CHAPTER 7
RUBBLE MOUND DESIGN

Rubble mounds are gravity structures which derive their stability largely from the weight of the armoring units which cover and protect the core. The entire structure is typically graded, in layers, from the large exterior stone or concrete armor units, through two or more layers of intermediate sized materials, to small quarry run sizes at the core and finer material beneath it. The design of these structures may be considered in three phases, as shown in a flow diagram, Figure 7.1:

1. Structural Geometry
2. Construction Planning
3. Evaluation of Materials

These delineations are solely for the simplicity of discussion; these elements are inherently interrelated and must be addressed concurrently. The first two aspects are the subject of this chapter. Chapter 6 is devoted to the evaluation of rubble mound construction materials, particularly those used as cover layer armor units.

Structure geometry is considered in two subsections, cover layer stability and cross-section design. Mounds derive their stability from the hydraulic stability of the protective cover layer. The careful selection of cover layer armor unit characteristics constitutes a major portion of the design effort. Empirical methods have been developed which give a satisfactory representation of cover layer stability. These methods, their applicability and limitations, are the focus of the first section.
Figure 7.1a Flow Diagram for the Design of Rubble Mound Structures (CERC, 1977, p. 7-205)
Figure 7.1a

EVALUATION OF PRELIMINARY DESIGNS

FUNCTIONAL EVALUATION
Does the structure provide the desired protection?
1. Sand barrier
2. Wave protection
Are model tests indicated?

ECONOMIC EVALUATION
Considerations:
1. Material costs
2. Construction costs
3. Maintenance costs
4. Value of benefit
5. Intangible benefits (safety)

AESTHETIC AND ENVIRONMENTAL EVALUATION
Effect of structure on the environment

MODIFY LEVEL OF DESIGN AND REANALYZE DESIGN IF NECESSARY
Increase or decrease allowable overtopping, reflux or lower damage protection, change site and/or orientation, etc., to increase benefits and reduce costs

EVALUATE PROMISING ALTERNATIVES

BENEFITS VERSUS COSTS
Benefits/Costs Too Low
Benefits/Costs Adequate
ABANDON DESIGN

FINAL DESIGN
(Based on policy, select alternative which has greatest benefit-cost ratio that increases net benefits)

Figure 7.1b  Flow Diagram for the Design of Rubble Mound Structures (CERC, 1977, p. 7-206)
There are an infinite number of possible rubble mound cross-section designs, the variations depending on the precise combination of wave climate, structure orientation, water depth, material availability, foundation conditions, construction techniques, and the purpose of the structure and degree of protection it must provide. As it is impossible to cover every alternative, the second section considers the basic elements of cross-section geometry and provides general principles and guidelines. Recommendations are largely those of the U.S. Army Corps of Engineers (CERC, 1977), supplemented where appropriate. The designs suggested apply to breakwaters and jetties where the seaward slope is exposed to significantly more wave attack than the relatively sheltered leeward side. When both sides receive similar wave action, as with groins and some jetties, both sides should be of similar design.

The ideal final design of a rubble mound is a cross-section that will meet the functional requirements of the structure at a minimum of costs. The problems involved in arriving at the optimum design are substantial. For relatively small structures, on the order of those considered in this report, the design and cost estimate analyses are usually made using all available information in the literature and an experience base. For large, expensive structures, it is common to perform hydraulic model studies including:

1. A three-dimensional harbor wave action model study to evaluate optimum length and orientation of the proposed structure
2. A two-dimensional rubble mound stability model study to determine the optimum structure cross-section
For very large waves and unusual, complex structure shapes, three-dimensional stability model investigations may be necessary (Hudson, 1974).

Practical construction considerations are as important as stability theories in rubble mound design. The cross-section design and construction scheme must evolve simultaneously, through cooperation of designers and builders. Both land-based and floating equipment are used in rubble mound construction. The precise methods and sequence of operations specified depend on the location and design of the mound, site conditions, and equipment availability. Details of construction practice are highlighted in the last section.

7.1 COVER LAYER HYDRAULIC STABILITY

Rubble mounds are heterogeneous assemblages of discrete units and are not amenable to analytical treatment (Wang, 1977). At present, it is not possible to quantify the forces required to displace individual armor units from the cover layer. Units may be displaced en masse, by sliding down the slope, or individually lifted and rolled down or up the slope (CERC, 1977).

The cover layer armor unit weight is perhaps the most important single parameter in assuring rubble mound stability against wave attack. The current state-of-the-art of rubble mound design dictates calculation of individual armor unit weights from stability formulas, and verification by hydraulic model studies where economically possible. This design process provides satisfactory design, but has many
limitations. These are presented at the end of this section and are themselves as important as the formulas to which they relate.

**Stability Formulas**

There are more than a dozen formulas in the literature for computing the required weights of armor units. These semi-empirical methods have evolved predominantly on the basis of small-scale model tests and involve simplifications of field conditions. Generally, wave action is considered as the sole destructive environmental force, and the actions of currents, wind and ice (See Chapter 5) are neglected. It is important, then, to understand the range of applicability and limitations of each method. It is stressed that all the stability formulas currently employed are design 'guides, rather than absolute principles, and their use must accordingly be tempered with judgment and experience.

Most of the stability formulas now in use indicate the dependence of armor unit weight on wave characteristics, the specific weights of the armor units and water, and the seaward mound slope in the form:

\[
W = \frac{H^3 \gamma_r}{K_D (S_r - a)^b f(\theta)}
\]

(7.1)

in which: \( W \) = weight of individual armor units; \( H \) = design wave height; \( \gamma_r \) = unit weight (saturated surface dry) of armor unit; \( S_r = \gamma_r / \gamma_w \), specific gravity of armor unit, relative to the water at the structure. \( \gamma_w \) = unit weight of water, fresh water = 62.4 pcf, sea water = 64.0 pcf; \( \theta \) = angle of seaward slope measured from horizontal; \( K_D \) = stability coefficient. The form of the function \( f(\theta) \) depends on the force
The variables \( a \) and \( b \), and the values of \( K_D \), are unique for a specific set of experimental conditions. Among the stability formulas, that developed by Hudson (1959) is the most popular and will be highlighted in this report. The Hudson formula was derived based on earlier works by Iribarren (1938). It was assumed that the drag force is the primary force acting to dislodge individual armor units from the slope, and that the major force opposing this movement is the buoyant weight of the unit submerged in still water (Hudson, 1974). The form of Equation 7.1, determined through numerous tests performed at the U.S. Waterways Experiment Station (WES) and limited full-scale verification, was developed with two fundamental simplifications (Hudson, 1959):

1. The crown elevations of the test structures were of sufficient height to prevent major overtopping.

2. Design wave heights chosen caused less than 5 percent damage; that is, less than 5 percent by volume of armor units in the test section were displaced, and the stability of the section was not affected.

These testing conditions are together referred to as the no-damage, no- (or minor-) overtopping criteria. The widely used formula derived under these constraints is:

\[
W = \frac{H^2 Y_T}{K_D (S_r - 1)^3 \cot \theta}
\]  

(7.2)

The following restrictions should be observed in applying this formula (CERC, 1977):

1. \( W \) is the weight of armor units of nearly uniform size. For quarrrystone, the sizes can range within 0.75 to 1.25\( W \), with 75 percent of the individual stones weighing more than \( W \).
2. The cover layer slope angle, $\theta$, is partly determined on the basis of stone sizes economically available. The formula is for structures with uniform slope between 1 on 1.5 to 1 on 3.

3. The formula is for monochromatic waves approaching at right angles to the structure.

4. The specific weight of the armor units should be within the range of 120 to 180 pcf (19 to 28 kN/m$^3$).

5. The values of $K_D$ should not exceed those recommended; the selection of stability coefficients and additional constraints regarding their use are covered subsequently.

Figures 7.2 through 7.5, generated by CERC (1977), enable a graphical solution of Equation 7.2. Another quick graphical method is shown in Figure 7.6. The use of these charts is illustrated by Design Example 7.1.

Selection of Stability Coefficient

The dimensionless coefficient, $K_D$, represents the combined effect of all influencing variables not directly evaluated in Equation 7.2. The most important contributing factors are (CERC, 1977):

1. Shape of armor unit
2. Number of layers
3. Manner of unit placing (random or special)
4. Friction and interlocking of units
5. Wave shape (breaking or nonbreaking)
6. Part of structure (trunk or head)
7. Angle of incidence of wave attack

Extensive small-scale tests have been conducted at the WES to determine values of $K_D$. Most tests were performed on idealized breakwater trunk sections with large water depths, relative to wave height, using nonbreaking monochromatic waves with no overtopping and
Figure 7.2 Armor Unit Weight \( k \) versus Wave Height for Various Slope Values

\( \gamma = 140 \text{ pcf} \) and \( 145 \text{ pcf} \) (CEMC, 1977, p. 7-182)
Figure 7.3: Armor Unit Weight x \( K_n \) versus Wave Height for Various Slope Values

\( \gamma_r = 150 \text{ pcf} \) and 155 pcf (CEC, 1977, p. 7-183)
Figure 7.4 Armor Unit Weight x $K_D$ versus Wave Height for Various Slope Values
($\gamma_r = 160$ pcf and 165 pcf) (CERC, 1977, p. 7-184)
Figure 7.5 Armor Unit Weight $\times K_p$ versus Wave Height for Various Slope Values
($\gamma_r = 170$ pcf and 175 pcf) (CERC, 1977, p. 7-185)
Figure 7.6 Graphical Solution to Hudson's Formula for Rubble Mound
Cover Layer Stability (ASCE, 1969, p. 70)
DESIGN EXAMPLE 7.1

DETERMINATION OF ARMOR UNIT WEIGHT

GIVEN: ROUGH ANGULAR QUARRYSTONE JETTY, QUARRYSTONES PLACED RANDOMLY WITH n = 2 (2 UNITS THICK) ON THE COVER LAYER
\( \gamma_r = 160 \) PCF
DESIGN BREAKING WAVE HEIGHT \( H = 8 \) FT; NO DAMAGE CRITERION
CUT \( \theta = 1.5 \) (1 ON 1.5 SLOPE)
FRESH WATER, \( \gamma_w = 62.4 \) PCF

REQUIRED: COVER LAYER ARMOR UNIT WEIGHT, \( w \), BY:
a) EQUATION 7.2 DIRECTLY
b) CERC GRAPHICAL METHOD (FIGURES 7.2 TO 7.5)
c) ASCE (1949) GRAPHICAL METHOD (FIGURE 7.6)

SOLUTION:

FROM TABLE 7.1, \( K_d = 3.5 \)

a) \[ w = \frac{\gamma_r H^3}{K_d (\gamma_r - 1)^3 \cot \theta} \]
\[ = \frac{160 (8^3)}{3.5 \left( \frac{160}{62.4} - 1 \right)^3 1.5} = 4878 \times 2.04 \text{ TONS} \]
\[ w = 2.0 \text{ TONS} \]

b) USING FIGURE 7.4, TOP (FOR \( \gamma_r = 160 \) PCF),
\( H = 8 \) FT, CUT \( \theta = 1.5 \),
READ \( w \times K_d = 1.6 \times 10^4 \) *

FOR FRESH WATER, MULTIPLY BY 0.875 \( \rightarrow w \times K_d = 1.4 \times 10^4 \)
\[ w = \frac{w \times K_d}{K_d} = \frac{1.4 \times 10^4}{3.5} = 4000 \times 2.0 \text{ TONS} = v \]
c) Using Figure 7.6, enter graph at left for $k_0 = 3.5$.

$H = 8 \text{ ft}$; read $V_{130} = 3.1 \text{ tons}$

Using bottom chart, for $q_f = 160 \text{pcf}$, cut $\theta = 1.5$;

Read $\%$ of $V_{150} = 77$

$\therefore V = 77 \% \times V_{130} = (0.77)(3.1) = 2.39 \text{ tons}$

For fresh water, multiply by 0.875 $\rightarrow W = 2.1 \text{ tons}$

The graphical methods yield quick solutions with satisfactory accuracy, as demonstrated. The CERC method is the superior of the two.
no damage. Some tests have been made on idealized conical breakwater heads with nonbreaking waves. Also, some tests have been run on breakwater trunks subject to breaking waves (Hudson, 1974). The sum of these efforts is tabulated in Table 7.1, the \( K_D \) values recommended for design by CERC (1977). Certain limitations in the practical application of these values should be noted:

1. A two unit thickness (\( n=2 \)) is recommended. If one layer only is used, smaller values of \( K_D \), corresponding to larger values of \( W \), must be used for design. Displacement of units on a one layer thick slope can easily expose underlayers and threaten cover layer integrity. Therefore, quality control during construction is crucial.

2. It is recommended that the random placement \( K_D \) values be used for design. It is unlikely that the high degree of interlocking of special placement could be reproduced in the field, especially below the water level.

3. CERC (1977) recommends that cover layer slopes should not be steeper than 1 on 1.5. However, Hudson (1974) notes that, in practice, leeward slopes as steep as 1 on 1.25 have been used.

4. Laboratory waves were monochromatic and did not simulate real wave conditions.

5. Test data for the breaking wave condition are limited. \( K_D \) values for armor units not tested for breaking waves were obtained by applying a reduction factor to the \( K_D \) values for nonbreaking waves.

6. Rubble mound head segments generally experience the most severe wave action and overtopping. Accordingly, \( K_D \) values for the head sections are smaller than the corresponding trunk values.

The next section, on limitations of the stability formulas, addresses additional pertinent considerations.

The \( K_D \) values listed in Table 7.1 refer specifically to the no-damage, no-overtopping criterion discussed earlier. This is a well-known, but often uneconomical and unrealistic design condition. The results of model tests conducted and reported by Hudson (1959) and
<table>
<thead>
<tr>
<th>Armor Units</th>
<th>n *</th>
<th>Placement</th>
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<th>Structure Head</th>
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<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
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<td>Nonbreaking wave</td>
</tr>
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<td></td>
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<td></td>
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<td>Nonbreaking wave</td>
</tr>
<tr>
<td></td>
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<td></td>
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<td>special ‡</td>
<td>4.8</td>
<td>5.5</td>
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<td>2.2</td>
<td>2.5</td>
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</table>

- * n is the number of units comprising the thickness of the armor layer.
- † The use of single-layer of quarrystone armor units subject to breaking waves is not recommended, and only under special conditions for nonbreaking waves. When it is used, the stone should be carefully placed.
- ‡ Special placement with long axis of stone placed perpendicular to structure face.
- § Applicable to slopes ranging from 1 on 1.5 to 1 on 5.
- ‖ Until more information is available on the variation of $K_D$ value with slope, the use of $K_D$ should be limited to slopes ranging from 1 on 1.5 to 1 on 3. Some armor units tested on a structure head indicate a $K_D$-slope dependence.
- ‖ Stability of dolosse on slopes steeper than 1 on 2 should be substantiated by site specific model tests.

Table 7.1 Recommended No-Damage $K_D$ Values for Use in Calculating Armor Unit Weight (CERC, 1977, p. 7-181)
Jackson (1968a) allow evaluation of the effect of damage on rubble mound stability. This information may be used in two ways, described in the following paragraphs:

1. To evaluate the safety factor of rubble mounds against waves higher than the no-damage design wave

2. To design the mound purposely such that some damage will occur (damage design)

The data summarized in Table 7.2 present $K_D$ values as a function of percent cover layer damage for various armor units. The percent damage, $D$, is based on the volume of armor units displaced for a significant wave height, $H$. Damage volumes for a typical test section are shown in Figure 7.7. $H_{D=0}$ is the significant wave height corresponding to the no-damage criterion, for 0 to 5 percent damage. The uses of Table 7.2, described below, are demonstrated by Design Example 7.2.

It is important that rubble mounds be designed such that they will not fail when subjected to waves moderately higher than the selected design wave height. Storm wave trains contain waves which are higher than the significant wave height, $H_s$, often specified in mound design (See Table 5.1). For the no-damage condition, then it is necessary to evaluate beforehand the effect of waves higher than the no-damage significant wave height. The frequency of occurrence of design exceeding waves can be evaluated from statistical wave data. The cover layer damage caused by these waves is evaluated with the use of Table 7.2.

If some degree of damage to the cover layer can be permitted, mound design can proceed with a damage, rather than no-damage, criterion. Larger values of $K_D$, corresponding to smaller required armor unit
<table>
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<th>Unit</th>
<th>Damage (D) in Percent</th>
<th>0 to 5</th>
<th>5 to 10</th>
<th>10 to 15</th>
<th>15 to 20</th>
<th>20 to 30</th>
<th>30 to 40</th>
<th>40 to 50</th>
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<td>Quarrystone (smooth)</td>
<td>$\frac{H}{H_{D=0}}$</td>
<td>1.00</td>
<td>1.08</td>
<td>1.14</td>
<td>1.20</td>
<td>1.29</td>
<td>1.41</td>
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<td></td>
<td>$\frac{K}{K_D}$</td>
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<td>3.6</td>
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<td>5.1</td>
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<td>$\frac{H}{H_{D=0}}$</td>
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<td>1.08</td>
<td>1.19</td>
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<td>$\frac{K}{K_D}$</td>
<td>4.0</td>
<td>4.9</td>
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<td>12.4</td>
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<td>$\frac{K}{K_D}$</td>
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<td>45.9</td>
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Breakwater Trunk, n = 2, Random Placed Armor Units, Nonbreaking Waves, and Minor Overtopping Conditions.

Table 7.2 $H/H_{D=0}$ and $K_D$ as a Function of the Cover Layer Damage Parameter, D, and Armor Unit Type (CERC, 1977, p. 7-189)
Damage $D$ is the ratio of the amount of material removed from the core of the armor stone to the total weight of the armor stone section, in percent; i.e., area of section $abcda$ is 5 percent of area $ABCDEF$.  

$W_r = \text{weight of armor stone}$

$w_1 = W_r / 10$

$w_2 = W_r / 20$

$w_3 = W_r / 3$

Figure 7.7 Damage Parameter, D, for a Typical Model Test Section (Jackson, 1968a, pl. 21)
DESIGN EXAMPLE 7.2

SELECTION AND USE OF K₀ COEFFICIENTS

GIVEN: SMOOTH QUARRYSTONE BREAKWATER TRUNK SUBJECT TO NONBREAKING WAVES AND MINOR OVERTOPPING. LAYER THICKNESS n=2, NO-DAMAGE SIGNIFICANT DESIGN WAVE HEIGHT H₀=0 = 10.0 FT

REQU: a) For no-damage design, determine K₀. Determine the damage anticipated from 12 FT waves.

b) It is desired to allow 20 to 30 % damage. Determine H and K₀ to be used in this design.

SOLUTION:

a) Exceeding no-damage condition:

\[ K₀ = 2.4 \quad \text{(from Table 7.1 or 7.2)} \]

\[ H = 12 \text{ FT}, \quad \frac{H}{H₀=0} = \frac{12}{10} = 1.2 \]

From Table 7.2, for smooth quarystone, this value corresponds to 15 to 20 % damage.

If the structure is designed for no-damage H₀=0 = 10 FT and K₀ = 2.4, and is subsequently attacked by waves of H=12 FT, the cover layer damage anticipated is 15 to 20 %.
b) DAMAGE (20 - 30 %) DESIGN:

FROM TABLE 7.2, FOR D = 20 to 30 %, 

\[ \frac{H}{H_{D=0}} = 1.29 \Rightarrow H = 1.29 \times 10 \times 12.9 \text{ ft} \]

AND \( K_0 = 5.1 \)

IF THE STRUCTURE IS DESIGNED FOR \( H_{D=0} = 10 \text{ ft} \) AND \( K_0 = 2.4 \), WAVES OF \( H = 12.9 \text{ ft} \) COULD CAUSE COVER LAYER DAMAGE OF 20 TO 30 %.

ALTERNATIVELY, IF THE DESIGN USED \( H = 10 \text{ ft} \) AND \( K_0 = 5.1 \), 20 TO 30 % DAMAGE COULD BE CAUSED BY 10 FT WAVES.
weights, can be specified. A structure which will resist moderate storm wave action and suffer damage without complete destruction during a severe storm will have a lower total annual cost than one designed to be completely stable for larger waves (Hudson, 1974). The \( K_D \) values listed in Table 7.2 are used to evaluate alternative designs which allow cover layer damage. Selection of the optimum cover layer design for a rubble mound shore protection structure involves a tradeoff: it is desirable to permit a high damage percent to lower costs, but the damage must not be so large that it will significantly threaten overall stability or impede the functioning of the structure.

Hudson (1974) suggests that the use of slightly larger, less conservative \( K_D \) values may be partially justified by the nature of the mound itself. Settlement and readjustment of the cover layer generally result in increased interlocking of units and a structure more stable than the original. Design for damage also takes advantage of the fact that rubble mounds deform gradually as wave heights become progressively more severe. They tend to break down in a relatively "graceful" way in response to small percents of damage (Bruun, 1979). It is cautioned, however, that concrete armor unit layers may not behave in this rather benign fashion. Figure 7.8 documents typical damage development for dolos and quarystone slopes. The second stages of damage for dolosse may occur very rapidly once a hole in the armor layer has been created. Once damage is initiated the coherence of the structure is lost, and subsequent small increases in wave height will produce inordinate damage, necessitating considerable rebuilding rather than repair (Gravesen, Jensen and Sorensen, 1979). The effect of damage on
Figure 7.8 Damage Development for Quarystones and Dolosse for Oblique (70-80°) Incident Waves (Gravesen, Jensen and Sorensen, 1979, p. 16)
dolos-armored mounds in particular must be verified by hydraulic model tests.

**Conclusions.** In light of the limitations discussed, the stability coefficients in Table 7.1 incorporate little or no safety factor. Deviation to higher, less conservative $K_D$ values than those recommended for the no-damage criterion must be fully and critically evaluated (CERC, 1977). In cover layer design for the damage condition the chosen $K_D$ value depends ultimately on the margin of safety and degree of risk that the designer can afford to assume. The design of small-scale shore protection structures almost always provides for some degree of damage, for reasons of economy. As in all phases of coastal engineering design, experience and judgment are necessary in selecting the proper $K_D$ value in each case.

**Limitations of Stability Formulas**

During the past quarter of a century, most engineers and research workers have used the Hudson formula (Equation 7.2) for rubble mound cover layer design purposes. For the design of the simplest mounds, Hudson's formula has been used directly. For more important structures, where the results of hydraulic model tests provide the design basis, the empirical formula is still used in the interpretation and correction of test results (Mettam, 1980). It has become increasingly clear in recent years that there are definite limits to the applicability of Hudson's formula. Criticisms of the method and suggestions for new rubble mound design techniques are surveyed below.

**Contact Friction and Interlock.** Hudson's formula was developed to explain the behavior of natural rock units which owe their stability
under wave action principally to their own weight. The development of
congee armor units, which to varying degrees behave differently than
rock, has accentuated the limitations of Hudson's formula (Mettam,
1980). Dolosse and other fabricated concrete armor units derive their
stability largely from interlock with surrounding and underlying units.
This vital attribute is not considered in Hudson's equation. The
unsuitability of the empirical equation to describe dolos slope
stability was demonstrated by Brorsen, Burcharth and Larsen (1974). For
these units the stability coefficient, $K_D$, varies with structure slope
as indicated approximately by Figure 7.9. This has made it difficult to
interpret and compare the results of various model tests.

Contact friction between units is an important factor which has not
been adequately addressed. Mettam (1980) emphasizes the need to
represent contact friction accurately in model tests. New techniques
have been developed to form model units from materials other than the
previously used cement mortar. While these techniques are quicker,
cheaper and more reliable with respect to dimensional accuracy, they
often do not model the contact friction of the prototype concrete units.
The different contact friction changes the directions of forces acting
between the units in the model and alters the natural angle of repose
which, in turn, has a pronounced effect on mound slope stability.
Gravesen, Jensen and Sorensen (1979) feel that quantification of surface
friction is not very important in the prototype, but agree that it is of
particular significance in model testing and the extrapolation from
model to prototype values. Representative values of the natural angle
of repose for various armor units and some model units are presented in
Table 7.3.
Figure 7.9 Variation of the Stability Coefficient with Structure Slope for Dolosse (Bromsén, Burchart and Larsen, 1974, p. 1699)
Table 7.3  Representative Values of Angle of Repose for Various Armor Units and Model Units (Gravesen, Jensen and Sorensen, 1979, p. 9)

<table>
<thead>
<tr>
<th>Armor Unit</th>
<th>Angle of Repose $\mu = \tan \phi$</th>
<th>$\phi$</th>
<th>$\phi$ for Model Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quarrystones</td>
<td>1.1</td>
<td>48°</td>
<td></td>
</tr>
<tr>
<td>Cubes</td>
<td>1.2</td>
<td>50°</td>
<td></td>
</tr>
<tr>
<td>Tetrapods</td>
<td>$\approx 1.4$</td>
<td>$\approx 55°$</td>
<td>54° plastic 58° concrete</td>
</tr>
<tr>
<td>Dolos</td>
<td>$\approx 2.7$</td>
<td>$\approx 70°$</td>
<td>79° concrete 70° octangular plastic 68° round plastic</td>
</tr>
</tbody>
</table>
Unit Weight. Brantzaeg (1966) focused on the effect of the specific weights of armor unit material and fluid on stability. He suggests that when unit weights are either unusually large or small, Hudson's design formula (Equation 7.2) should be modified to include a variable term in the denominator:

$$W = \frac{H^3 \gamma_r}{K_D \left(\frac{\gamma_r}{\gamma_w} - \phi\right)^3 \cot \theta} \quad (7.3)$$

where $\phi$ is a variable quantity. Based on preliminary, inconclusive tests, $\phi$ ranged from 0.37 to 1.05.

The material unit weight occupies a prominent position in Hudson's equation. The unit weight of stone from a specific quarry will likely vary over a narrow band of values. The unit weight of concrete containing normal aggregates is usually between 140 and 155 pcf (22 to 24 kN/m$^3$). Concrete unit weight can be altered by including special heavy or light weight aggregates, which are usually more costly than typical aggregates. Designers should evaluate the feasibility of increasing the unit weight of armor units to lower overall structure costs (CERC, 1977). The effect of varying the unit weight, $\gamma_r$, on the required weight of armor units, $W$, can be evaluated with Figure 7.10. The weight factor of armor unit, $f$, on the abscissa of Figure 7.10 is the ratio of:

$$\left(\frac{\gamma_r}{\gamma_w} - 1\right)^3 \text{ to } \left(\frac{\gamma_a}{64} - 1\right)^3 \quad (7.4)$$

where $\gamma_a$ corresponds to the standard values in Figure 7.10:

- $\gamma_a$ concrete = 150 pcf
- $\gamma_a$ quarrystone = 165 pcf
Figure 7.10 Effect of Unit Weight Changes on Required Armor Unit Weight (CERC, 1977, p. 7-192)
The use of Figure 7.10 is illustrated by Design Example 7.3.

**Angle of Wave Incidence.** There are insufficient data to quantify the effect of angle of wave approach on armor unit stability. Quarroystone slopes are assumed to be more stable to oblique wave attack since the wave heights are reduced by refraction. Limited test results by Whillock and Price (1976) indicate that a corresponding improvement in stability might not occur with blocks that are susceptible to drag forces. The stability of dolos units on a 1 on 2 slope decreased from normal wave incidence to an angle of 60 degrees and then improved rapidly (Figure 7.11). It was theorized that when waves break at an angle, surging flow over the dolos surface, coupled with high velocities directed up the slope, cause high drag. This in effect "fluidizes" the dolos layers and the benefits of interlocking and contact friction disappear. Above 60 degrees, the advantages of refraction and wave height reduction are reasserted and stability improves.

The response of dolos slopes to small percentages of damage is described in the previous section. Many concrete block layers degrade quickly once damage is initiated. Figure 7.8 illustrates typical damage development for dolosse under oblique incident waves. As indicated, there may be a fine line between wave attack which will produce acceptable damage and that which causes failure. Whillock and Price (1976) recommend that dolos mounds subject to oblique attack be designed only for the no-damage wave height at normal incidence. The stability of these structures should be verified by model tests.

**Wave Period.** The crucial role of wave frequency in rubble mound design has been explored by Bruun and Gunbak (1976) and Bruun and
DESIGN EXAMPLE 7.3

EFFECT OF VARYING ARMOR UNIT SPECIFIC WEIGHT

**Given:** Rough quarystone armor layer, \( y_r = 150 \text{ pcf} \)

**Required armor unit weight, \( W = 12 \text{ tons} \)

**Req'd:** For the same wave action, determine the quarystone weight if

a) \( y_r = 140 \text{ pcf} \)

b) \( y_r = 175 \text{ pcf} \)

**Solution:**

Obtain the weight factors from the upper curve in Figure 7.10:

\[
\begin{align*}
  f (y_r = 140 \text{ pcf}) & = 2.03 \\
  f (y_r = 150 \text{ pcf}) & = 1.49 \\
  f (y_r = 175 \text{ pcf}) & = 0.78
\end{align*}
\]

a) \( y_r = 140 \text{ pcf} \):

\[
W_{140} = W_{150} \times \frac{f_{140}}{f_{150}} = 12 \times \frac{2.03}{1.49} = 16.3 \approx 16 \text{ tons}
\]

For \( y_r = 140 \text{ pcf}, W = 16 \text{ tons} \)

b) \( y_r = 175 \text{ pcf} \):

\[
W_{175} = 12 \times \frac{0.78}{1.49} = 6.3 \approx 6 \text{ tons}
\]

For \( y_r = 175 \text{ pcf}, W = 6 \text{ tons} \)

*For the given stone, an increase in unit weight of \( -17\% \) will enable a \( 100\% \) decrease in the rock weight required. Stone of the lower unit weight would be \( 33\% \) heavier to provide the same protection.*
Figure 7.11 Effect of Angle of Wave Approach on Dolos Slope Stability (Whillock and Price, 1976, p. 2571)
Johannesson (1976). According to the researchers, it is not logical to ignore the various flow characteristics which occur on the mound by assuming a constant stability coefficient for the entire range of wave periods, as in Hudson's formula. A "surf similarity parameter", $\xi$, was proposed to describe flow characteristics:

$$\xi = \frac{tg \theta}{\sqrt{H/L_0}}$$  \hspace{1cm} (7.5)

in which: $t =$ wave period, $g =$ gravitational acceleration, $\theta =$ mound slope angle with the horizontal, $H =$ wave height in front of structure, and $L_0 =$ deepwater wavelength. The maximum destructive forces on rubble mounds were observed at the "resonance" state, when deep rundown occurs simultaneously with collapsing-plunging wave breaking at a given location. Impact and uplift forces on the armor units seem to maximize around resonance, accompanied by large-scale turbulence. This crucial condition corresponds to $2.0 < \xi < 3.0$. Buildup of hydrostatic pressure within the core due to wave uprush increased with decreasing core permeability and with increasing $\xi$ values for $\xi < 4.0$. Wave runup and rundown increase progressively and reach a constant value at approximately $\xi > 5.0$. It is believed that the $\xi$ parameter will be useful in developing a more reliable, better reasoned design procedure.

Other Factors. Additional factors that are known to affect rubble mound stability, but are not adequately covered in Hudson's stability formula, include:

1. Wavelength variations
2. Duration of the storm
3. Randomness of incident waves
4. Degree of overtopping
5. Variations in the water depth

6. Other external loads, as from winds, currents and ice (See Chapter 5)

Conclusions and Future Trends

It has been demonstrated that stability formulas, although presently used exclusively in rubble mound design, are not wholly satisfactory. A proposed force balance method (Wang, 1977, after Bruun and Johannesson, 1974), while neither widely accepted nor extensively tested, seems promising as it is based on more rational analysis and can be extended to include loadings other than wave forces. The procedure evaluates armor stability by considering the simple force balance on individual armor units. If the resultant uprush or downrush forces exceed the interlocking and frictional forces between units, the layer becomes unstable. Similarly, the uplift force must not be greater than the opposing net weight of the unit. The problems with applying this method result from the lack of experience and current difficulties in estimating and quantifying the individual force components and the interlocking and frictional forces of armor units.

Hudson's stability formula, and other similar equations, represent the state-of-the-art in rubble mound cover layer design. These empirical methods are tried and trusted, supported by an extensive tabulation of $K_D$ values from model tests and a broad prototype data base. For the design of simple groins and other small-scale structures, Hudson's formula is and will continue to be a very convenient design tool. However, in a single formula it oversimplifies the complex behavior of armor units in the cover layer. As rubble mound protective
structures are built in deeper waters and more severe environments, the need to reduce reliance on existing formulas becomes increasingly urgent. Research is currently progressing toward the development of new analytical techniques for rubble mound design. Although it is unlikely that theoretical methods will totally replace model testing, they should constitute a major advance in the understanding of rubble mound behavior.

At present, Hudson's formula serves well to give a preliminary determination of armor unit weights. For small-scale structures, this initial design modified on the basis of engineering judgment and experience may be sufficient for implementation. Final design of larger-scale rubble mounds is usually based, to some degree, on the results of hydraulic model tests. Effects described in the preceding section should be accounted for in laboratory simulations. Until new techniques have been developed, tested and proven, it is imperative to recognize the limitations of the current design methods and be very careful not to use them out of their intended context (Mettam, 1980).

7.2 CROSS-SECTION DESIGN

The typical rubble mound cross-sections shown in Figures 7.12 and 7.13 are those recommended by the U.S. Army Corps of Engineers for nonbreaking and breaking wave conditions, respectively (CERC, 1977). Most rubble mound breakwater cross-sections resemble these standard designs, although changes might be made depending on actual site conditions. Jetty and groin sections are usually similar, but somewhat less complex. Design guidelines for the basic features of the cross-sections are presented below, specifically:
Figure 7.12  Rubble Mound Cross-Section for Nonbreaking Wave Condition with No- to Moderate Overtopping (CERC, 1977, p. 7-203)
Figure 7.13 Rubble Mound Cross-Section for Breaking Wave Condition with Moderate Overtopping (CERC, 1977, p. 7-204)
1. Crest elevation and width
2. Primary cover layer
3. Secondary cover layer
4. Underlayers
5. Layer thickness and number of armor units
6. Core
7. Foundation bedding layer (See Chapter 5)

In Figures 7.12 and 7.13, the average rock size for each layer is expressed as a fraction of the cover layer armor unit weight, $W$. Each layer size gradation is given as a percentage of the average rock size. To prevent smaller rocks in an underlayer from being pulled through the adjacent overlayer by wave action, the rock size gradations may be checked by the filter criteria detailed in Chapter 5, particularly:

$$5d_{85} \text{ (underlayer)} > d_{15} \text{ (overlayer)} \quad (7.6)$$

where $d_{15}$ and $d_{85}$ are the particle sizes on a grain size distribution plot at 15 and 85 percent, respectively, finer by weight.

Alterations to the standard rubble mound profile are advocated by Bruun and Johannessen (1976). They propose optimization of the mound slope by designing for the wave action which occurs on each section. Since slopes should be gentlest where destructive forces are greatest, they suggest the slope be modified with a flatter section near the still water level. The slope below SWL should be relatively steep, to make the backwash-incipient breaker interaction less violent. This design is best for areas with a limited tidal range. The proposed S-shaped slope resembles that which is often naturally developed when rubble mounds readjust and settle under wave action, as shown in Figure 7.14.
Figure 7.14 S-Shape Profile of "Mature" Breakwaters at Plymouth, England and Cherbourg, France
(Bruun and Kjelstrup, 1981, p. 172)
Crest Elevation and Width

Overtopping of mound crests can usually be tolerated if it does not generate detrimental waves in the lee of the structure. The crest elevation relative to the design SWL and height of wave runup determines the extent of wave overtopping which will occur. Selection of crest elevation for breakwaters, groins and jetties is discussed in Chapter 3.

Crest width depends largely on the allowable overtopping. Where only minor overtopping is permitted the crest width is not critical. For overtopping conditions, CERC (1977) recommends a minimum width equal to the combined widths of three armor units (n=3). Crest width may be computed as:

\[
B = n k_A \left( \frac{W}{\gamma_r} \right)^{1/3}
\]  

(7.7)

in which: \( B \) = crest width, \( n \) = number of stones (n=3 minimum), \( k_A \) = layer coefficient, \( W \) = cover layer armor unit weight, and \( \gamma_r \) = unit weight of armor unit.

Primary Cover Layer

The exterior or primary cover layer armor unit weight, \( W \), is calculated from Equation 7.2, according to the principles discussed in Section 7.1. CERC (1977) recommends a two unit thickness (n=2) on the cover layer.

The primary layer coverages recommended for various combinations of water depth and overtopping are summarized in Table 7.4. Required extension down the seaward slope is based mainly on the water depth at the structure relative to the wave height, and on the type of wave which acts on the face. As shown in Figure 7.12, armor unit weight can be
Table 7.4 Minimum Downslope Extension of the Primary Cover Layer for Various Conditions (after ASCE, 1969; CERC, 1977)

<table>
<thead>
<tr>
<th>Water Depth at Structure ≤ 1.3H (Figure 6.13)</th>
<th>Water Depth at Structure &gt; 1.3H (Figure 6.12)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overtopping</td>
<td>Nonovertopping</td>
</tr>
<tr>
<td>Seaward slope, Extension</td>
<td>To Bottom</td>
</tr>
<tr>
<td>Leeward slope, Extension *</td>
<td>To Bottom</td>
</tr>
</tbody>
</table>

* Note: When both slopes receive similar wave action, as on groins and some jetties, both sea and leeward slopes should be of similar design and cover layer extension.
reduced in water deeper than about 1.5H (H is the design wave height) below SWL, because the wave forces acting on the slope at depth are smaller than those nearer the surface. Design of the lee side cover layer depends on the extent of wave overtopping, wave forces which may act directly on the lee slope, porosity of the structure, and differential hydrostatic head resulting in uplift forces which may dislodge the back slope armor units (CERC, 1977). For overtopping structures, back slope stability is an important concern. According to Magoon, Sloan and Foote (1974), one of the most common maintenance efforts on rubble mound structures is necessitated by the loss of material from the leeward slope because of overtopping. On these structures, protective units on the back slope must be as large and well placed as those on the seaward slope. Dunham and Finn (1974) warn that this design feature must be emphasized in the project reports to avoid future misunderstandings or recriminations if minor damage should occur.

Critical toe instability may occur at the intersection of the cover layer with the sediment bed or bedding layer. Whenever economically feasible, model studies should be made (CERC, 1977). Instability may also be initiated at the leeward toe of an overtopping structure. Overspill and waves breaking directly on the back slope can cause significant leeward trenching. The section in Chapter 5 on scour and scour control should be consulted.

Under similar wave conditions, the rubble mound head may be expected to sustain more extensive and frequent damage than the trunk. The head is usually subject to overtopping and wave attack from all directions. CERC (1977) recommends that the head armor be the same
on both the seaward and lee slopes for a distance of about 50 to 150 ft (15 to 46 m) from the structure end. The exact distance depends on structure length and crest elevation at the seaward end.

**Secondary Cover Layer**

When the structure is located in shallow water \((d < 1.3H)\) the primary cover layer covers the entire seaward slope (Figure 7.13). In deeper water a secondary cover layer of lighter units can be placed downslope of the first layer. Referring to Figure 7.12, the average weight of armor units in the secondary layer should about equal one-half of the primary armor unit weight \((W/2)\) between \(-H\) and \(-1.5H\), and be reduced to \(W/15\) below \(-1.5H\), assuming a constant slope. These ratios are valid for both quarrrystone and concrete armor units (CERC, 1977).

When the size of cover layer stone is reduced below \(-H\), the number of layers, \(n\), should be increased to maintain a thickness at least equal to that of the primary cover layer to prevent it from sliding. A sample calculation of secondary layer thickness is performed in Design Example 7.5. Often, the primary layer elements are concrete and the secondary layer is composed of stone. It is important that the weight of units in the secondary stone layer be based on the equivalent weight of stone required for stability in the primary layer, \(W_{eq}\), rather than on the actual weight of the concrete units, \(W\) (ASCE, 1969). This principle is used in Design Example 7.4.

**Underlayers**

It is customary to use quarrrystone for the underlayer system beneath the cover layer(s). These should be large enough to prevent their withdrawal through voids in the adjacent upper layers. Unless the
DESIGN EXAMPLE 7.4

DESIGN OF SECONDARY COVER LAYER

GIVEN:
- DESIGN NONBREAKING WAVE HEIGHT, \( H = 15 \) FT
- \( \cot \theta = 2.0 \) (1 ON 2 SLOPE)
- SEA WATER \( \gamma_\text{w} = 64.0 \) PCF
- PRIMARY ARMOR UNITS - QUADRIPODS, \( \gamma_\text{c} = 140 \) PCF
- SECONDARY ARMOR UNITS - SMOOTH QUARRYSTONE, \( \gamma_\text{q} = 165 \) PCF

REQUIRED:
- a) WEIGHT OF PRIMARY LAYER UNITS, \( W \)
- b) WEIGHTS OF SECONDARY UNITS, \( \frac{W}{2} \) AND \( \frac{W}{15} \)

SOLUTION:

a) USING HUDSON'S FORMULA, EQN 7.2,

\[
W = \frac{\gamma_c H^3}{K_0 (5 - 1)^3 \cot \theta}
\]

From Table 7.1, \( K_0 = 8.3 \)

\[
= \frac{11065^3}{8.3 (140 - 1)^3 2.0} = 5.53 \text{ TONS}
\]

\( W \) QUADRIPODS \( \approx 5.5 \) TONS

b) SECONDARY LAYER WEIGHTS MUST NOT BE BASED ON THE REQUIRED QUADRIPOD WEIGHT, AS THE LOW CONCRETE ARMOR UNIT WEIGHT REFLECTS THE INTERLOCKING AND ENHANCED STABILITY OF THOSE UNITS. INSTEAD, A WEIGHTEQUAL FOR QUARRYSTONE MUST BE COMPUTED.

From Table 7.1, \( K_0 = 2.4 \)

\[
W_{\text{equiv}} = \frac{165 \times 19^3}{2.4 \times \left(\frac{165}{64} - 1\right)^3 2.0} = 9.61 \text{ TONS} \approx 9.5 \text{ TONS}
\]

THEN, \( \frac{W}{2} = \frac{9.5}{2} = 4.8 \text{ TONS} \)

\( \frac{W}{15} = \frac{9.5}{15} = 0.63 \text{ TONS} \approx 1270^* \)

QUARRYSTONE: \( \frac{W}{2} = 4.8 \text{ TONS} \) ; \( \frac{W}{15} = 1270^* \)
cover stone is relatively small, two or more underlayers will be required for proper filter action (See Chapter 5). Each underlayer should have a minimum equivalent thickness of two quarystones (n=2). The following additional recommendations are shown graphically in Figures 7.12 and 7.13. For this discussion, the weight of the first underlayer stones is referred to as \( W_1 \), the second underlayer stone weight is \( W_2 \), etc.

**First Underlayer.** Based on filter criteria, the weight of stones in the first underlayer, \( W_1 \), can be about \( W/20 \). The common practice is to use \( W/10 \) for the first underlayer to provide larger voids for better nesting of primary armor units and to reduce back pressure on the cover layer (Hudson, 1974). The results of recent tests reported by Carver (1980) indicate that variations in \( W_1 \) from \( W/5 \) to \( W/20 \) do not have a significant effect on armor stability or wave runup or rundown. The \( W/10 \) criterion applies where the armor elements are quarystone or concrete units with a stability coefficient \( K_D \leq 12 \). Again, the weight of stones under concrete units must be specified based on the equivalent weight of stone required for stability in the cover layer, \( W_{eq} \), rather than on the actual concrete armor unit weight, \( W \) (See Design Example 7.4).

As the stability coefficient increases for concrete armor units, the required armor size, \( W \), decreases (See Equation 7.2). Underlayer stone sized according to \( W_{eq}/10 \) will become proportionately too large compared with the smaller armor unit size. Hudson (1974) and CERC (1977) recommend that the first underlayer stone beneath dolosse and armor units with \( K_D > 12 \) be specified as \( W_1 = W/5 \) (note that \( W \) is the actual weight of the concrete units and not \( W_{eq} \)).
For the nonbreaking wave cross-section, Figure 7.12, the first underlayer below \(-1.5H\) should weigh about \(1/20\) of the overlying secondary armor units \((W_1 = 1/20 \times W/15 = W/300)\).

**Secondary Underlayer.** Stone in the second and successive underlayers should weigh \(1/20\) the weight of the adjacent overlying layer. That is, \(W_2\) will weigh \(W_1/20\) \((1/20 \times W/10 = W/200)\), \(W_3\) equals \(W_2/20\), etc.

**Gradation.** The underlayer material can be graded to some extent. The stone in the first underlayer should be graded the least, and succeeding layers can be composed of progressively wider ranges of stone sizes (Hudson, 1974). Suggested gradations are given in Figures 7.12 and 7.13.

**Layer Thickness and Number of Armor Units**

The thickness of cover and underlayers and the number of armor units required per unit area can be calculated from the following formulas when values of the experimentally determined coefficients are available:

\[
r = nk_\Delta \left(\frac{W}{\gamma_r}\right)^{1/3} \tag{7.8}
\]

\[
\frac{N_r}{A} = nk_\Delta \left(1 - \frac{P}{100}\right) \left(\frac{\gamma_r}{W}\right)^{2/3} \tag{7.9}
\]

in which: \(r\) = the average thickness of \(n\) layers of armor units of weight, \(W\), and specific weight, \(\gamma_r\); \(N_r\) = the required number of armor units for a given surface area, \(A\); \(k_\Delta\) = layer coefficient; \(P\) = average porosity of cover layer. The magnitudes of \(k_\Delta\) and \(P\) vary with the shape
and manner of placement of armor units. Table 7.5 lists available values, from the results of small-scale experiments (Hudson, 1974).

The average dimensions for a range of quarrystone weights, based on $Y_r = 165$ pcf, are given in Table 7.6; that is, Equation 7.8 is solved for $n=1$. Design Example 7.5 demonstrates a use of Equation 7.8.

The designer must calculate the total number of armor units needed for a rubble mound section to ensure that an adequate number are used to meet stability requirements and to estimate the total armor unit cost. Carver and Davidson (1977) conducted tests on dolos-armored slopes to study the effect on rubble mound stability of a decreased number of armor units in the cover layer. The data showed that decreasing the number of armor units by 25 percent reduced the stability coefficient by as much as 50 percent. These results illustrate the critical role of the number of units in maintaining mound stability.

Core

The core stone can be as light as $W/6000$. Quarry run is the most frequently used rubble mound core material, and gravel, sand and clay have all been used successfully in the core of rubble structures. The underlayer adjacent to the core should be graded such that piping and loss of fine core material is avoided.

The height and permeability of the core can affect mound stability. High, impervious cores tend to increase wave reflection and build up hydrostatic head, which may generate uplift forces beneath the cover (ASCE, 1969).

Foundation Bedding Layer

The need for filter blankets, and their design and construction,
<table>
<thead>
<tr>
<th>Armor Unit</th>
<th>n</th>
<th>Placement</th>
<th>Layer Coefficient $k_A$</th>
<th>Porosity (P) percent</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quarrystone (smooth)</td>
<td>2</td>
<td>random</td>
<td>1.02</td>
<td>38</td>
</tr>
<tr>
<td>Quarrystone (rough)</td>
<td>2</td>
<td>random</td>
<td>1.15</td>
<td>37</td>
</tr>
<tr>
<td>Quarrystone (rough)</td>
<td>&gt;3</td>
<td>random</td>
<td>1.10</td>
<td>40</td>
</tr>
<tr>
<td>Cube (modified)</td>
<td>2</td>
<td>random</td>
<td>1.10</td>
<td>47</td>
</tr>
<tr>
<td>Tetrapod</td>
<td>2</td>
<td>random</td>
<td>1.15</td>
<td>47</td>
</tr>
<tr>
<td>Quadripod</td>
<td>2</td>
<td>random</td>
<td>0.95</td>
<td>49</td>
</tr>
<tr>
<td>Hexapod</td>
<td>2</td>
<td>random</td>
<td>1.04</td>
<td>50</td>
</tr>
<tr>
<td>Tribar</td>
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<td>random</td>
<td>1.02</td>
<td>54</td>
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<td>Dolos</td>
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<td>1.00</td>
<td>63</td>
</tr>
<tr>
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<td>1</td>
<td>uniform</td>
<td>1.13</td>
<td>47</td>
</tr>
<tr>
<td>Quarrystone</td>
<td></td>
<td>graded</td>
<td>random</td>
<td>37</td>
</tr>
</tbody>
</table>

Table 7.5 Layer Coefficient and Porosity for Various Armor Units (CERC, 1977, p. 7-208)

<table>
<thead>
<tr>
<th>Weight (tons)</th>
<th>Dimen. (ft.)</th>
<th>Weight (lbs.)</th>
<th>Dimen. (ft.)</th>
<th>Weight (lbs.)</th>
<th>Dimen. (in.)</th>
<th>Weight (lbs.)</th>
<th>Dimen. (in.)</th>
<th>Weight (lbs.)</th>
<th>Dimen. (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.64</td>
<td>100</td>
<td>0.97</td>
<td>5</td>
<td>4.30</td>
<td>0.5</td>
<td>2.00</td>
<td>0.025</td>
<td>0.74</td>
</tr>
<tr>
<td>2</td>
<td>3.33</td>
<td>200</td>
<td>1.23</td>
<td>10</td>
<td>5.42</td>
<td>1.0</td>
<td>2.52</td>
<td>0.050</td>
<td>0.93</td>
</tr>
<tr>
<td>3</td>
<td>3.81</td>
<td>300</td>
<td>1.40</td>
<td>15</td>
<td>6.21</td>
<td>1.5</td>
<td>2.88</td>
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Table 7.6 Weight and Average Size of Quarrystones (CERC, 1977, p. 7-210)
**DESIGN EXAMPLE 7.5**

**DESIGN OF SECONDARY LAYER THICKNESS**

**GIVEN:** PRIMARY COVER LAYER OF MATERIAL X, ARMOR UNIT WEIGHT \( \gamma_p \), NUMBER OF LAYERS \( n_p = 2 \).

SECONDARY COVER LAYER OF SAME MATERIAL. BETWEEN -1.5H AND -1.5H, ARMOR WEIGHT \( \gamma_s = \frac{\gamma_p}{2} \) BELOW -1.5H, REQUIRED WEIGHT \( \gamma_s = \frac{\gamma_p}{15} \), AS SHOWN IN FIGURE 7.12.

**REQUIRED:** SECONDARY COVER LAYER THICKNESS

**SOLUTION:**

USE EQN 7.8. RECALL THAT THE SECONDARY COVER LAYER THICKNESS MUST AT LEAST EQUAL THAT OF THE PRIMARY LAYER TO PREVENT EN MASSE SLIDING OF THE HEAVIER UNITS.

\[
\tau = n_k_s \left( \frac{\gamma_p}{\gamma_s} \right)^{1/3}
\]

**HERE,** \( \gamma_{\text{secondary}} \approx \gamma_{\text{primary}} \). \**SET** \( \gamma_s = \gamma_p \)

\[
n_s \frac{k_{bs}}{\gamma_s} \left( \frac{\gamma_s}{\gamma_s} \right)^{1/3} = n_p \frac{k_{ap}}{\gamma_p} \left( \frac{\gamma_p}{\gamma_p} \right)^{1/3}
\]

**BECAUSE THE MATERIALS ARE THE SAME,** \( k_{bs} = k_{ap} \) \**AND** \( \gamma_s = \gamma_p \), \**AND THESE TERMS CANCEL OUT:**

\[
n_s \left( \frac{\gamma_p}{\gamma_p} \right)^{1/3} = n_p \left( \frac{\gamma_p}{\gamma_p} \right)^{1/3}
\]

**AS \( \gamma_s \) DECREASES, \( n_s \) MUST INCREASE. SOLVING FOR \( n_s \):**

\[
n_s = n_p \left( \frac{\gamma_p \gamma_s^{-1/3}}{\gamma_s \gamma_p^{-1/3}} \right), \quad n_p = 2
\]
THEN, FOR \( W_3 = \frac{W_p}{2} \),

\[
N_3 = 2 \frac{W^{3/3}}{(\frac{W}{2})^{3/3}} = \frac{2}{(\frac{1}{2})^{3/3}} \approx 2.52 \approx 2.5
\]

**BETWEEN \( -H \) AND \( -1.5H \), \( N_3 = 2.5 \)**

FOR \( W_3 = \frac{W_p}{15} \),

\[
N_3 = \frac{2}{(\frac{1}{15})^{3/3}} = 4.93 \approx 5.0
\]

**BELOW \( -1.5H \), \( N_3 = 5 \)**
are investigated at some length in Chapter 5. Rubble mound foundation design is an important topic which warranted a separate discussion; therefore, the appropriate section should be studied carefully.

7.3 CONSTRUCTION PLANNING

Theory and practice are of equal importance in rubble mound design. The construction scheme must be developed at the same time as the structure cross-section, as indicated by Figure 7.1.

Inexperienced designers may fail to recognize the inherent difficulties of certain operations and are likely to establish unrealistic requirements in the specifications (Peck, 1973). Some cross-section details, which do little to enhance stability, can be difficult and unnecessarily expensive to build. Profiles that are too complex may be impossible to construct. In these cases, the contractor might depend more on his experience than the construction plans, and proceed according to standard practice. This deviation will generate problems if his changes alter the stability of the mound. Therefore, the designer must clearly understand the construction techniques and site conditions under which the work will be carried out, and design within their limits. Close cooperation between the designers and builders will curb problems in translating designs into a completed structure (Bruun and Kjelstrup, 1981).

Construction Techniques and Equipment

Rubble mound construction equipment can be considered in two categories: 1) land-based and 2) offshore-based (floating) equipment.
This delineation depends largely on the location of the proposed construction. When the mound extends offshore beyond the limited reach of land-based machinery, floating equipment will be necessary. Many projects combine these two types, as exemplified by the operations discussed and illustrated below.

Floating rigs are often used for placement of the core and smaller underlayer rubble, especially when material supplies are brought in by hopper barges. Floating cranes can be used to place toe protection and larger cover layer units. It is well known that the risk of damage to floating equipment is higher, and the progress of construction can be impeded by poor weather and surf conditions (Sanko and Smith, in preparation). Therefore, more down time is expected with the use of a floating plant.

Placement of armor blocks without damaging them is essential. Wave-induced motions of floating equipment can result in breakage of armor units as they are being positioned. Construction breakage of concrete blocks with slim geometry placed near the mean sea level is most critical and can result in a severely weakened mound which may eventually fail (Bruun and Kjelstrup, 1981).

Shore-connected rubble structures, including groins, jetties and some breakwaters, are usually built by modern variations of the truck-haul technique described by Kidby, Powell and Roberts (1964). The appropriate materials are dumped at the advancing end of the mound and then, typically, pushed over the crest with a dozer. The crest of the built-up core thus serves as a working platform for subsequent armor unit placement. There are some disadvantages associated with the use of this method (Quinn, 1972):
1. The crest width needed for maneuvering machinery may be greater than that otherwise dictated by cross-section design.

2. The upper surface of the core will become clogged with fines and compacted from the travel of machinery. The top surface will have to be removed, or the fines washed out, prior to armor unit placement. Alternatively, a coarse filter several feet thick can be placed over the roadway; this may increase the mound height, and the corresponding overall volume of material, considerably.

3. Unless armoring commences quickly, the exposed core material may be washed away by storm wave action.

Despite these factors, this method is most economical, especially for the construction of smaller shore-connected mounds.

When rubble is dumped over the crest, the larger stones tend to roll to the bottom of the slope (Figure 7.15). Kjelstrup (1979) discusses a satisfactory solution recently achieved in Norway. The "back-hoe method" uses excavators of about 50 tons (445 kN) with back-digging equipment to scrape stopes up the slope and smooth out the core material and underlayers and to construct the layer of cover stones. Figure 7.16 illustrates these operations. With improvements, it is anticipated that this scheme could handle armor units of up to 20 tons (178 kN).

The top of the core cannot be used as a working base if it does not extend above water level, as in Figure 7.17. There, the core is placed as dredged material or dumped from scows. This configuration enables use of more of the fine quarry waste material and permits use of other media, such as sand, coral and dredgings (Quinn, 1972).

Another land-based construction technique entails building a wood pile trestle from which the crane and other equipment can operate. The construction of Coco Solo breakwater, Panama Canal Zone, was accomplished from a trestle built over the cover layer of the proposed
Figure 7.15 Separation of Large Rubble
(Kjelstrup, 1979, p. 139)

Figure 7.16 Back-Hoe Method to Finish Rubble
Mound Layers (Kjelstrup, 1979, p. 140)
Figure 7.17  Design Cross-Section, Submerged Core Rubble Mound Breakwater (Quinn, 1972, p. 176)
shore-connected mound (Figure 7.18). A temporary trestle might also be built out from the shore to an offshore site for the transport of land-based equipment (Dunham and Finn, 1974). Because this procedure is quite costly, it is feasible only for larger-scale structures.

Theoretical cross-section design and practical considerations of construction equipment and methods must evolve concurrently. It may be possible to alter the design to one easier and cheaper to build. For example, if the rock mound slopes are relatively flat and the water is deep, the reach from the core crest may be excessive and marine equipment will be needed to place overlayer materials. Similarly, the reach required of a crane operating from the core will become excessive to handle heavier armor units. In this case, it may prove economically desirable to use steeper slopes and supplement the cover layer rock with concrete units if necessary (Quinn, 1972).

The availability of equipment is another influencing factor. For example, if the only crane economically available has a capacity of 10 tons, the designer should propose a cross-section comprising armor units weighing only 10 tons or less. The fact that the quarry might have been able to produce sufficient quantities of heavier rock is not at issue here; the limiting factor is equipment availability. Even though this plan, then, does not reflect optimal material usage, the design is justified if it lowers overall project costs.

The lifting capacity and dimensions of the selected mechanical equipment must be sufficient to build the final rubble mound cross-section. The range in equipment characteristics is wide and constantly developing. Current and detailed information can be obtained from manufacturer’s literature.
Figure 7.18  Typical Cross-Section of Trestle and Traveler used for Construction of Coco Solo Breakwater (Quinn, 1972, p. 204)
Sequence of Operations

Regardless of the technique of rubble mound construction, the core and smaller stones must be placed before the cover layers can be laid. Consideration must be given to holding the core slopes stable against wave action until they can be armored. The construction should be carried out in stages to assure that the last completed portion is at least temporarily resistant to premature failure prior to completion of the entire structure (Sanko and Smith, in preparation).

The typical construction procedure followed at the Sines breakwater, Portugal, is shown in Figure 7.19. Most of the rubble core was dumped by 1000 ton (8896 kN) hopper barges and the remainder tipped from trucks on the core. The underlayer stone and dolosse were placed by floating cranes and a crawler-mounted crane. To restrict the amount of work at risk during storms, core placement above −20 CD (chart datum) could not proceed more than 50 m (164 ft) ahead of the secondary armor, which itself was not allowed more than 50 m (164 ft) ahead of the dolosse (Mettam, 1976).

When the construction is entirely with land-based equipment, the laying of armor proceeds in a "reverse" direction; that is, the core is placed from the shore seaward, and the application of armor units begins at the seaward end. This staged procedure, shown in its simplest form in Figure 7.20, aids progress by reducing congestion of haul traffic over the breakwater crown.

The mound can be built up gradually, in horizontal layers or lifts, rather than in full height sections. The breakwater at Rotterdam, Europort was constructed in six phases with floating equipment. Illustrated schematically in Figure 7.21, these were: 1) dredging,
Figure 7.19  Staged Construction, Sines Breakwater, Portugal (Zwamborn, 1979, p. 428)
Figure 7.20 Two-Stage Construction of a Shore-Connected Rubble Mound Structure
(Dunham and Finn, 1974, p. 57)
Figure 7.21 Lift Construction of Breakwater at Rotterdam, Europort.
See Text for Descriptions (TAMU, 1971, p. 71)
2) laying small gravel, 3) laying larger gravel and rubble, 4) laying rubble of 1 to 6 tons, 5) laying 43 ton concrete blocks, and 6) final filling with rubble.

Ideally, the rubble mound will be permitted to settle and adjust under its own weight and wave action for one or two years before the permanent construction is completed. The breakwater at Gryllefjord, Troms, Norway was erected in this manner. Figure 7.22a shows the breakwater after the first construction season, built to a height of 6.0 m (19.7 ft). After one or two years (Figure 7.22b) cover layer blocks were removed from the upper seaward slope and some of the core moved to the leeward slope, resulting in a flatter outer profile. This was accomplished in one operation with a medium sized backhoe (See Figure 7.16). After two more years, the structure had settled to the design crest elevation of 5.5 m (18.0 ft) and the construction was finalized with a concrete cover and armor blocks (Bruun and Kjelstrup, 1981). This technique increases the stability of the final product. However, some structures are too exposed to be left in an unfinished form. Similarly, many projects, as jetties and groins, must be complete to fulfill design objectives.

The length of the construction season is of major importance in the planning of operations. Local variability in surf and tide conditions similarly affects sequencing. It is usually, although not always, cheapest and easiest to work during the most climatically favorable period of the year. Hasty work should be avoided but may sometimes become imperative. The need for rapid execution may at some sites be so pressing that the stability of the rubble mound will have to be compromised to some extent. Major modifications in the design,
Figure 7.22 Breakwater at Gryllefjord, Troms
after a) Initial construction,
b) One or two years, c) Four years.
necessitated by the practical aspects of construction, must be discussed and agreed upon by the designers and builders before the work commences (Bruun and Kjelstrup, 1981).

Scour at the working end can be the source of construction difficulties. The sequence of operations can be modified to control such problems when they are anticipated. Techniques of construction erosion control recommended by Hale (1980) are presented in Chapter 5.

When more than one rubble mound structure is being installed, the necessity for organized and efficient sequencing of operations is apparent. The complexity of arrangements which must be planned is demonstrated by Figure 7.23, the network of structures of a sizeable casting yard for the manufacture of concrete breakwater armor units. Harbor or marine protection devices are usually placed first, for ease of inner harbor construction. Jetties are often built before channel dredging for dredge protection and drift exclusion purposes (Dunham and Finn, 1974). The groin at the downdrift end of a series should be constructed first, to reduce downdrift damage. The order of groin construction is discussed in Chapter 3.

**Quality Control**

Supervision of construction must be strict. Close and continuous inspection of all phases of the work will be required. Good supervision requires a full understanding by the field personnel of all aspects of design and construction, so that they can make technically sound decisions for modifications as the work progresses. It is, however, often the less experienced personnel who are sent out to the sites. In such cases good rapport between the designer and the construction
Figure 7.23  Casting Yard for Coco Solo Breakwater Concrete Armor Units. 1) Cement storage shed, 2) Block casting platform, 3) Aggregate hopper, 4) Settling tank, 5) Storehouse, 6) Saw shed, 7) Carpentry shed, 8) Concrete building, 9) Whirley crane, 10) Water tank, 11) Oil storage, 12) Concrete building, 13) Office and quarters, 14) Concrete plant, 15) Screening plant, 16) Aggregate conveyor, 17) Sand stock pile, 18) Gravel stock pile, 19) Shed, 20) Whirley crane (Quinn, 1972, p. 207)
supervisor is essential (Bruun and Kjelstrup, 1981). The combination of a neophyte inspector and an experienced contractor may be detrimental to the quality and progress of the job (Peck, 1973).

Submerged sections of construction are difficult to control and therefore require frequent inspections. At shallow depths a plumb line and water telescope can be used for the survey. At greater depths the periodic use of divers will be necessary. A common problem is that the core is placed with side slopes too steep for stability. If the superstructure blocks cannot be situated underwater with a crane with a long enough boom, the submerged blocks may have to be moved to the prescribed slope by down-blasting with small charges, as shown in Figure 7.24 (Bruun and Kjelstrup, 1981).

In addition to visual inspection, the progress of the rubble mound should be monitored and recorded by surveys and photographs. Construction and post-construction repairs should be reported in detail to permit judgment as to why maintenance was particularly heavy in certain portions. Details of the wave climate which caused the damage allow analysis of the wave-structure interaction (Bruun and Kjelstrup, 1981).

It is the opinion of Kjelstrup (1979), based on 25 years of experience in rubble mound construction in Norway, that poor workmanship has been the largest cause of rubble mound damages. He attributes this problem mainly to inadequate inspection and control during construction, and to a system of economic compensation which may seem to encourage the builder to cut corners and do shoddy work. It must be admitted that poor quality construction may result from the honest efforts of an experienced contractor to build an impractical cross-section. It is
Figure 7.24 Down-Blasting of Steep, Unstable Rubble Slope Below Water Level
(Bruun and Kjelstrup, 1981, p. 190)
reemphasized that the design as a whole, as well as in detail, must be developed with thought to constructability.

7.4 SUMMARY

The complex hydraulic stability of rubble mound cover layers is not understood completely from a theoretical viewpoint. Assumptions and simplifications are made to facilitate practical design and the use of empirical formulas yields satisfactory results. Hudson's formula, Equation 7.2, is the most widely used basis for computing the required armor unit weight. The popularity of this approach owes largely to its status as a "tried and true" method, and to the extensive field verifications of the laboratory $K_D$ values given in Table 7.1. In short, it seems to work. The $K_D$ values can be increased systematically to incorporate an allowance for damage into the design. For small-scale shore protection structures, the damage condition is usually the more realistic design case.

For the design of larger, more expensive mound structures, and especially those armored with concrete units, the limitations of empirical approaches can be significant. The effects of unit contact friction, angle of wave approach, wave period, and other influential phenomena should be studied in hydraulic laboratory model investigations. Research and developments in rubble mound technology are directed toward including these parameters in new theoretical design methods.

Each layer of the rubble mound cross-section must be graded such that smaller rocks in an underlayer cannot wash through the adjacent overlayer. The materials must also be heavy enough for hydraulic
stability. Details of the cross-section design recommended by CERC (1977) originated from these criteria. The CERC guidelines presented in this chapter note salient aspects of cross-section design and serve as a reasonable starting point. The actual cross-section will vary from this standard depending on the nature of the project and site.

Proposed rubble mound designs must be feasible not only with regard to hydraulic stability and economy, but to constructability as well. Engineers and planners must be familiar with construction techniques and difficulties. Awareness of the constraints of the construction season, equipment availability and other factors allows rational sequencing of placement operations. A program of visual inspection and other supervisory measures assure the good quality of the completed mound.

Structure geometry design, evaluation of materials, and construction planning must occur concurrently. All those involved in the design phase should communicate their continuing progress to the other planners. An informed and appropriately weighed selection from among the alternatives analyzed will result in the optimum mound design.