number of special-shaped concrete armor units, for use in the protective cover layer of rubble-mound structures exposed to storm-wave action, have been developed throughout the world in the past 20 to 25 years. Quarrystone armor units may be used when available at a competitive price and when the wave conditions at the structure site are not too severe. The purpose of this report is to provide design information and guidance in selecting the shape and size of concrete armor units for use in constructing rubble-mound structures that will be stable at a minimum of cost. The factors that determine the choice of design waves are described; the development of a stability formula and stability coefficients for different amounts of damage to the structure are presented; the advantages of using high-density concrete in the fabrication of armor units are stressed; and the casting of armor units, the problems of breakage, and the legal aspects of using concrete armor units of special shape are discussed. Based on the results of test data available to date, it was concluded that the tetrapod, tribar, and dolos armor units should be considered for use in the design of rubble-mound breakwaters and jetties when the use of quarrystone is not feasible. The dolos armor unit is believed to be the most efficient, and procedures for the design of a typical breakwater cross section, using these armor units for the protective cover layer, are presented. The use of hydraulic model studies to determine the optimum design of proposed breakwaters is discussed. Appendices A and B present summaries of the Corps of Engineers experience in the use of concrete armor units, and flow diagrams for decision-making in the design of rubble-mound structures.

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	ROLE	WT	ROLE	WT	ROLE	WT
Armor units (shore protection)						
Breakwaters						
Coastal structures						
Concrete armor units						
Rubble-mound breakwaters						
Shore protection						
Water wave forces						