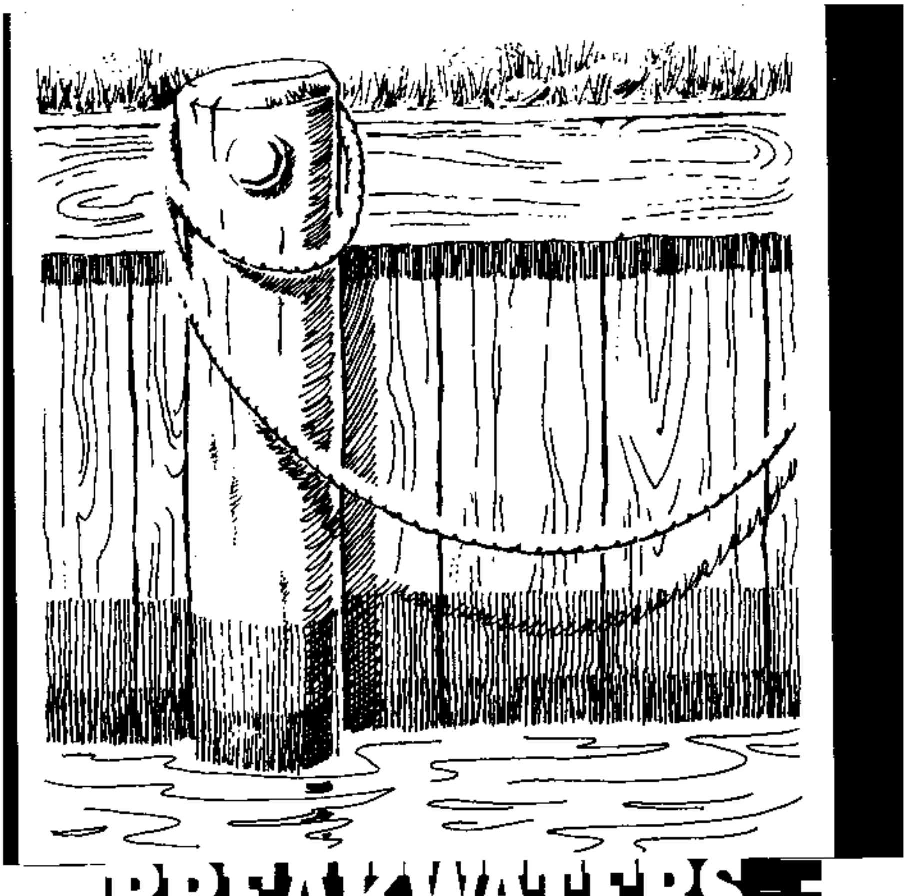
COASTAL STRUCTURES HANDBOOK SERIES



BREAKWATERS JETTIES & GROINS

Laurie A. Ehrlich
and Fred H. Kulhawy

NEW YORK SEA GRANT INSTITUTE

FIRST IMPRESSION

This manual is part of the Coastal Structures Handbook Series. The series is being prepared for the New York Sea Grant Institute by the Geotechnical Engineering group at Cornell University, coordinated by Fred H. Kulhawy.

COVER DESIGN: DICK GORDON

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New York Sea Grant Institute
State University of New York
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Albany, New York 12246

FIRST IMPRESSIONS ARE SELECTED UNPUBLISHED SEA GRANT RESEARCH PAPERS AVAILABLE FOR THE PRICE OF PHOTOCOPYING.

BREAKWATERS, JETTIES AND GROINS: A DESIGN GUIDE

by

Laurie A. Ehrlich and Fred H. Kulhawy

Report
to
New York Sea Grant Institute
Albany, New York

by

School of Civil and Environmental Engineering
Cornell University
Ithaca, New York

March, 1982

This report will constitute a chapter in a manual entitled, "Analysis, Design and Construction of Coastal Structures". This manual is being prepared for the New York Sea Grant Institute by the Geotechnical Engineering Group at Cornell University, and is being edited by Fred H. Kulhawy and Philip L.-F. Liu.

PREFACE

The analysis, design and construction of coastal structures is of great concern to a broad cross-section of the population living near major fresh and salt water bodies. Realizing this concern, the New York Sea Grant Institute instituted a project to develop a manual to assist a variety of user groups in addressing the problems associated with the development of coastal structures and coastal facilities. Although the engineering community will find the manual to be of use, the focus of this manual has been to develop a simplified user's guide which focuses on the analysis, design and construction of coastal structures. The emphasis has been on understanding the structures and their behavior, minimizing higher level mathematics, and presenting design charts and design examples for smaller scale structures, typical of those of importance to a small community and the individual homeowner. Large scale developments should be handled by design professionals with expertise in the field.

This project was initiated in late 1977 by the New York Sea Grant Institute and has been developed by the School of Civil and Environmental Engineering at Cornell University. The project was initiated by Drs. Fred H. Kulhawy and Dwight A. Sangrey. Dr. Sangrey left Cornell before much progress was made, and subsequent work has been supervised by Drs. Fred H. Kulhawy and Philip L-F. Liu.

Under the auspices of this project, the following reports have been prepared and submitted to New York Sea Grant:

1. Regulatory Processes in Coastal Structures Construction, August 1979, by Susan S. Ronan, with the assistance of Dwight A. Sangrey

2. Coastal Construction Materials, November 1979, by Walter D. Hubbell and Fred H. Kulhawy

3. Environmental Loads in Coastal Construction, November 1979, by Walter D. Hubbell and Fred H. Kulhawy

4. Analysis, Design and Construction of Pile Foundations in the Coastal Environment, April 1981, by Francis K.-P. Cheung and Fred H. Kulhawy

5. Breakwaters, Jetties and Groins: A Design Guide, March 1982, by Laurie A. Ehrlich and Fred H. Kulhawy

Additional reports to be completed in the near future include:

a. Bulkheads

b. Boat Ramps

c. Docks, Piers and Wharves

Further topics to complete the manual should be initiated prior to the end of 1982.

ABSTRACT

Small-scale breakwaters, jetties and groins are constructed to control coastal erosion, and to stabilize beaches and inlets. The functional and structural design of these coastal structures are presented in this study.

Siting of the structures requires a background knowledge of the dynamics of littoral processes. Functional design characteristics include orientation, length, height, spacing and other geometrical components. These topics are discussed in their relation to structure purpose and operation. The structural configurations highlighted are mound and wall type constructions and low cost shore protection methods.

Rubble mounds are the most common form of breakwaters, jetties and groins. The structural design of mounds is the focus of the second part of this study. Preliminary design considerations include quantification of environmental loads and assessment of geotechnical conditions. The selection of rock for rubble mound construction is an important facet of design. Guidelines for the evaluation of rock durability in the coastal zone are presented. In the final section, technical design methods are reviewed. The design emphasis is on practical, sound engineering procedures.

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LIST OF SYMBOLS

English Letters

A - area

B - structure crest or beam width

C_t - wave transmission coefficient

D - FTB draft

D - percent damage to rubble mound

D_t - tire diameter

d - water depth

d_b - depth of water at breaking wave

d_n - particle diameter of n^{th} percentile of soil sample

d_{nb} - base soil particle size

d_{nf} - filter particle size

d_s - water depth at structure toe

f - weight factor of armor unit

g - gravitational acceleration

H - design wave height

H_b - breaker height

$H_{D=0}$ - no damage wave height

H_i - incident wave height

H_o - deepwater significant wave height

H_o' - equivalent deepwater wave height, unaffected by refraction and friction

H_s - significant wave height

H_t - transmitted wave height

H_1 - average height of highest 1% of all waves for a given time period

H_{10} - average height of highest 10% of all waves for a given time period

I_f - fracture spacing index

i_f - hydraulic gradient over the 1 inch of soil adjacent to filter fabric

i_g - hydraulic gradient over the 2 inches of soil between 1 and 3 inches from filter fabric

K' - diffraction coefficient

K_D - armor unit stability coefficient

K_R - refraction coefficient

k_Δ - layer coefficient of rubble structure

L - wavelength

L_o - deepwater wavelength

m - beach slope

N - climatic index

N_r - required number of individual armor units

n - number of layers of armor units

n - number of armor units across rubble structure crest

P - average proosity of rubble mound cover layer

r - total rubble layer thickness

S_r - specific gravity of armor units

T - wave period

t - time

W - weight of individual armor units

W_{eq} - equivalent weight of armor stones

W_n - weight of the n^{th} underlayer stone

X - distance from shore to detached breakwater

X_b - distance from shore to breaking wave point

X_p - breaker travel distance

<u>Greek Letters</u>

α - upper limit of observed d_b/H_b

α_b - angle between breaking wave crest and shoreline

β - lower limit of observed d_b/H_b

γ_r - unit weight of armor units

γ_w - unit weight of water

η - wave suppression efficiency

θ - angle of rubble mound seaward slope

μ - coefficient of friction

ξ - surf similarity parameter

ϕ - angle of internal friction

LIST OF CONVERSIONS

To Convert From	To	Multiply By
°F	°C	°C = (5/9)(°F -32)
foot	meter	0.3048
inch	millimeter	25.4
pound	Newton	4.448
lb/ft	kN/m	0.0146
lb/ft^2	kN/m^2	0.0479
lb/ft^3	kN/m^3	0.1571
lb/in^2	kN/m^2	6.895

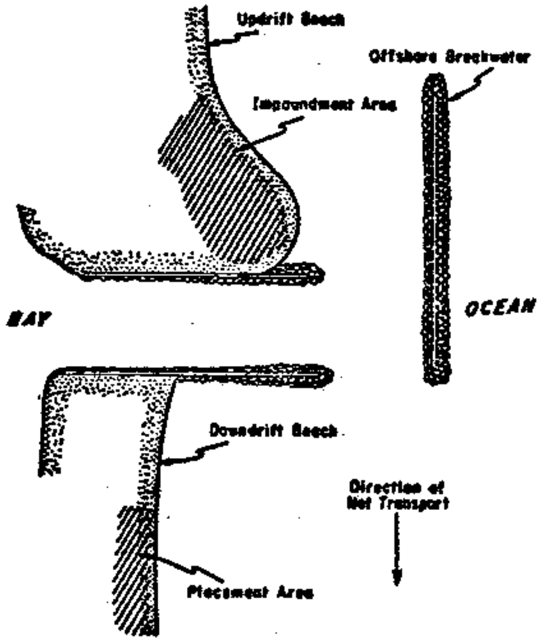
CHAPTER 1

INTRODUCTION

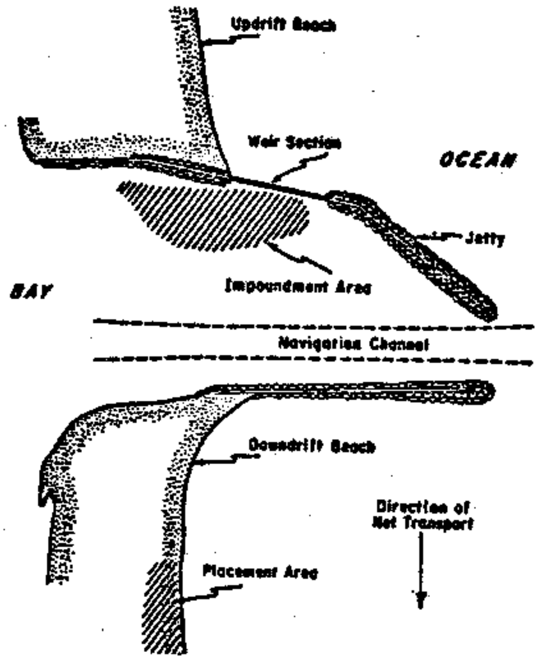
Coastal protection structures are constructed to control coastal erosion, and to maintain beaches and inlets. Erosion occurs when more material is eroded than is deposited. It can result from natural sources such as storms, tides and sea level rise, or can be caused by man-made structures in the coastal zone. Coastal structures, then, are part of the problem, as well as one of the solutions.

Breakwaters, jetties and groins are "process alteration structures." They extend into the water and stabilize the shore region by actively changing the natural equilibrium of the coastal processes. Simple examples of these structures are illustrated in Figure 1.1. Groins are strictly for shore protection; by trapping littoral drift they are intended to retard erosion of the affected beach. Jetties are built for inlet and harbor protection; they accrete littoral material that would otherwise fill the inlet and stabilize the location of navigation channels. Breakwaters can function predominantly as either shore or harbor protection or can serve both purposes at once.

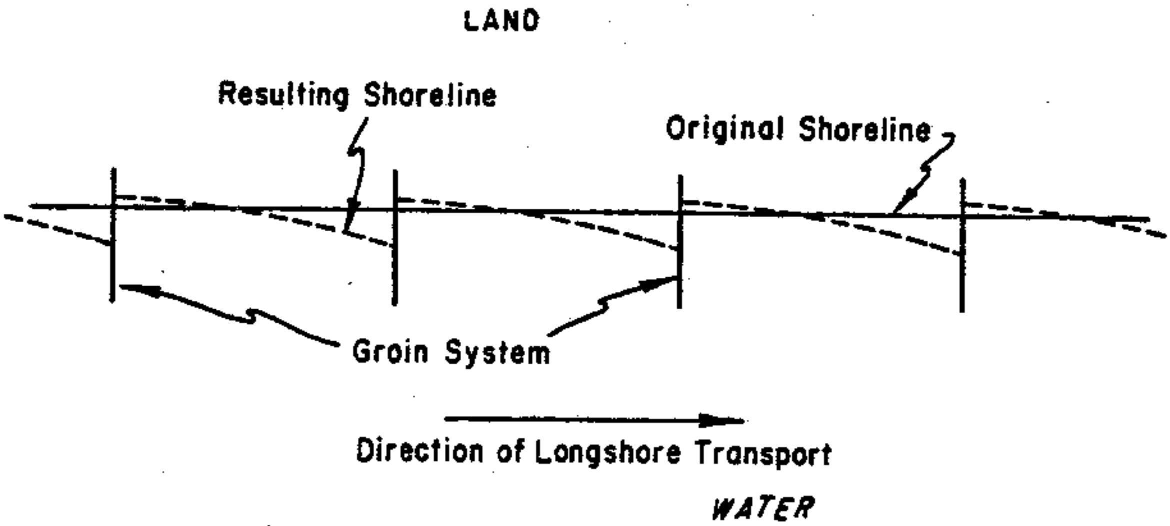
The design and behavior of process alteration structures may seem deceptively simple in concept. However, the coastal environment in which they are situated is characterized by complex and variable processes that are often difficult to quantify. The actual operation of breakwaters, jetties and groins, then, is correspondingly complex. Structural design must be guided by an understanding of the prevailing shoreline processes. The negative effects of inadequate and improperly designed "protection" can be far reaching and severe.



a. Offshore breakwater



b. Weir jetty



c. Groin field

Figure 1.1 Process Alteration Structures (Weggel, 1981, p. 12; CERC, 1977, p. 5-35)

There are many aspects to the philosophy of shore protection. Groins and other structures that interfere with natural coastal processes usually increase erosion rates on adjacent areas. Some extremists advocate prohibition of the use of shore protection structures, to slow the rapid exploitation of coastal regions. Alternatively, many consider protective structures as a necessary adjunct to development of the coastal zone. It is generally agreed, however, that structural solutions to coastal erosion problems should be used as carefully and selectively as possible. A current trend is toward the use of non-structural methods to control or lessen the problems caused by erosion, rather than to stop the erosion. Some of these methods are listed in Table 1.1 and should always be considered as alternatives to structural solutions, especially: 1) when economics do not clearly favor structural solutions, 2) when structures would result in unacceptable adverse impacts, and 3) when structural solutions might encourage development that would in turn necessitate increased and more costly shore protection (Sanko and Smith, in preparation).

The first part of this study focuses on the functional design of breakwaters, jetties and groins. It is essential to understand littoral processes and inlet hydraulics prior to planning structures which will alter them. With this background the fundamental nature of the structures - how they work - can be explored. Structure orientation, length, height, spacing and other geometrical components delineate the area which will be protected. Various combinations are examined in the design phase to discover the set of characteristics which will provide the required protection at the lowest cost and with a minimum of negative effects. Structures which impose the least on the coastal

Table 1.1 Non-Structural Methods for Erosion Control (compiled from Sanko and Smith, in preparation)

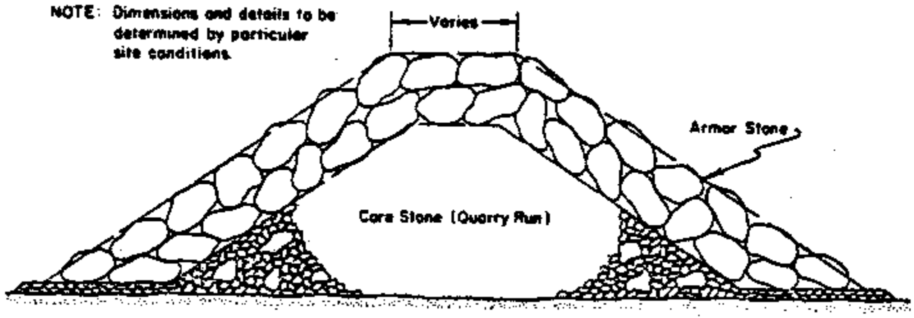
Active Methods	Passive Methods
Move threatened structures	Land-use controls
Vegetative methods	Coastal setbacks
Remedy the cause	Do nothing

zone, i.e., are as short and low as possible, are preferable from an environmental viewpoint.

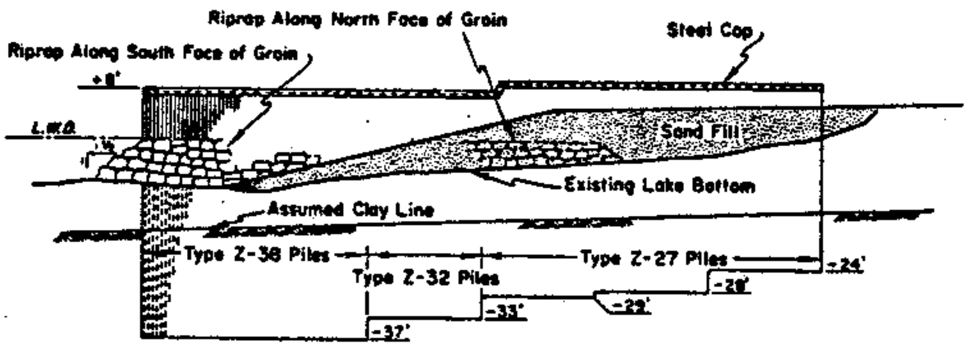
Structural design is central to the second part of this study. Structure configurations common to breakwaters, jetties and groins are classed in two general categories, mound and wall constructions. A third category, low cost shore protection, reflects a recent trend toward developing low cost, smaller-scale protection devices appropriate for implementation by private landowners. Some typical structural alternatives are shown in Figure 1.2. Designs are further individualized by their material components. Commonly used materials include rock, concrete, steel and wood, and newer options, such as gabions and synthetic fabrics. The structural type which is suitable depends largely on the scale and purpose of the project. The emphasis of this study is on engineering smaller-scale shore stabilization structures and, consequently, less massive designs. Rubble mounds are the most common form of breakwaters, jetties and groins, and are singled out for more extensive design treatment.

The destructive forces of the sea, in the form of waves and currents, impose the largest design loads on shore protection structures. Correct characterization of the design wave is key to proper rubble mound design. Ice, earthquakes, and impact pressures may also inflict damaging forces and should be evaluated on a site-specific basis.

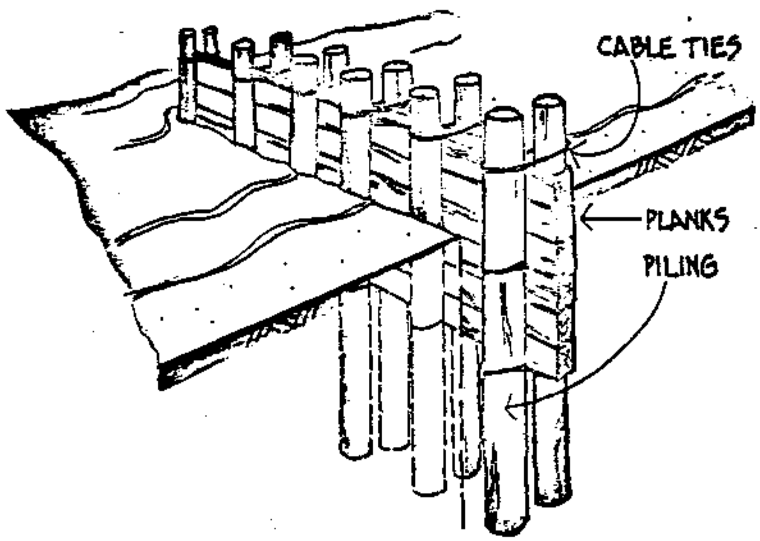
The significance of geotechnical conditions in rubble mound design is often underestimated. Problems generated by settlement, insufficient soil bearing capacity and toe scour can be critical to stability. These can usually be alleviated if they are anticipated, and countermeasures



a. Rubble mound



b. Cantilever sheet pile structure



c. Timber structure

Figure 1.2 Structural Variations (CERC, 1977, pp. 6-63 and 6-79; Rogers, Golden and Halpern, 1981, p. 20) are incorporated into the foundation scheme. Foundation design should be given as much attention as the structural design of the mound.

Selection of the rubble mound cover layer material is especially important, as stability depends largely on the hydraulic and structural integrity of cover layer elements. Rock is the most widely used material; however, few guidelines have been proposed for the evaluation of rock quality for coastal engineering applications. The discussion in Chapter 6 attempts to fill this void by suggesting procedures and standards for the investigation of rock durability. Rock parameters which influence quarry design are also reviewed. Concrete armor units have superior hydraulic stability characteristics and may be a feasible alternative at sites of more severe wave attack.

Current methods of rubble mound design are based on empirical formulas and recommendations. These approaches are generally adequate for the design of small-scale structures. Limitations to these procedures become increasingly important for larger, more expensive structures, and must be recognized in their application. Supplementary hydraulic model investigations should be performed whenever practical. Theoretical design must proceed with full awareness of construction practices and constraints. The proposed structure must be feasible with respect to constructability and economic concerns, as well as structural and hydraulic stability.

The information reviewed in this study comprises rational design guidelines, based on experience and field and laboratory verifications. The recommendations set forth are not absolute; no convenient "standard" design can prevail. A brief perusal of the contents will indicate the wide range of parameters and variables which contribute to shore protection analyses. The information presented should form a logical framework for the detailed assessment of a specific coastal erosion problem and aid in designing a structural protection solution.

CHAPTER 2

LITTORAL PROCESSES

A primary goal of coastal engineering design is to adapt the coastal region to man's benefit while maintaining a stable shoreline. The construction of breakwaters, groins, jetties and other nearshore structures mars the natural shoreline equilibrium, sometimes with disastrous consequences. To preclude such effects, coastal structures must be planned in harmony with their surroundings. A fundamental appreciation of coastal processes and the coastal environment is a necessary prelude to rational coastal engineering design.

Shoreline stability is dependent on the rate at which sediment is supplied to and removed from the shore. Sediment movement, or littoral drift transport, results from the interaction of waves, winds, currents, tides and other environmental forces (see Hubbell and Kulhawy, 1979b). Interrelated with the regional littoral processes is the existence of coastal inlets. The sedimentation, hydraulic and stability characteristics unique to inlets have been the subject of extensive research.

The science of coastline and inlet stability is complex. In this chapter, the subject is introduced and discussed qualitatively, to promote a simple understanding of the effects of littoral processes on coastal planning and the operation of shore protection structures. A more detailed treatment which covers, in particular, quantitative analysis methods may be referred to in Volume 1 of the Shore Protection Manual (Coastal Engineering Research Center, CERC, 1977).

2.1 NEARSHORE CIRCULATION

As waves approach the shore, they become shorter in wavelength and greater in wave height. They increase in steepness until they reach the limit of stability, then break. The breaker line delineates two distinct regions of the coastal environment. The surf zone reaches from the breaker line to the shore, and the offshore zone extends seaward of the breaker line (Silvester, 1974). These and other basic nomenclature which describe beach geometry are illustrated in Figure 2.1.

Significant sand transport takes place throughout the surf zone. The width of this zone is affected substantially by the wave climate. Higher waves break further offshore, widening the surf zone. Bottom topography is another contributing factor (CERC, 1977). The extent of the surf zone, then, is unique for each coastal region. Approximate surf zone widths for various coastal areas are given in Table 2.1.

Longshore Transport Mechanisms

Waves are the predominant impetus for littoral dynamics in the coastal zone. Their effect on sediment transport is two-fold: they initiate sediment movement, and drive the current systems that sustain the transport of littoral drift (CERC, 1977). Initially, the oscillation of waves in shallow water imposes stresses on the seabed that place sand in motion. Particles are lifted and rolled along the bottom in bed-load transport. Once in suspension, the sediment is carried along largely by wave-induced longshore currents.

The energy supplied by breaking waves results in a complex system of nearshore water currents. The nearshore circulation system (Figure 2.2) originates with the mass transport of water shoreward. Longshore

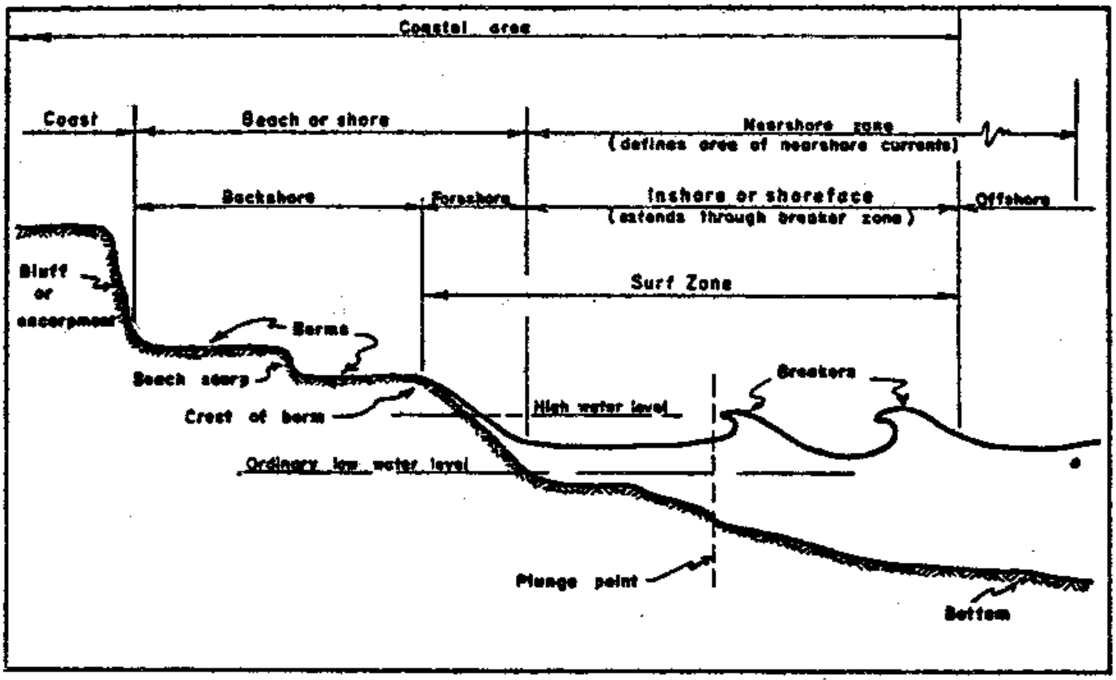


Figure 2.1 Beach Geometry - Related Terms (CERC, 1977, p. 1-3)

Table 2.1 Approximate Surf Zone Widths for Various Coastal Areas (compiled from CERC, 1977; Bruun and Manohar, 1963)

Coast	Depth to Which Surf Zone Extends Seaward	
	Feet	Meters
Atlantic	6	1.8
Gulf	3-4	0.9-1.2
Great Lakes - Less exposed	3-4	0.9-1.2
Great Lakes - More exposed	6	1.8
Pacific - Exposure dependent	7-10	2.1-3.0
North Sea	12-18	3.6-5.5

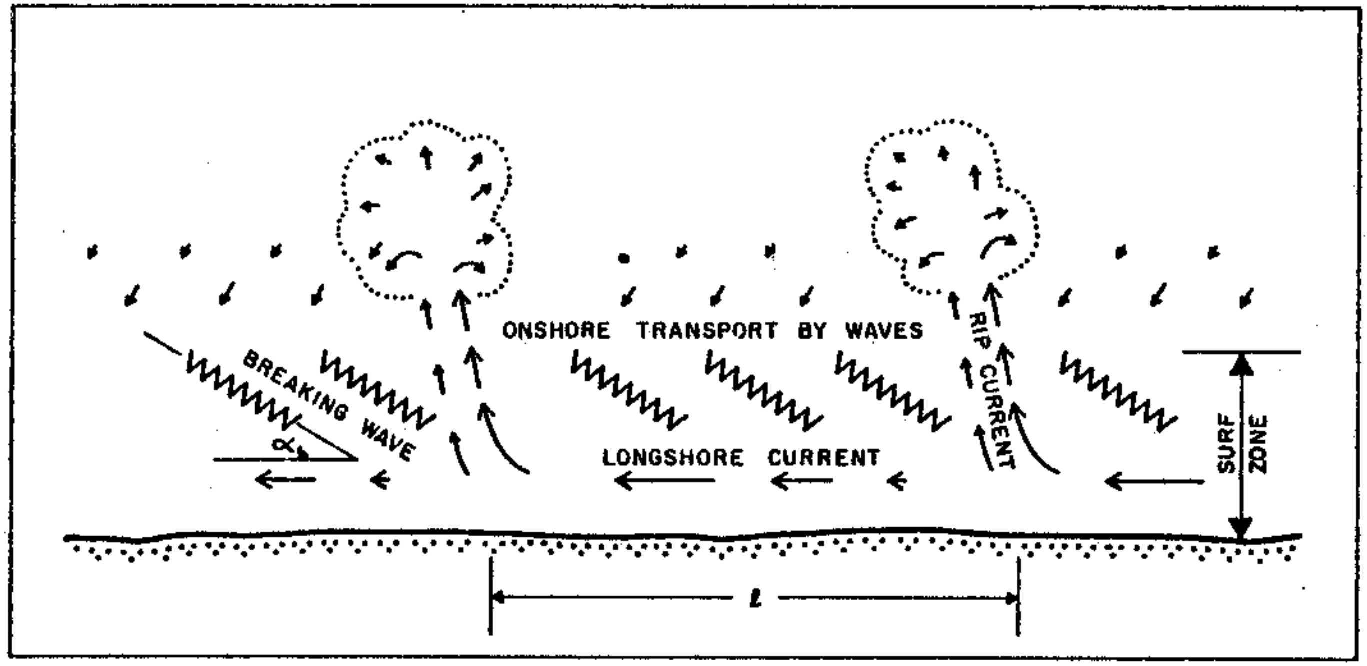


Figure 2.2 Schematic Diagram of the Nearshore Circulation System (Inman and Frautschy, 1965, p. 516)

currents are generated by the lateral components of oblique wave attack. These currents are the principal mechanism for maintaining longshore transport of sediment in the surf zone. Offshore transport is enabled by rip currents, which are concentrated jets that flow seaward through the breaker zone (Inman and Frautschy, 1965). The precise role of rips as sediment transporting agents has not been substantiated conclusively.

Transporting currents may be other than wave-induced. Tidal currents are especially effective in moving material in the vicinity of coastal inlets. Currents precipitated by storm surge also contribute to the net littoral movement. The increase in water level caused by tides and surges is significant, because it allows waves and longshore currents to act over a wider band of the beach profile (CERC, 1977).

Enclosed seas and landlocked lakes exhibit a different wave regime and, consequently, different characteristics of littoral drift. On waters of limited extent, waves are generated locally or have little space in which to disperse. Oceanic swell, which is instrumental in onshore transport, is absent or minimal. The resulting longshore transport is almost wholly confined to the surf zone and progresses at a slower rate than on open coasts, as demonstrated in Table 2.2 (Silvester, 1974).

Obstruction of Longshore Transport

Shore protection structures interrupt the natural dynamics of shoreline processes. Their operation combines two facets:

1. They form a physical barrier to the passage of littoral sediment.

2. They attenuate waves and reduce the transporting capacity of associated longshore currents.

LOCATION	DRIFT RATE (m³/yr)	PREDOMINANT DIRECTION	YEARS OF RECORD
Atlantic Coast			
Suffolk Co., N.Y.	255,000	W	1946–55
Sandy Hook, N.J.	377,000	N	1885–1933
Sandy Hook, N.J.	334,000	N	1933–51
Asbury Park, N.J.	153,000	N	1922–25
Shark River, N.J.	255,000	N	1947–53
Manasquan, N.J.	275,000	N	1930–31
Barnegat Inlet, N.J.	191,000	S	1939–41
Absecon Inlet, N.J.	306,000	S	1935–46
Ocean City, N.J.	306,000	S	1935–46
Cold Springs Inlet, N.J.	153,000	S	—
Ocean City, Md.	115,000	S	1934–36
Atlantic Beach, N.C.	22,600	E	1850–1908
Hillsboro Inlet, Fla.	57,000	S	—
Palm Beach, Fla.	115,000 to 172,000	S	1925–30
Gulf of Mexico			
Pinellas Co., Fla.	38,000	S	1922–50
Perdido Pass, Ala.	153,000	W	1934–53
Galveston, Texas	334,700	E	1919–34
Pacific Coast			
Santa Barbara, Calif.	214,000	E	1932–51
Oxnard Plain Shore, Calif.	756,000	S	1938–48
Port Hueneme, Calif.	382,000	S	1938–48
Santa Monica, Calif.	207,000	S	1936–40
El Segundo, Calif.	124,000	S	1936–40
Redondo Beach, Calif.	23,000	S	—
Anaheim Bay, Calif.	115,000	E	1937–48
Camp Pendleton, Calif.	76,000	S	1950–52
Great Lakes			
Milwaukee Co., Wis.	6,000	S	1894–1912
Racine Co., Wis.	31,000	S	1912–49
Kenosha, Wis.	11,000	S	1872–1909
Ill. state line to Waukegan	69,000	S	—
Waukegan to Evanston, Ill.	44,000	S	—
South of Evanston, Ill.	31,000	S	—
Outside the United States			
Monrovia, Liberia	383,000	N	1946–54
Port Said, Egypt	696,000	E	—
Port Elizabeth, South Africa	459,000	N	—
Durban, South Africa	293,000	N	1897–1904
Madras, India	566,000	N	1886–1949
Mucuripe, Brazil	327,000	N	1946–50

Table 2.2 Littoral Drift Rates along Coasts (Johnson, 1957, p. 1211-6)

The net effect of these mechanisms is a reduction in wave energy and accretion of sediment in the vicinity of the structures. These features are shown schematically in Figure 2.3. A secondary effect of accretion is erosion of downdrift areas, resulting from the lack of material supply. The operation and effects of breakwaters, jetties and groins are presented in more detail in Chapter 3.

Parameters of Sediment Transport

A rigorous evaluation of the characteristics of longshore sediment transport is essential to the planning of beaches and structures through the coastal zone. It is necessary to know the rate and direction of littoral drift to predict the effect of coastal construction on the beach profile. The transport rate depends on the local wave climate and the longshore component of wave power available to move sediment. Coastal orientation is an additional influencing factor, as demonstrated by the transport directions indicated in Figure 2.4. As shown, littoral drift along the northeast coast of the United States converges toward estuaries and bays, such as the Chesapeake and Delaware, because of changes in coast orientation (Komar, 1976).

Littoral transport may change in direction seasonally or in response to alterations in the site conditions. Of engineering interest is the net longshore transport rate, the difference between the quantities of sediment transported to the left and to the right of a point on the shoreline in a given time period. The net transport is generally small and, on some coasts, may be essentially zero. The net rate is much smaller than the gross rate, the sum of all the littoral drift passing a point on the beach. Longshore transport rates are

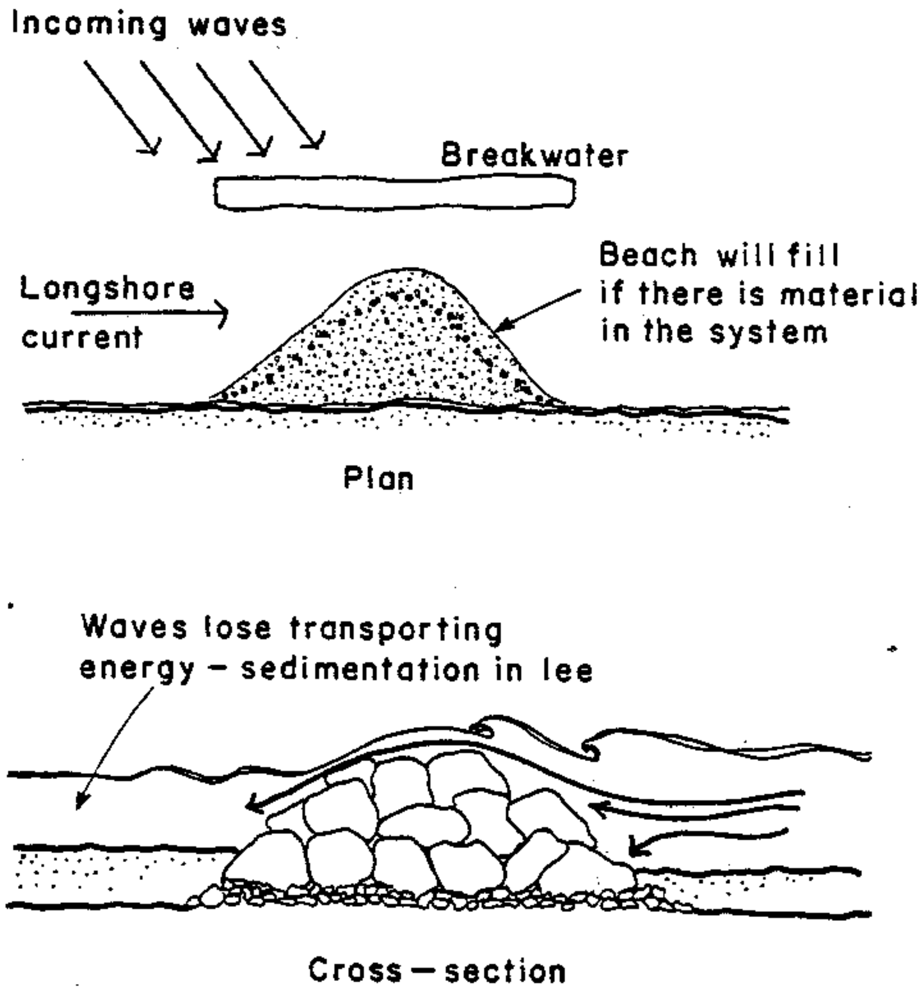
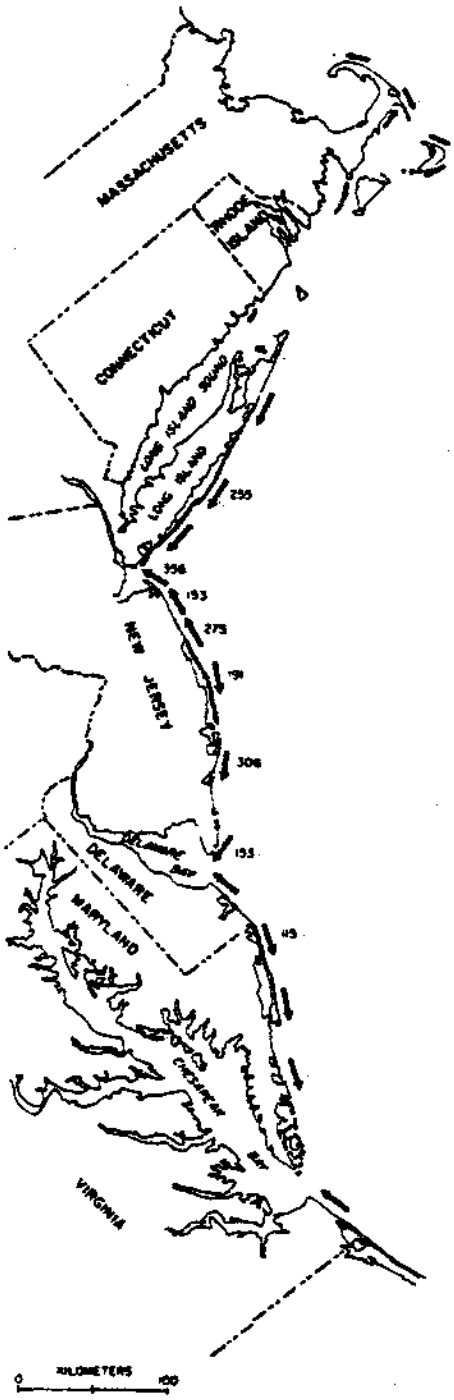


Figure 2.3 Operation of Shore Protection Structures (Rogers, Golden and Halpern, 1981, pp. 14-15)



(Magnitudes given in cubic meters x 10^3 per year)

Figure 2.4 Littoral Drift Directions and Magnitudes for the Northeastern United States (Komar, 1976, p. 219) expressed in units of volume per time increment. Typical net rates for various coasts of the world are compiled in Table 2.2.

Methods used to predict the longshore transport rate are detailed and illustrated with examples in CERC (1977). Briefly, the four basic methods are:

1. Adopt a proven estimated rate from a nearby site and modify for local conditions. Considerable engineering judgment is required in the initial choice and subsequent rate adjustments.

2. Compute the rate from historical data showing changes in the littoral zone topography. The value of the interpretation is proportional to the data reliability. Data sources include charts, maps, aerial photography, and dredging and beach fill records.

3. Measured or calculated wave conditions can be used to compute the longshore component of wave energy flux. This parameter can then be related to the transport rate with an empirical curve. This method may be used when neither of the aforementioned is practicable and the necessary wave data are available.

4. The gross transport rate can be obtained, through empiricisms, from the mean annual nearshore breaker height. The gross rate indicates the upper limit on the net drift rate. The essential factor in this method is the availability of wave data.

The status of a shore segment, whether advancing, eroding or at equilibrium, depends on the local balance of littoral drift. Evaluation of this budget of sediments is an invaluable tool in the planning of coastal works. The principle of conservation of mass can be applied to the regime of littoral sediments. Sedimentary contributions (credits) and losses (debits) are assessed for a given nearshore zone. Table 2.3 summarizes the various sources and sinks of sand. The relative importance of these elements and methods for their quantitative evaluation are detailed in Komar (1976) and CERC (1977). The net gain or loss of material is reflected as local beach deposition or recession

Table 2.3 Sand Budget Considerations (compiled from Bowen and Inman, 1966)

Sources	Sinks
Longshore transport into area	Longshore transport out of area
Onshore transport	Offshore transport
River and stream transport	Inlets and lagoons
Cliff, dune and backshore erosion	Beach storage
Biogenous deposition	Submarine canyons
Hydrogenous deposition	Mining and dredging
Wind transport into area	Wind transport out of area
Beach nourishment	

(Komar, 1976). A thorough sediment budget analysis includes the following features:

1. Delineation of the boundaries of the littoral cell under study. A cell is a self-sufficient segment of the coast which neither contributes sediment to, nor receives sand from, adjacent areas by longshore transport.

2. Identification of all sediment transporting processes within the cell and between adjacent cells. Mechanisms are ranked by the magnitude of their effect on the cell.

3. Careful estimation of the rates and volumes of supply and loss of material in the cell. Unknown rates may be assessed by balancing sediment gains against losses (Johnson, 1957).

2.2 COASTAL INLET DYNAMICS

A coastal inlet is a short, narrow waterway connecting an open ocean or lake with an inland water body, such as a bay or harbor. Inlets are of engineering importance in that they provide access to navigable waters. The mixing of sea and bay waters through the inlet is also of significance, as in the control of water temperature and salinity, the dilution of industrial and municipal wastes, and the migration of fish (Escoffier, 1977).

Inlet Hydraulics

As the ocean tides rise and fall, there occurs a water level difference between the bay and sea. To adjust this tidal imbalance, water flows into and out of the inlet, called a tidal inlet. The bay level is influenced to a lesser degree by inflow from its tributaries. Wind stresses, water density variations, and the Earth's rotation may also affect the inlet-bay system.

Practically all coastal inlets may be classified as tidal. On smaller water bodies, however, inlet flow is not caused by astronomic tides. On the Great Lakes, for example, reversing inlet currents are produced primarily in response to storm-generated seiches (Seelig and Sorenson, 1977). Lake water level fluctuations are also caused by wind setup and barometric pressure variations (CERC, 1977).

Littoral drift is carried into the inlet by the flood tide and partially deposited there as a bay or inner shoal, as shown in Figure 2.5. Some of the material transported by the seaward ebb tide is stored in the outer bar. The relative transporting capabilities of the flood and ebb currents determine the capacity of the inlet to flush itself of sediment and remain open. Flushing capacity is highly dependent on the volume of the tidal prism as compared to the quantity of sand supplied to the inlet from adjacent shores. An entrance channel that maintains a constant cross-sectional area across a sandy shore has attained a tidal flow equilibrium and is referred to as a scouring channel (Inman and Frautschy, 1965).

Sedimentation

Inlets act as large sand sinks for drift acquired from adjacent beaches (Figure 2.4). In this manner, they may considerably degrade the surrounding shoreline. In Florida, shore recession rates in the vicinity of tidal inlets are an order of magnitude higher (10 to 70 ft or 3 to 21 m per year) than average rates away from inlets (1 to 3 ft or 0.3 to 0.9 m per year) (Walton and Adams, 1976).

It is desirable to estimate the volume of sand lost to inlets from bordering shores. With this purpose, sand storage on inner and outer shoals has been the subject of some study. The inner shoals appear to reach an equilibrium shoaling volume with time, thereby decreasing the

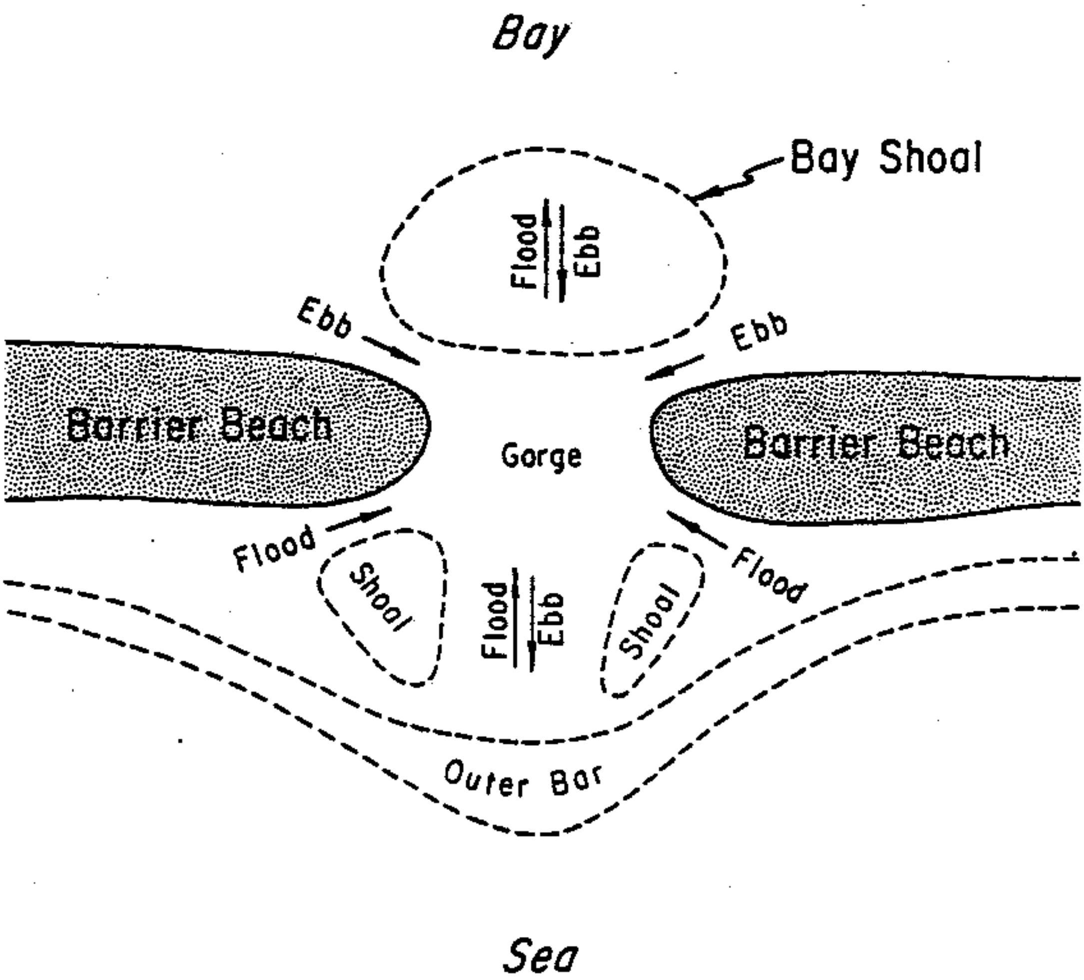


Figure 2.5 Barrier Beach Tidal Inlet (Escoffier, 1977, p. 7)

erosional influence on adjacent shores (assuming there is no dredging of the inner shoals). The volume of sand collected on the outer bar is strongly related to the local wave climate. In general, more material is stored on the outer shoals of low wave energy coasts. On high energy coasts, the wave action tends to transport the deposited sand back to shore (Walton and Adams, 1976). The relationship between tidal prism and outer bar storage is less clear. More flow (larger tidal prism) does not necessarily mean an increase in quantity of material in the ocean bars (Bruun, 1978).

A portion of the littoral drift naturally bypasses the inlet. At each ebb tide, the outgoing currents deposit sand in an often extensive shoal on the downdrift side of the outer bar. Wave action moves this material landward, where it can resume a course as longshore drift (Escoffier, 1977).

Stability of Inlets

The stability of inlets on littoral drift shores is a "dynamic stability" characterized by relatively small changes in inlet location, planform and cross-sectional area and shape. The stable condition implies a balance of natural forces. Littoral drift processes tending to shoal the channel are opposed and, for stability, balanced by the tidal currents flushing and maintaining the channel. Wave action, seasonal changes and other factors also play a role in inlet stability (Bruun, 1978).

It is of considerable engineering importance to be able to evaluate and predict the stability of inlets. A number of empirical and semi-empirical theories have been proposed to relate hydraulic characteristics, such as tidal prism and cross-sectional area, to inlet stability. These theories are approximations, however, and an inlet which is stable under normal circumstances may be unstable during a severe storm or other extreme conditions. These theories, too, are subject to revision as additional data become available (Escoffier, 1977). Presentation of inlet stability formulas and theories is not within the scope of this work. Reference to Escoffier (1977) and Bruun (1978) will provide a good introduction to the literature available on the subject.

Shoaling, closure and channel migration are among the symptoms of an unstable inlet. As described, the extent of shoaling is a function of the littoral drift magnitude, the flow capacity (tidal prism) and the wave action versus the shoal characteristics. In all cases, shoals tend to impede flow and close the inlet. Inlets may shoal quickly and be closed as a result of severe storms which deposit excessive volumes of sand in the channel and decrease the tidal prism (Bruun, 1978). An excess of sand introduced to the system generally causes the inlet to migrate. A portion of the incoming sand accretes on the updrift side as the downdrift flank erodes. Consequently, the inlet is displaced opposite the direction of the dominant longshore transport (Escoffier, 1977).

To be useful for navigation purposes, an inlet must possess sufficient dimensions, an adequate sediment flushing capacity and the attribute of stability. If these qualities are absent, it will probably be necessary to improve the inlet with regulatory works. Available methods include bank protection, dredging, jetties and artificial bypassing. In inlets with a high degree of natural stability, it may be possible to attain the desired improvement with dredging only. Aspects of sand bypassing and jetty design, a focus of this study, are covered in subsequent chapters.

2.3 SUMMARY

Shore protection structures act as obstacles to the natural movement of littoral drift and modify the equilibrium beach plan and profile. It is necessary to predict accurately the extent of such changes and their overall effect on the coastal environment. This task requires, foremost, a thorough conceptual understanding of the dynamics of sediment transport and the stability of tidal inlets. With this basis and the appropriate site data, the net longshore transport rate and sediment budget can be evaluated using the methods outlined. For projects involving a coastal inlet, its stability characteristics must also be determined. These analyses are typically performed in the initial stage of project planning, long before the structural design is formalized.

CHAPTER 3

FUNCTIONAL DESIGN

Breakwaters, jetties and groins alter the natural site processes in an attempt to stabilize and protect expanses of the coast. Groins are built to protect the shoreline from erosion, jetties are intended to protect inlets and adjacent areas, and breakwaters may serve either or both purposes. This chapter explores the fundamental nature of each structure by answering the basic questions of what each does and why, where and how each operates.

Functional characteristics of shore protection structures define the precise way in which they attain their purpose. Functional design involves delineation of the following general aspects:

1. Mechanics of operation - How they work; the manner and degree to which the proposed structure will attenuate waves and accrete sediment.
2. Physical layout - Where they work; the placement of the structure on the site, i.e., its position relative to the shoreline, and geometrical components, such as height, length and spacing.
3. Structural design.

These three are interrelated; changes in the structural design may necessitate changes in the physical layout, and alterations in structure geometry change the mechanics of operation. Structural design is discussed only briefly in this chapter, in its relation to the other functional determinants. Chapters 4 through 7 concentrate more fully on structural design.

Breakwaters, jetties and groins share the same general theory of operation. The structure imposes a physical barrier in the nearshore zone and blocks the flow of littoral drift. The action of waves and currents is interrupted as well, resulting in a relatively calm water area on the downdrift side of the barrier. The way in which these two effects are combined for shore protection purposes is different for each structure.

Length and height are the two basic geometrical parameters. Alignment relative to the shoreline and to the predominant direction of wave attack will have a significant effect on the operation of the structure. When more than one structure is built, as in a jetty pair or a groin field, spacing is another important variable. These components are described for each of the structures in the following sections.

Breakwaters, jetties and groins all accrete littoral material and consequently upset the natural equilibrium, reducing the material supply to downdrift beaches. Depending on the functional design characteristics of the structure and aspects of the site, the resulting downdrift erosion can range from minor to catastrophic. The potential negative impacts of construction must be weighed heavily. Adverse effects can be reduced by keeping the lengths and heights of structures to an absolute minimum. Sand bypassing and nourishment techniques, described in Sections 3.3 and 3.4 respectively, also help to alleviate downdrift erosion. The surest answer is simply not to build the structure; this option must always exist as an alternative. Construction of a protection device which stabilizes one coastal area at the expense of adjacent areas is an abuse of shore protection principles.

3.1 BREAKWATERS

Breakwaters are constructed primarily to reduce or prevent wave action in an area which is to be sheltered. The waters directly behind the structure are shielded from wave action and are noticeably calmer than the seaward waters. If this calm area, or "wave shadow", encompasses the shoreline behind the breakwater, the shore will be protected from waves as well. Because the wave energy available for moving sediment decreases sharply in the breakwater lee, the rate of littoral transport in the protected waters is reduced. This often results in increased sediment deposition, a secondary consequence of breakwater installation.

Breakwaters can be classified as either harbor protection or shore protection structures, depending on the nature of the protected area, or may serve both purposes. As aids to navigation, they create sufficiently quieted waters to allow safe maneuvering of vessels and use of harbor facilities. Also, they may provide anchorages where small craft can seek refuge from storms. Breakwaters have been used in conjunction with sand bypassing techniques, forming a calm area from which a pipeline dredge can operate. On a lesser scale, a breakwater may be sited to shield a small docking facility or recreational beach from excessive wave action. In shore erosion control, a breakwater can be used to promote accretion of a protective beach.

Certain characteristics must be defined to describe a breakwater fully. These are:

1. Structural type - Whether bottom-supported or floating, and construction material used

2. Physical layout - Whether shore-connected or offshore
3. Geometrical components

The structural design and behavior determine, in large part, the mechanisms by which the breakwater attenuates wave action. The physical layout and functional design delineate the region which will be sheltered and the degree of protection to be afforded. For ease of discussion these three features are treated as separate items in this section; in fact, their effects are interrelated. Breakwater wave attenuation cannot be attributed solely to structure height, length, surface roughness or offshore distance, for example, but depends on the optimum combination of these to attain the required protection.

Physical Layout and Sediment Accretion

Breakwaters installed for harbor protection are predominantly shore-connected. A typical shore-connected breakwater is illustrated in Figure 3.1. The shore arm of the structure acts as a barrier to littoral drift between its seaward end and the limit of wave uprush on the shore. The littoral drift thus intercepted accretes on the updrift side of the shore arm. Correspondingly, shoreline recession occurs on the downdrift side, because of a lack of material supply. Once the capacity of the structure is reached, natural movement of sand past the structure resumes (CERC, 1977). A permeable or low weir section built into the shore arm allows some material to pass to the lee side of the breakwater. The sand caught in this impounding zone can be pumped to downdrift shores to prevent shoreline recession (Sanko and Smith, in preparation).

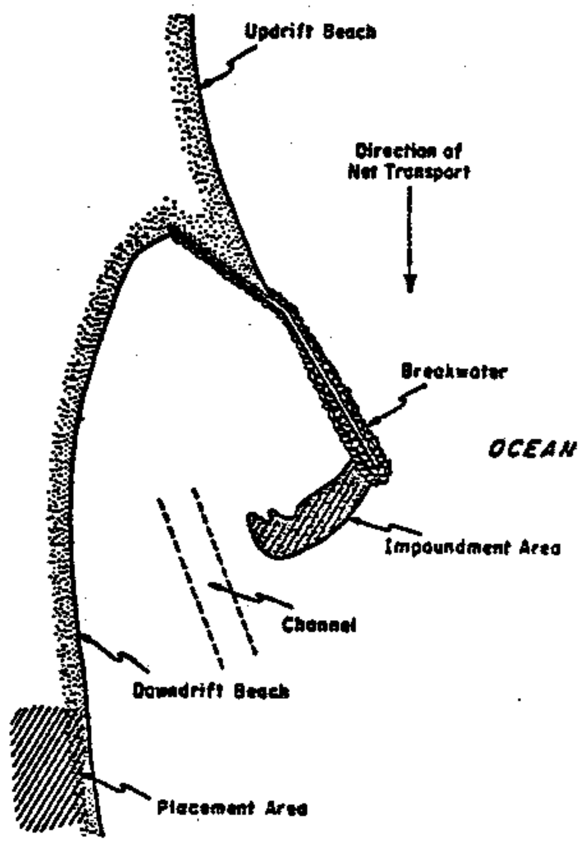


Figure 3.1 Shore-Connected Breakwater
(Weggel, 1981, p. 12)

Drift materials are also accreted by the sea arm of a shore-connected breakwater. Deposition extends along the seaward face and forms an impoundment shoal at the end of the breakwater, as shown in Figure 3.1. The accreted zone at the breakwater tip is also a choice location for sand bypassing equipment (CERC, 1977).

Offshore or detached breakwaters, usually placed parallel or subparallel to the shoreline, are the second common configuration (Figure 3.2). They are expensive to construct, but are quite efficient in attenuating wave action. They are generally located in deeper water than shore-connected breakwaters, jetties or groins and, therefore, influence a wider area of the littoral zone. If it is desired to trap littoral materials within this area, an offshore breakwater is a most capable means of interception. The structure dissipates wave energy, thereby slowing littoral transport and allowing deposition in its lee. Noticeable accretion begins as a bulge in the shoreline behind the breakwater. This shore projection then acts as a groin, encouraging the updrift deposition of sand (Figures 3.2 and 3.3). As the shoreline advances in this manner, the breakwater becomes increasingly efficient as a littoral barrier. If the breakwater is long relative to its distance offshore, and the littoral drift characteristics are conducive, deposition may continue until the projection joins the breakwater, forming a tombolo, as illustrated in Figure 3.3 (CERC, 1977).

Once the sand trapping capacity of the breakwater is reached, drift will pass the structure and again resume natural patterns of transport. However, depending on the dimensions of the breakwater and the littoral drift climate, this process could take several years. The downdrift

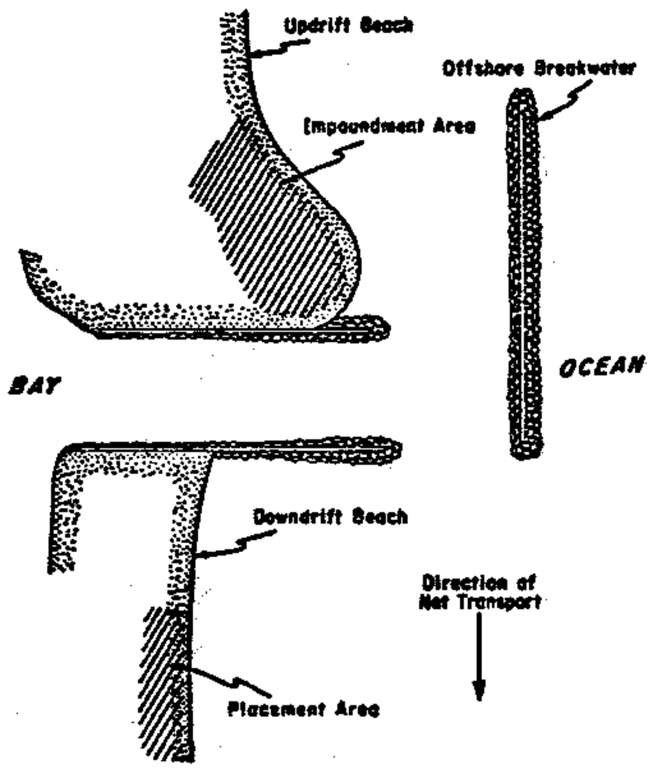
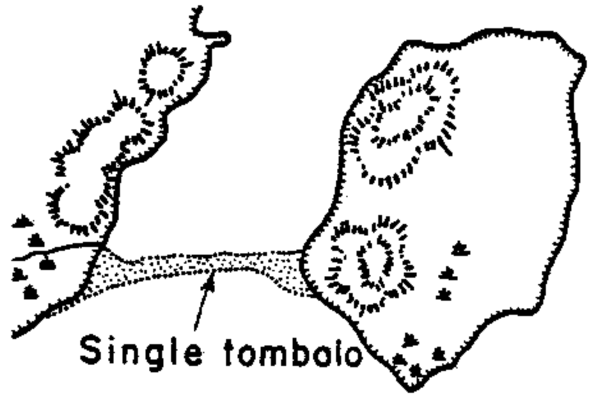
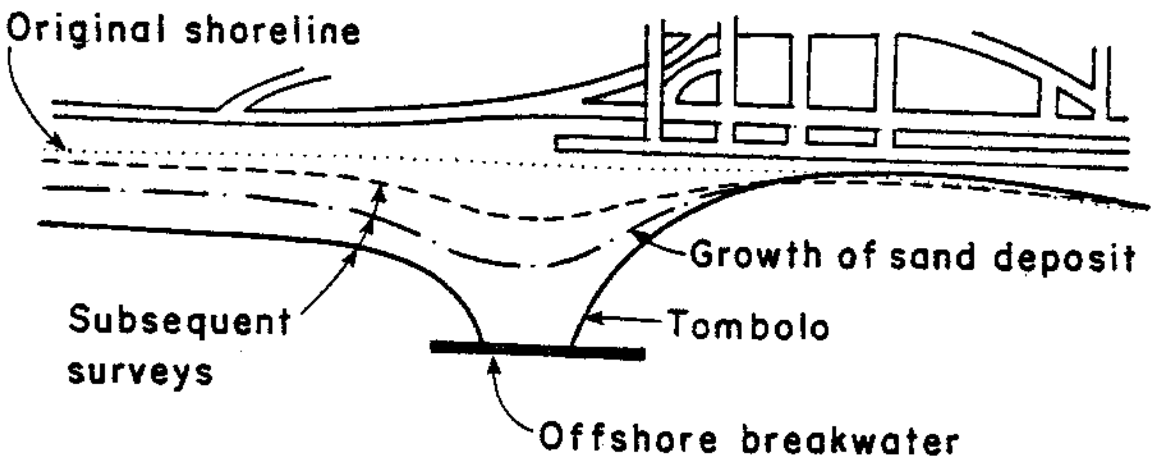


Figure 3.2 Offshore Breakwater (Weggel, 1981, p. 12)



a. Natural Tombolo Beach Form
(CERC, 1977, p. A-52)



b. Artificial Tombolo Formed by a Breakwater

Figure 3.3 Tombolos

shores necessarily recede during updrift accretion. If deposition is excessive, erosion may be severe. In many cases, the complete filling of the leeward area and the corresponding erosion are undesirable. The height, length, orientation and offshore distance of the breakwater must all be carefully planned and manipulated, so that the design objectives are fulfilled without causing adverse shoreline changes.

Offshore breakwaters are constructed in deeper waters for protection of harbor entrances. On a smaller scale, they may be located in shallow waters, of 4 to 6 ft (1.2 to 1.8 m) in depth. Inshore breakwaters are shore protection structures designed to build protective beaches by allowing shoreline advancement. They are normally constructed only in waters of 1 to 2 ft (0.3 to 0.6 m) in depth, on low slope beaches, and within 100 to 200 ft (30 to 60 m) of shore (Hanson, Perry and Wallace, 1978).

Structural Type and Wave Attenuation

Most breakwaters are bottom-supported. By definition, the forces acting on these structures must be sustained by the foundation materials. A dissimilar structural group, floating breakwaters, is the focus of Section 3.2. The two general classes of bottom-supported breakwaters are mound and wall constructions. Specific designs are further identified by their material compositions. A variety of these are illustrated in Chapter 4. Fully 95 percent of existing breakwaters comprise rock or concrete as their chief construction element. Steel and timber serve to a lesser degree (Quinn, 1972). Structural types are enumerated briefly here, with emphasis on their effect on breakwater operation, and are detailed in Chapter 4.

Rock mounds are the most common breakwater type used in North and South America. In the United States, offshore breakwaters are almost exclusively of rubble mound construction (CERC, 1977). The terms "rubble mound" and "breakwater" are even used interchangeably by some, although such usage is technically incorrect. Natural rock or specially shaped concrete units form the cover layers of the mound and provide stability against wave attack. An in-depth examination of the nature of these armor materials is presented in Chapter 6. Mound type installations can be topped by concrete superstructures to enhance their protection capabilities.

Massive vertical wall breakwaters are less commonly constructed in the United States. These gravity walls are founded on rubble mats to distribute the load over a sufficient area. Table 3.1 enumerates the advantages and disadvantages of the gravity wall breakwater as compared with mound structures. Other wall type configurations include concrete caissons and rock-filled cribs. Cellular sheet pile and sheet pile wall breakwaters have been successfully installed along less exposed shorelines (Quinn, 1972).

The structure configuration and construction materials influence the mechanism and efficiency of wave attenuation. Breakwaters disperse wave energy by a combination of absorption, reflection and transmission. When a wave impacts against a breakwater, the water, carried by its momentum, is forced up the face of the structure. Through this runup, wave energy is absorbed or dissipated by the structure. Structural characteristics, such as surface roughness and seaward slope, are major determinants of wave runup (Hubbell and Kulhawy, 1979b). Rubble mound faces, which are highly frictional, are more effective wave absorbers

Table 3.1 Vertical Wall Breakwaters - Advantages and Disadvantages (Quinn, 1972, p. 244)

Advantages	Disadvantages
1. Provide a larger harbor area and narrower entrance	1. Can be constructed only where foundation conditions are favorable
2. Harbor side of breakwater can be used for mooring wave ships	2. Not flexible, e.g., in adjusting to settlement and disturbance
3. Subject to more exact analytical analysis	3. Difficult to repair if damaged
4. Maintenance costs are nearly eliminated	4. Top elevation much higher than for mound structure
5. Where rock is economically unavailable, may save time and money	5. Construction requires more extensive and heavier equipment

than smooth surfaces. Similarly, the limit of wave uprush is smaller for shallower structure slopes. Hubbell and Kulhawy (1979b) summarize methods for computing wave runup.

Wave energy may be reflected back toward the main water body rather than be absorbed by the structure. Reflection may be a critical concern within a harbor, because multiple reflections and lack of dissipation can result in excessive wave agitation. Impermeable smooth vertical walls reflect almost all incident wave energy; that is, the reflected wave height is nearly equal to the incident wave height (CERC, 1977). Reflection decreases, in general, for rubble mound structures with increased structure slope, permeability and roughness. Rubble mounds can, however, reflect incident long period wave components, for normal or oblique wave approach. The short-crested waves produced by reflection remove bottom material along their crest paths. Resulting problems with scour in front and downcoast of a breakwater should be anticipated in design and maintenance schemes (Silvester, 1974).

Transmission of wave energy through coastal structures occurs, as at permeable breakwaters or semi-permeable rubble mounds. In some cases, transmission of a high percentage of long period wave energy will induce intolerable disturbance in the breakwater lee, as in a harbor complex (Dunham and Finn, 1974). More commonly, protection requirements are less stringent and some wave transmission can be allowed.

Geometrical Components

The siting and functional design of a breakwater depends intrinsically on the purpose of the structure, whether predominantly for shore or harbor protection. Breakwater geometry is defined by alignment and offshore distance, longshore length and height (crest elevation). The following paragraphs comprise comments and recommendations regarding these components. Simply stated, the overall guiding philosophy is one of minimums. A breakwater should be only as long and as high as is necessary to provide the required protection. It is well to consider the structure as an intrusion in the coastal environment, and to limit that intrusion as much as possible.

Siting. Selection of the construction site and alignment of the structure on that site are the two basic elements of siting. The first concern is usually limited by factors other than breakwater design. Location of shore protection breakwaters is largely dictated by a local need for erosion control. Harbor protecting devices are often developed as appurtenances to existing facilities or planned harbors for which the site has already been chosen. The second facet, placement of the breakwater on the site, is less clearly defined. The placement should be such that wave action is attenuated to allowable levels with a minimum of negative effects.

To reduce wave action, a breakwater necessarily alters the wave regime surrounding it. Most noticeable is the diffraction of waves around the tips of the barrier, as energy is transferred laterally along the wave crests (Figure 3.4). Since the underwater slopes are not typically flat, changes in wave height and direction of travel are expected to occur as well. This phenomenon is termed wave refraction. Siting studies should include wave diffraction and refraction analyses for the various proposed breakwater geometries and alignments. Graphical techniques for evaluation of these effects are reviewed by CERC (1977) and Hubbell and Kulhawy (1979b).

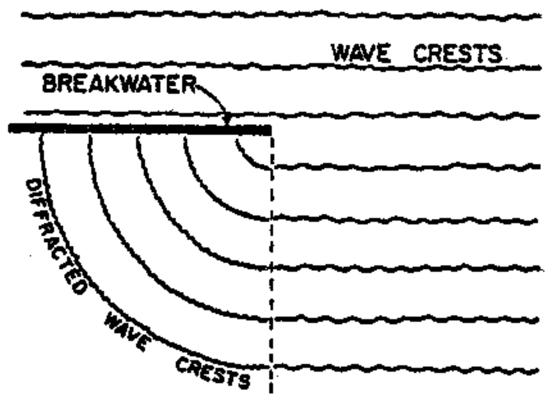


Figure 3.4 Wave Diffraction at a Breakwater (Komar, 1976, p. 114)

Wave diffraction analysis indicates the degree of wave attenuation attained within the protected area. For example, the typical diffraction diagram sketched in Figure 3.5 yields diffraction coefficients, K', of less than 0.5 within the breakwater geometric shadow. Thus, wave heights are decreased by more than 50 percent by the breakwater (CERC, 1977). The effects of diffraction on adjacent shorelines should also be examined. Focusing of wave energy can result in significant downshore beach erosion, as shown schematically in Figure 3.6. A more exact evaluation of attenuation combines the effects of diffraction and refraction. Although a definitive theory is lacking, Ippen (1966) may be consulted for an approximate graphical approach to this complex problem (Figure 3.7).

In the absence of other controlling criteria, the customarily recommended alignment is roughly perpendicular to the primary direction of wave attack. In this position the structure can most efficiently intercept wave action with the minimum longshore length (Dunham and Finn, 1974).

Length and Offshore Distance. Longshore length, d, and distance offshore, D, are interrelated parameters. The ratio of D/d is often used to evaluate the impact of offshore breakwaters on the littoral zone. Inman and Frautschy (1965) observed that, along the southern California coast, pronounced accretion does not occur when the D/d ratio is between three and six. Noble (1978) confirmed the earlier finding, acknowledging that the effect on adjacent shorelines is not discernible for D/d equal to six. These values are simply guidelines and may be inappropriate for certain combinations of waves, littoral action and

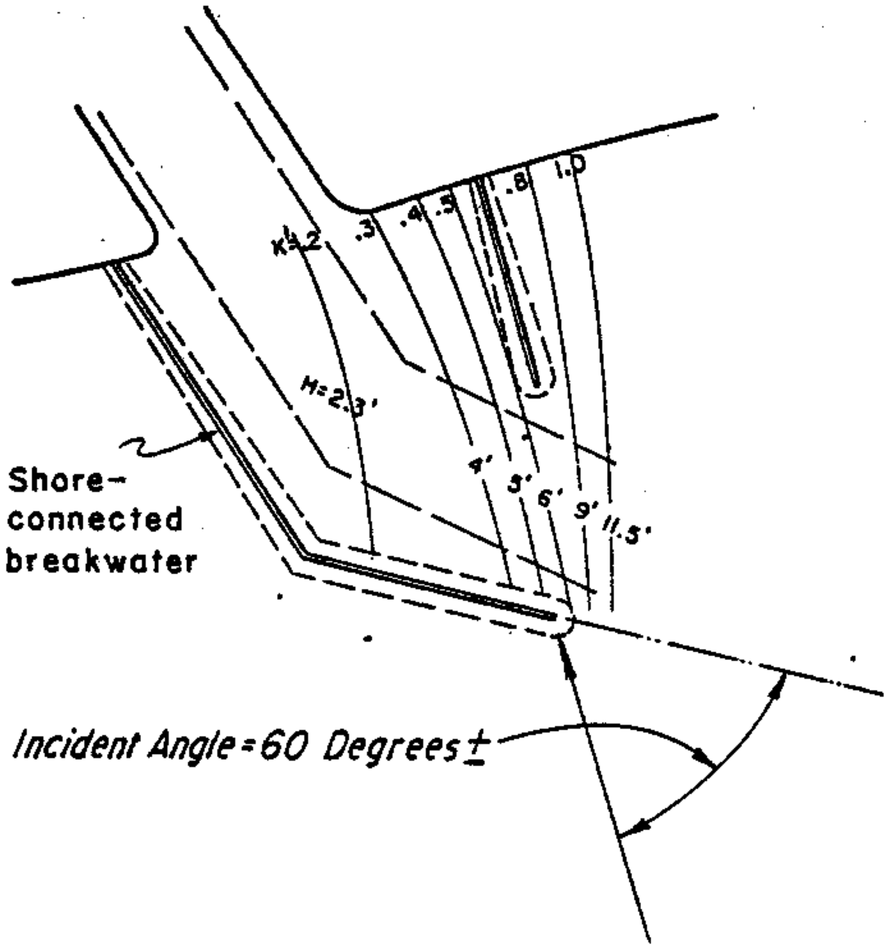


Figure 3.5 Typical Diffraction Diagram for a Shore-Connected Breakwater (Dunham and Finn, 1974, p. 27)

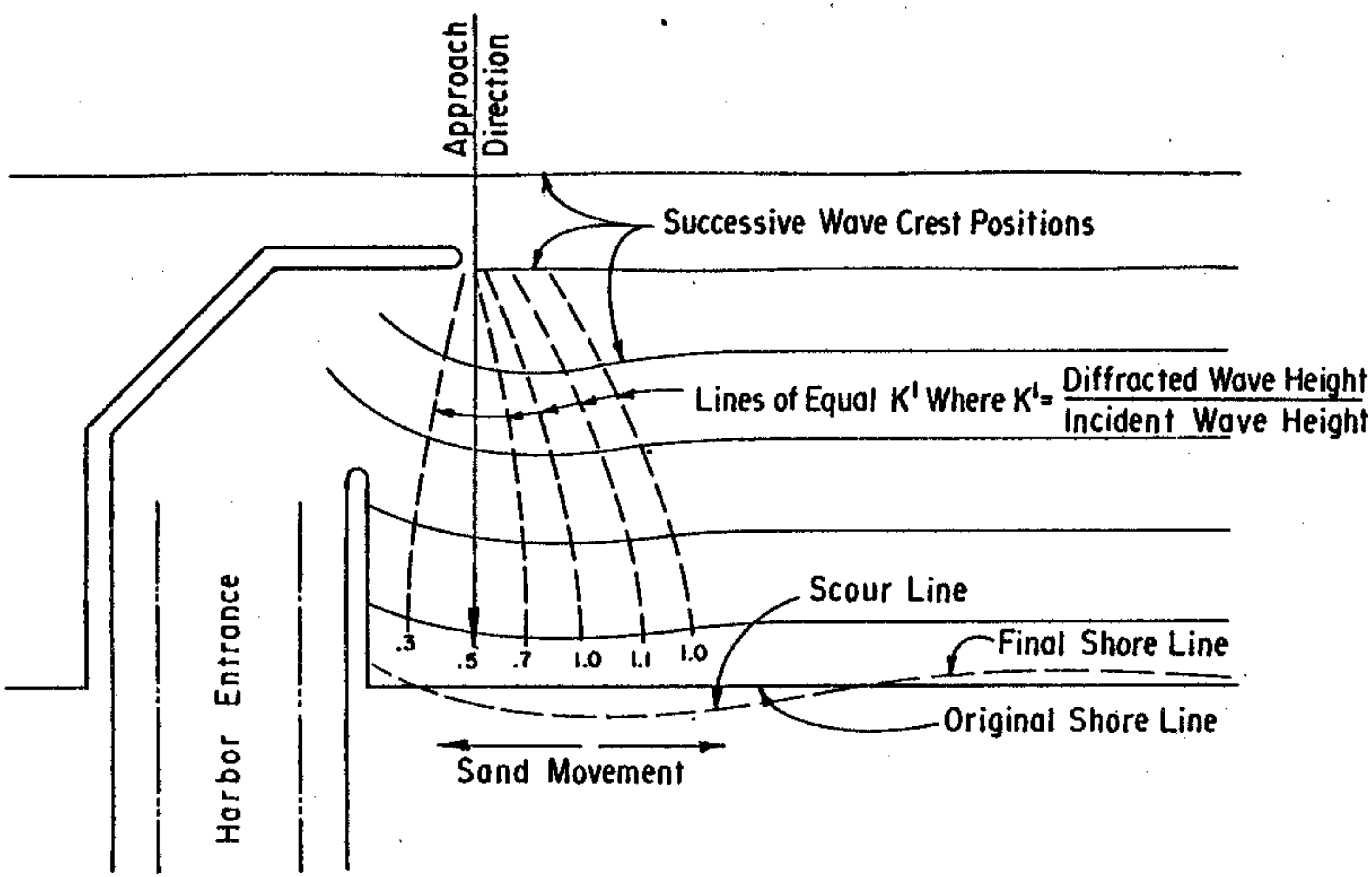


Figure 3.6 Littoral Transport and Beach Erosion due to Wave Diffraction (Dunham and Finn, 1974, p. 58)

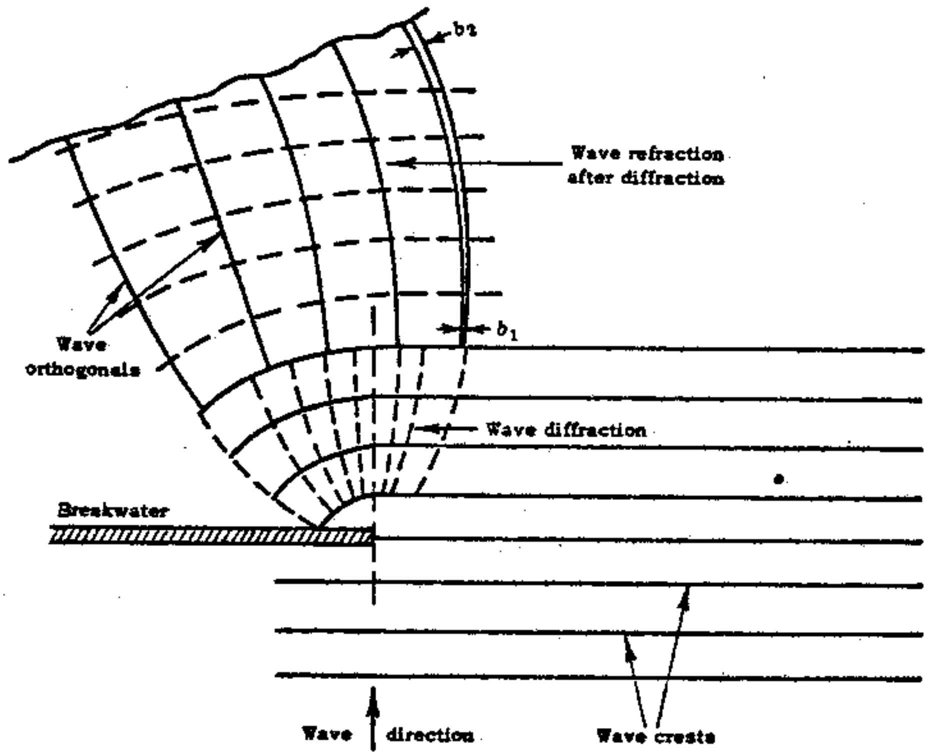


Figure 3.7 Combined Refraction and Diffraction (Ippen, 1966, p. 273)

other site specific characteristics. They should not be used in place of engineering analysis and verification.

The rule of minimums applies here, for environmental as well as economic reasons. As the water depth at the structure increases, project costs correspondingly multiply. There are exceptions to this general criterion. A breakwater may have to be lengthened to enable interception and retention of littoral drift for bypassing operations. The structure must not be so closely situated that it infringes on the water area needed for a harbor entrance or other purpose. Also, breakwaters low in height may have to be placed farther offshore to allow for leeward turbulence caused by overtopping waves (Dunham and Finn, 1974).

A series of short breakwaters may be installed rather than one of continuous length. The series will be less efficient as a littoral trap, but will otherwise function similarly to a long structure (CERC, 1977). For example, the use of a widely spaced breakwater series to stabilize a reclaimed shoreline in Singapore has been reviewed by Silvester and Ho (1972) and Chew, Wong and Chin (1974). The breakwaters were situated to act as headlands between which crenulate or scallop-shaped bays would form (Figure 3.8) Crenulate bays are proposed as equilibrium shoreline configurations. When this shape is attained, wave approach is wholly normal to the beach. This implies that there is no component of wave energy directed alongshore and, therefore, no opportunity for sediment transport. Preliminary progress reports on the Singapore project indicate that the headland breakwater system is effective in minimizing littoral drift and has provided an economic means of shore stabilization.

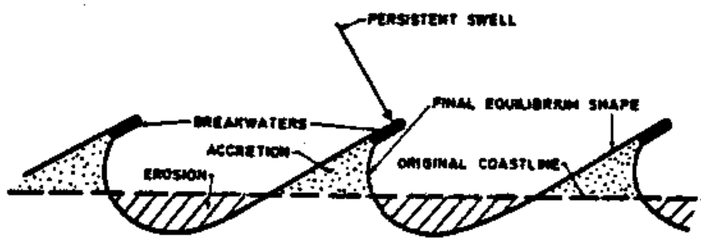


Figure 3.8 Crenulate-Shaped Bays (Chew, Wong and Chin, 1974, p. 1408)

<u>Height</u>. For conventional bottom-resting breakwaters, the principal consideration in determination of height is whether overtopping can or cannot be tolerated. This decision is dependent on the intended function of the structure. Excessive overtopping can cause agitation and choppiness in the leeward area and impede harbor operations. A breakwater whose crest permits no overtopping will form the most complete barrier to wave action and littoral processes (CERC, 1977). This is the most stringent height specification. For no overtopping, the top of the breakwater must be higher than the maximum design wave height, with maximum water level variations, plus anticipated wave runup.

In most cases, a certain amount of overtopping can be tolerated as long as disturbances in the protected areas do not exceed acceptable limits. Also, the overtopping waves must not in turn cause excessive waves in interior harbor areas. It is generally more economical to allow the minor damages caused by overtopping than to increase the crest elevation to nonovertopping dimensions. In the design of protective features for small craft harbors, breakwaters are usually planned to be overtopped by the design wave (Dunham and Finn, 1974).

It may actually be desirable in some cases to construct lower or overtopping breakwaters. Such design allows some wave action shoreward of the structure, thereby maintaining movement of littoral drift. This practice is especially appropriate where there is concern about excessive downdrift erosion.

When only partial protection is required, construction of a submerged breakwater might be suitable. These structures do not extend above the low water level and, because they are smaller, enable cost savings. They may also be more aesthetically pleasing than high breakwaters which obstruct the view of the water body. They permit relative continuity of littoral transport and allow water circulation in the leeward area. Accretion of materials passing over the crest is also enhanced. In low wave energy, high littoral drift environments, inshore submerged breakwaters are useful in forming and maintaining raised or "perched" beaches (Sanko and Smith, in preparation). These low-lying structures require warning markers so that they are not hazards to navigation and recreation.

An innovative variation of the submerged breakwater is reported by Zwamborn, Fromme and Fitzpatrick (1970). In an effort to protect and improve the beaches of Durban, South Africa, recommendations were made for the construction of an "underwater mound." The proposed mound is built of fine to medium sands supplied by adjacent harbor dredging works, to a length of 4.5 km (2.8 mi), 1200 m (0.7 mi) away from and parallel to shore, reaching to 7.3 m (24 ft) below LWOST (low water ordinary spring tide), with side slopes of 1:25 and a crest width of 61 m (200 ft). Extensive model testing suggested that the structure would remain stable under most wave conditions. This conclusion has been substantiated by prototype measurements. The accretionary patterns in the lee of the completed structure section indicate that the underwater mound offers an effective beach protection scheme.

3.2 FLOATING BREAKWATERS

Floating breakwaters are installed for the same purposes as bottom-supported breakwaters (Section 3.1) and, on that basis, belong to the same functional group of structures. However, floating devices are structurally and operationally unlike the conventional bottom-resting mounds or walls and, from that view, are quite dissimilar structures. Because they are in many ways unique, and because their use is particularly appropriate for smaller-scale projects, floating breakwaters warrant a separate discussion. In this section their functional and structural characteristics, strengths and disadvantages are investigated.

The concept of mobile, floating barriers to wave action dates from 1842 (DeYoung, 1978). Floating breakwater technology expanded during World War II, in response to the need to protect adequately the amphibious naval operations with structures which were easy to assemble and transport. Currently, the recreational boating market poses the largest demand for these devices. The increasing need for low cost protection of marinas and harbors provided the incentive for recent developments in floating breakwater technology.

The advantages and disadvantages of floating breakwaters, as compared with the more common bottom-resting structures, are summarized in Table 3.2. The listed considerations are important and warrant special attention. Floating structures are suited to a wide variety of civil uses, as indicated by Table 3.3.

Many floating breakwater designs have been proposed. The precise manner in which each device combines various damping mechanisms to attenuate wave action differs with the structural design. Details of operation are examined in the subsequent discussion on structural configurations.

Although the technological feasibility of many floating breakwater designs can be established in the laboratory, their use is often

Table 3.2 Floating Breakwaters - Advantages and Disadvantages

Advantages	Disadvantages
1. Usable where fixed breakwaters are not feasible because of poor foundation conditions, deep water or sediment transport problems	1. Provide less wave protection than bottom resting breakwaters
2. Low initial cost in deep water	2. Do not effectively damp waves of long period or low steepness
3. Material and construction costs are lower for some types	3. Can fail to meet design objectives abruptly, with no progressive structural damage as warning (as for long period waves)
4. In general, require little heavy equipment and erection time (as FTB)	4. Ongoing maintenance costs may exceed those for fixed breakwaters
5. Generally do not interfere with water circulation, sediment transport, fish migration	5. Cannot remain moored in icing conditions
6. Free from scour	6. Shorter structure life
7. Continued effectiveness during seasonal water level fluctuations	7. Lack of open-water prototype data
8. May be moved as protection needs change (multiple use potential)	8. Uncertainties in magnitudes of applied loads dictate conservative design principles and increase costs
9. Suitable for temporary protection	9. Some materials used may be unaesthetic (as tires of FTBs)
10. May be repaired in the water	10. Public reluctance to accept
11. Low profile may be aesthetic advantage	
12. May enhance biological resources by acting as an artificial reef	
13. Collects debris and attracts sea gulls away from recreational boats	

Table 3.3 Applications of Floating Breakwaters (compiled from McGregor and Miller, 1978)

1. Protection of inshore recreational boat mooring facilities
2. Shoreline erosion protection
3. Creation of safe natural anchorages
4. Protection of harbor entrances, transient marine activities, reclaimed lands
5. Temporary protection for work areas
6. Extension of existing berths or permanent breakwaters
7. Protection of offshore operations, diving, pipelaying, etc.
8. Use in mariculture industry (fish farms)

constrained by economic impracticality. A notable exception is the floating tire breakwater (FTB), which has proved cost-effective in many installations (Harms, 1979). As the FTB is an important addition to the sphere of shore protection methods, the functional design criteria of this device are highlighted in this section.

This discussion is limited to presentation of the basic characteristics of floating breakwaters. There is a large body of technical literature available on the subject. The bibliography compiled by Griffin and Jones (1974) is a reasonably thorough survey of this material as of 1974.

Structural Types and Wave Attenuation

Transportable breakwaters can be grouped by operational similarities into three categories: rigid structures, flexible structures, and pneumatic and hydraulic breakwaters. The components of a floating breakwater system are shown schematically in Figure 3.9. The structure attenuates incident wave energy by one or more of the following mechanisms (Richey and Nece, 1974; Kamel and Davidson, 1968):

1. Reflection by leading edge of the breakwater
2. Dissipation through turbulence of wave breaking (forced instability of incident waves)
3. Interference with internal orbital wave motions
4. Superpositon of waves generated by breakwater motion with transmitted waves
5. Inelastic deformations of the structure and its moorings

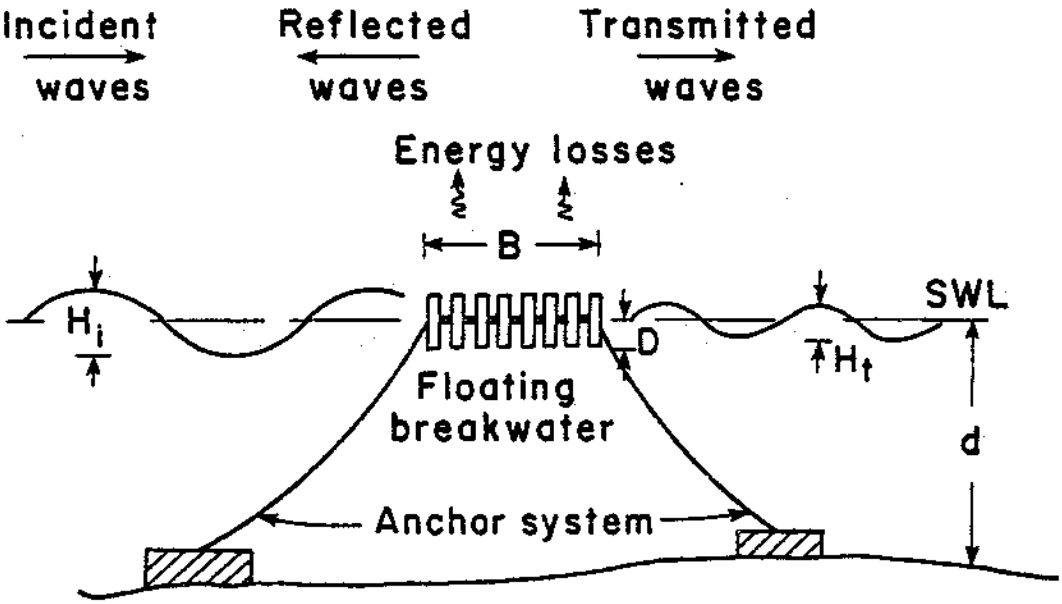


Figure 3.9 Floating Breakwater Terminology (after Richey and Nece, 1972, p. 75)

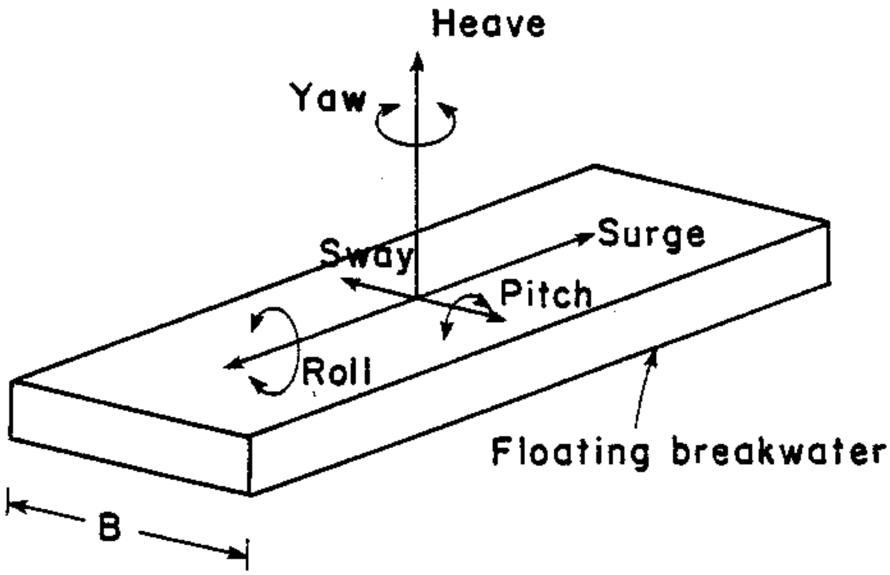


Figure 3.10 Floating Breakwater Modes of Oscillation

The performance of a floating breakwater can be evaluated by the wave transmission coefficient, C_t, given by:

$$C_t = \frac{H_t}{H_i} \tag{3.1}$$

in which H_t = transmitted wave height and H_i = incident wave height (Figure 3.9). Similarly used is the wave suppression efficiency, η:

$$\eta = 1 - C_t \tag{3.2}$$

All analyses must be consistent in the use of one or the other parameter. It should be noted that most contemporary work on floating breakwaters has been confined to the laboratory. Field measurements of prototype performance are scarce.

Rigid Floating Breakwaters. Rigid structures exhibit three modes of vibration caused by the restoring force of gravity: heave, roll and pitch. A moored floating structure has three additional modes of oscillation because of the restoring force of the moorings: surge, sway and yaw. These movements are defined graphically in Figure 3.10. To damp wave action effectively, a floating body must have a natural period sufficiently longer than the incident wave periods. This long natural period can be attained by enclosing a large mass of water within a light structure, such that the restoring force is reduced to a minimum. The same result has been achieved by the use of a large mass moment of inertia in floating breakwater design (Chen and Wiegel, 1970).

The single-prism pontoon system (Figure 3.11a) is the simplest form of rigid structure. Kato, et al. (1969) report on investigations of inverted trapezoidal sections designed by principles of phase differences. When the periods of the floating body and waves were in resonance and rolling was checked by the moors, wave attenuation was

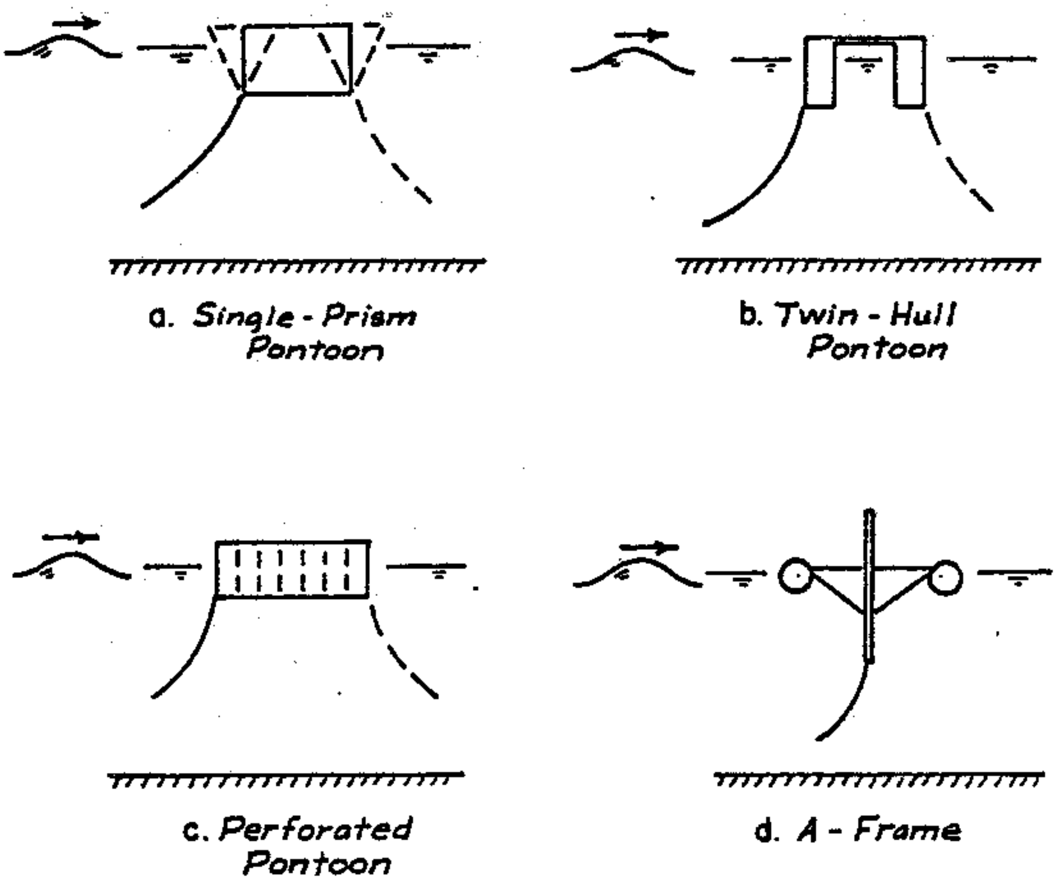


Figure 3.11 Rigid Floating Breakwaters (Richey and Nece, 1972, p. 77)

maximized. The twin-hull or catamaran pontoon (Figure 3.11b) achieves a longer roll period. Perforations in the pontoon (Figure 3.11c) enable additional energy loss and may disrupt wave motion below the structure (Richey and Nece, 1972). A fourth configuration, the A-frame (Figure 3.11d) comprises a central vertical wall and symmetrically located outriggers. The rigid wall serves as a reflecting surface. The outriggers provide stability and a large moment of inertia, which effects a long natural period (Chen and Wiegel, 1970).

Laboratory test results indicate that rigid floating breakwaters are, conceptually, useful for wave energy dissipation. Their practical use is hindered by the lack of a rational and affordable prototype design.

Flexible Floating Breakwaters. One proposed flexible structure geometry is the tethered float breakwater. The model, shown in Figure 3.12, consists of an array of buoyant floats independently tethered about one buoy diameter below the water surface. The floats oscillate in opposition to incident waves and attenuate energy by buoy drag (Seymour and Isaacs, 1974). The associated mathematical model has been substantiated by laboratory and field experiments, reported by Seymour and Hanes (1976). Although the wave attenuation characteristics of the tethered float breakwater have been satisfactorily demonstrated, economic factors are a barrier to widespread use of the device at the present time.

Matlike structures are another type of floating breakwater. Early members of this group comprised thin membranes and fluid-filled bags. These schemes met with little practical success (Richey and Nece, 1974). Conversely, the floating tire breakwater (FTB) appears to be a workable,

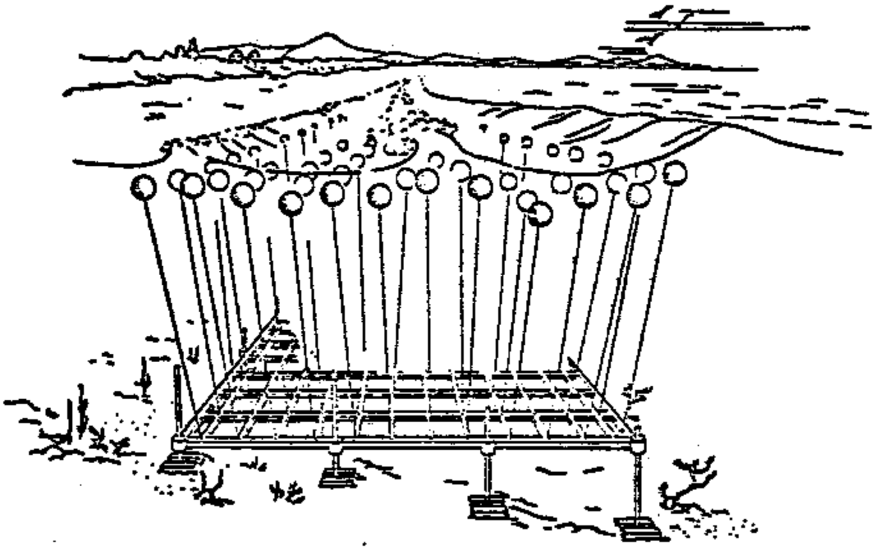


Figure 3.12 Tethered Float Breakwater (Agerton, Savage and Stotz, 1976, p. 2807)

economical addition to this group. FTBs are assemblages of tires bound together and floated at the surface of the water (Figure 3.13). The system is principally an energy dissipator. Incident wave energy is transformed to turbulence in and around the many small elements (tires) of the breakwater and dispersed. Little energy reflection occurs (Harms, 1979). Various FTB configurations have been proposed. Of these, only the Goodyear FTB (Candle, 1974; Kowalski, 1974) is presently backed by the results of numerous laboratory and field investigations. The technology generated has enabled the development of a rational design for this arrangement. Geometric characteristics and design considerations for the Goodyear FTB are explored subsequently.

Pneumatic and Hydraulic Breakwaters. The last group, pneumatic and hydraulic devices, are not strictly in the floating category, but are included here as they are not bottom-supported and are theoretically transportable. The attenuating mechanism is a surface current propagated in opposition to incident waves. The wavelengths are reduced and wave heights increased until instability occurs, and they break over the current or are reflected. In the pneumatic breakwater (Figure 3.14), the interfering current is produced by air bubbles released from a line of jets on the sea bottom (Bulson, 1968). The attenuating current of the hydraulic breakwater is generated by horizontal water jets from a pipe floating at the water surface (Chen and Wiegel, 1970).

A practical limitation on the use of pneumatic breakwaters is that they do not adequately damp long period waves, as demonstrated in a study by Iwagaki, Asano and Honda (1978). The combination of a pneumatic with a submerged breakwater was found to be a more useful system. As waves passed over the submerged structure, a portion of the

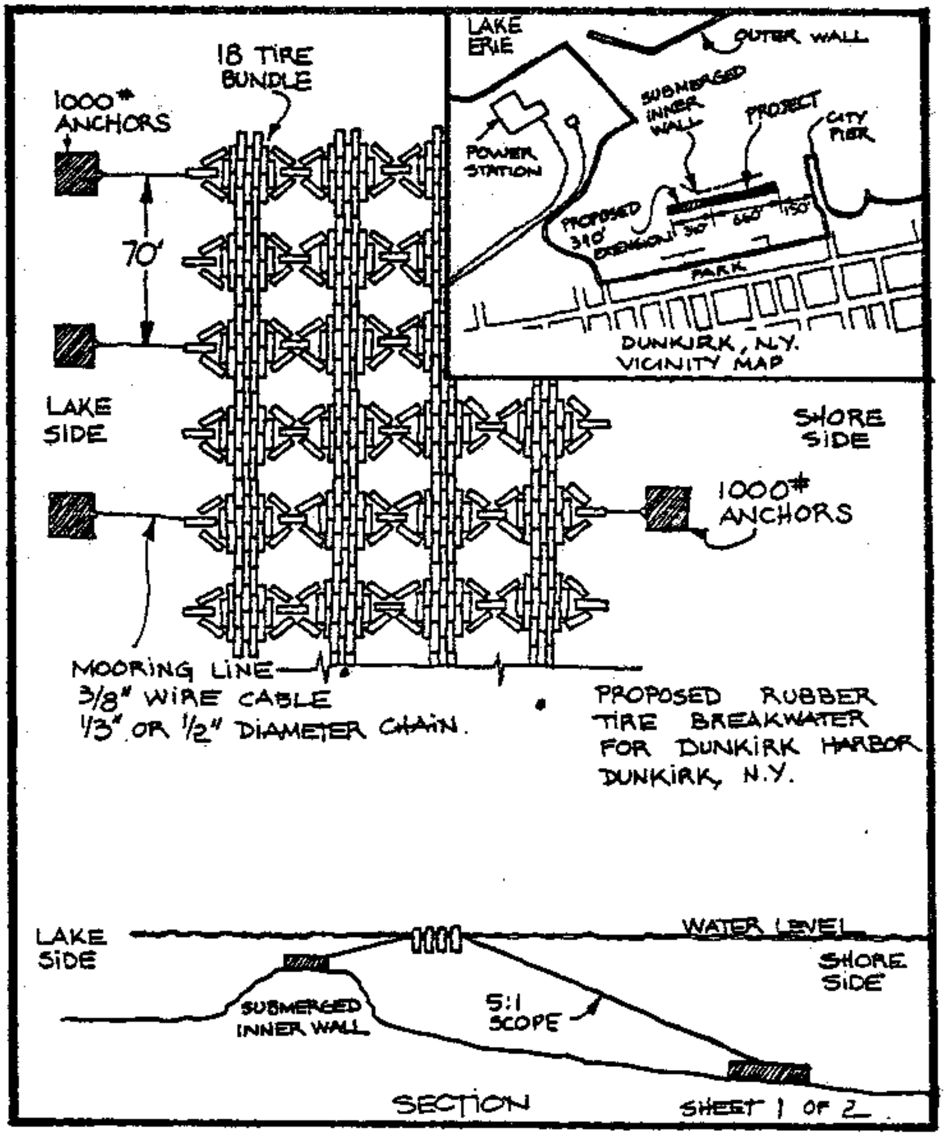


Figure 3.13 Floating Tire Breakwater (DeYoung, 1978, p. 10)

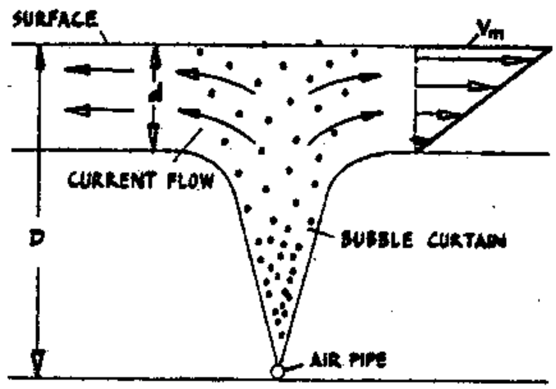


Figure 3.14 Pneumatic Breakwater
(Bulson, 1968, p. 998)

energy transfers to higher frequency waves which can be damped more effectively by the pneumatic breakwater.

Numerous experimental and theoretical studies have made it possible to estimate the air quantity required by a pneumatic breakwater and to design a workable system. However, these devices are characterized by very high operation and installation costs (Bulson, 1968). For this reason, they have not become popular. The same economic constraints apply to the use of hydraulic breakwaters. It has been noted that these systems are effective in eliminating wave reflection phenomena within a marina. Nece, Richey and Rao (1968) proposed that intermittent operation of short hydraulic breakwaters for this purpose might be economically justified.

FTBs - Functional Components

The Goodyear FTB comprises individual 18-tire modules bound together, as shown in Figure 3.13. The completed FTB floats with tires oriented vertically and attenuates wave action as previously described. Basic geometric and functional criteria are considered herein. The treatment of this subject is by no means exhaustive, as the available FTB technology is increasing. Details on each facet of design may be obtained from the references cited in this section. In particular, DeYoung (1978) and Harms (1979) provide comprehensive design and construction procedures and illustrative examples.

Siting. FTBs should be situated parallel to incident wave crests. The distance offshore should be minimized, to avoid regeneration of wind waves in the breakwater lee. FTBs are typically positioned within four wavelengths of the area to be sheltered (DeYoung, 1978). Since the structure is quite mobile, the optimal placement can be found in the field by trial. The orientation can also be varied with seasonal changes in the direction of attacking waves (Kowalski and Ross, 1975).

Geometry. FTB dimensions are defined by the beam (width), B, draft (immersed depth), D, and length (see Figure 3.9). The beam depends on the wavelength, L, of design waves and the transmission coefficient, C_t, specified. These parameters are related by the design curve proposed by Harms (1979) (Figure 3.15).

The Goodyear configuration is one tire in thickness. The draft, D, is related approximately to the tire diameter, D_t, as (Harms, 1979):

$$D = 0.85D_t \tag{3.3}$$

DeYoung (1978) indicates that, for a given wave condition, FTB protection is relatively constant for drafts between 6 and 50 percent of the site water depth, d. The relation D/d is termed the relative draft.

FTB length and conventional breakwater length are determined by site characteristics and wave diffraction and refraction analyses. Generally, FTB length is chosen to exceed the shoreline distance to be protected by about one wavelength (Harms, 1979). As most of the FTB modeling has been done in two dimensions, variations in length have not been extensively studied.

Materials. A distinct advantage of the FTB is the ease of obtaining primary building materials. Scrap tires, which usually present a disposal problem, are readily available from a number of sources.

Special care must be taken in choosing binding materials for the FTB. In some cases, the poor performance of this component has resulted in complete failure of the structure. Environmental demands on binding

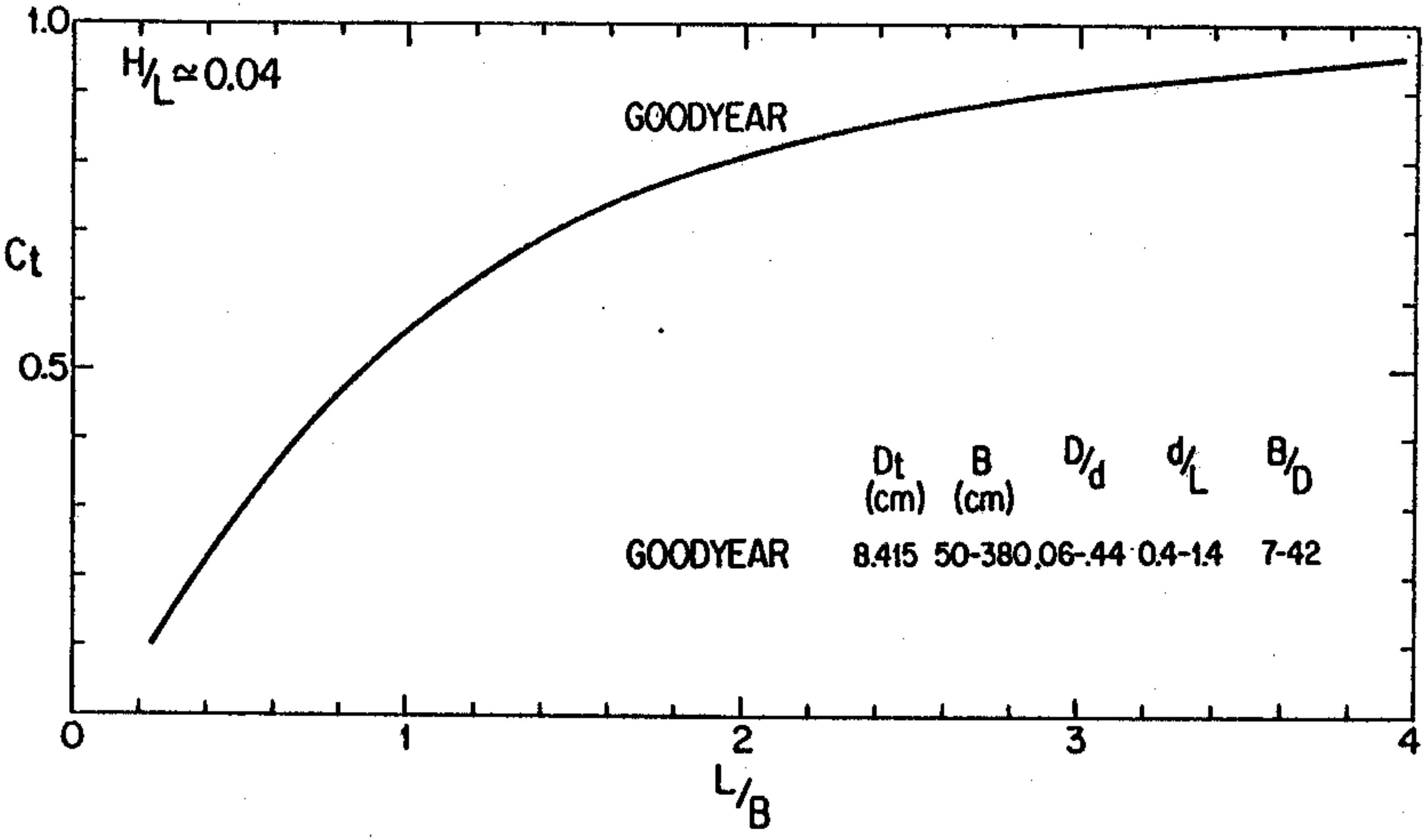


Figure 3.15 Wave Transmission Design Curve for Goodyear FTB (Harms, 1979, p. 27)

materials are severe. They are subject to fatigue by continual flexing, abrasion, galvanic corrosion of metallic parts and ultraviolet degradation of plastic materials. In an effort to assess the performance characteristics of several FTB tying materials, in-situ tests were executed at the University of Rhode Island (Davis, 1977). Of the substances evaluated, the preferred type is rubber conveyor belt edging, 2 to 3 inches (51 to 76 mm) wide, with nylon fasteners. Recommended next is galvanized chain, with a minimum wire diameter of one-half inch (13 mm). The third choice is polypropylene, with an ultraviolet screen to retard deterioration. Various steel wires and fiber ropes that were tested are not recommended for use as FTB tying materials.

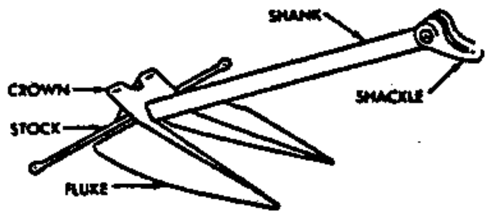
Flotation. Air trapped in the tire crowns provides buoyancy for the assembled modules. Commonly, the units float with about 6 inches (152 mm) of each tire above water. This low profile assures that the breakwater will not be directly affected by wind forces. The low-lying structure will require navigational markings in most areas (Kowalski and Ross, 1975).

Trapped air is replenished when waves impinge on the FTB. Extended periods of calm water might allow the tires to sink. Other causes of sinking include leakage of air from holes, heavy deposits of silt and sand, marine biofouling, and accumulations of snow and ice. Supplemental flotation may be required to counter these negative effects, especially in regions of heavy snowfall and salt water environments where marine fouling is a problem. An effective buoying material is liquid urethane foam, poured into the tires prior to FTB construction. Float materials not recommended include plastic containers, which are difficult to secure, and styrofoam, which is subject to environmental degradation (DeYoung, 1978).

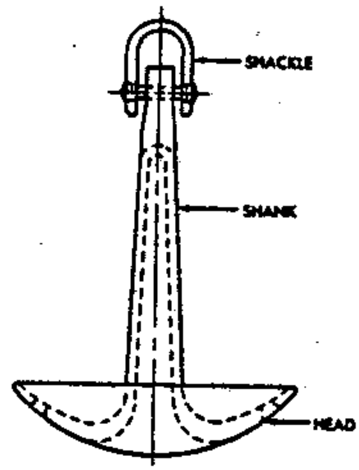
Mooring. FTB mooring forces must be accurately forecast for effective design. Graphical evaluation methods are presented in DeYoung (1978) and Harms (1979). Seaward mooring line tension increases dramatically as wave steepness and structure beam are increased (DeYoung, 1978). Shoreward mooring lines are usually designed to resist 20 percent of the seaward value, assuming no significant waves can approach from the leeward side (Harms, 1979).

Mooring and anchoring system design specifications depend on the bottom material and profile and local currents and tides. Local experience in mooring large craft, over 30 ft (9 m) in length, is a reasonable guide in planning FTB mooring schemes. Mooring line slopes of 6:1 (6 feet or meters of line for each 1 foot or meter of water depth) have proven to be effective. The type of mooring line specified is also an important concern, as its weight acts as an anchor. Chain has typically been used for this purpose (DeYoung, 1978). Danforth, stockless and mushroom anchors, illustrated in Figure 3.16, and concrete blocks heavy enough to resist drag have been used to anchor FTBs. Moorings should be placed at a maximum spacing of 50 ft (15 m) on the seaward side and every 100 ft (30 m) leeward. Distribution of the mooring load over two or more modules is desirable (Ross, 1977).

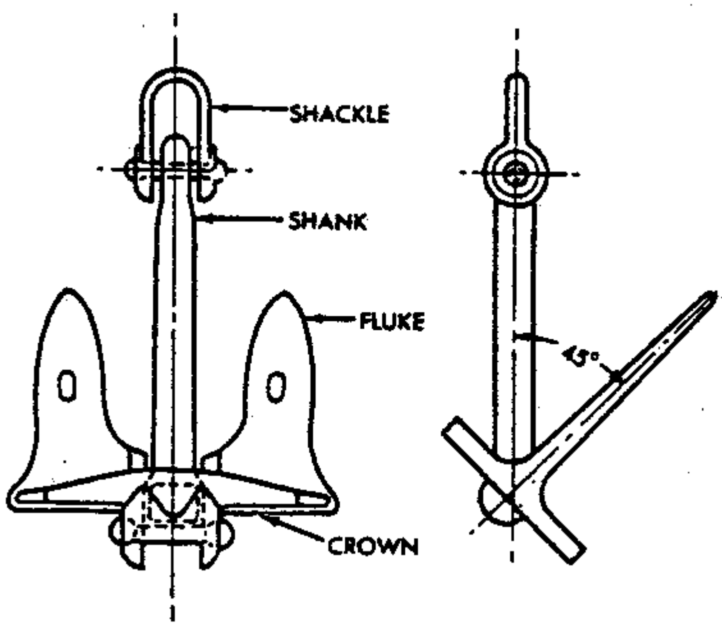
Construction. Each 18-tire module is constructed as shown in Figure 3.17b, in a 3-2-3-2-3-2-3 pattern. A tire rack may be used to arrange the tires. The binding material is woven through the unit as shown. Davis (1977) and DeYoung (1978) should be consulted for details on binding and fastening.



a. Danforth-type Anchor



b. Mushroom Anchor



c. Navy Standard Stockless Anchor

Figure 3.16 Anchor Types (Navfac, 1968, pp. 26-6-11 and 26-6-14)

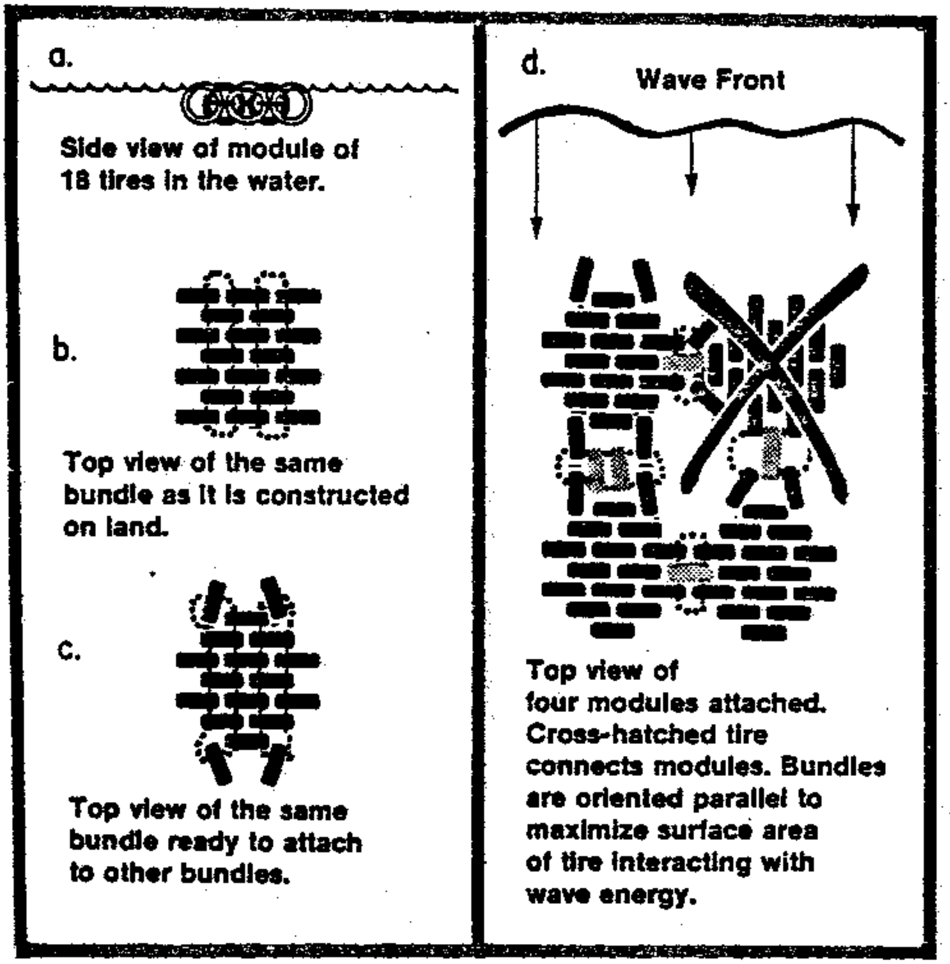


Figure 3.17 Construction of an FTB (DeYoung, 1978, p. 23)

Completed modules are joined as illustrated in Figures 3.17c and d. All tires should parallel the wave crests. The FTB is built in sections on land, and the sections later assembled in the water. The size of sections used depends on the proximity of the assembly area to the water, the difficulty of transporting tire sections on land and the power needed to launch sections of various dimensions.

Towing and mooring an FTB can be accomplished with one towboat with the necessary power and a small boat to carry divers, tools and other supplies. Following is outlined a simple procedure for mooring FTBs, from DeYoung (1978). A windward corner moorage chain is connected to the section and used as a tow line. When the unit is in place, the anchor is dropped (anchor 1 in Figure 3.18a). Anchors 2 and 3 are positioned in turn. Another section is then towed to the location. Anchor 4 is set after the two sections are joined with connecting tires. The process continues to completion, in the order indicated in Figures 3.18b and c.

Maintenance and Other Considerations. An FTB must be inspected regularly to ensure the integrity of the structural components. Damages in the binding materials should be promptly repaired. Periodic underwater examination of the mooring system is desirable as well.

The tires must be cleaned regularly of marine growth and other accumulations, as noted previously. Also, the structure will intercept and collect floating debris. This rubbish must be removed as well.

In icing environments, the FTB must be protected from moving ice floes. Usually, the structure is towed to a sheltered area or removed from the water prior to ice formation (Ross, 1977).

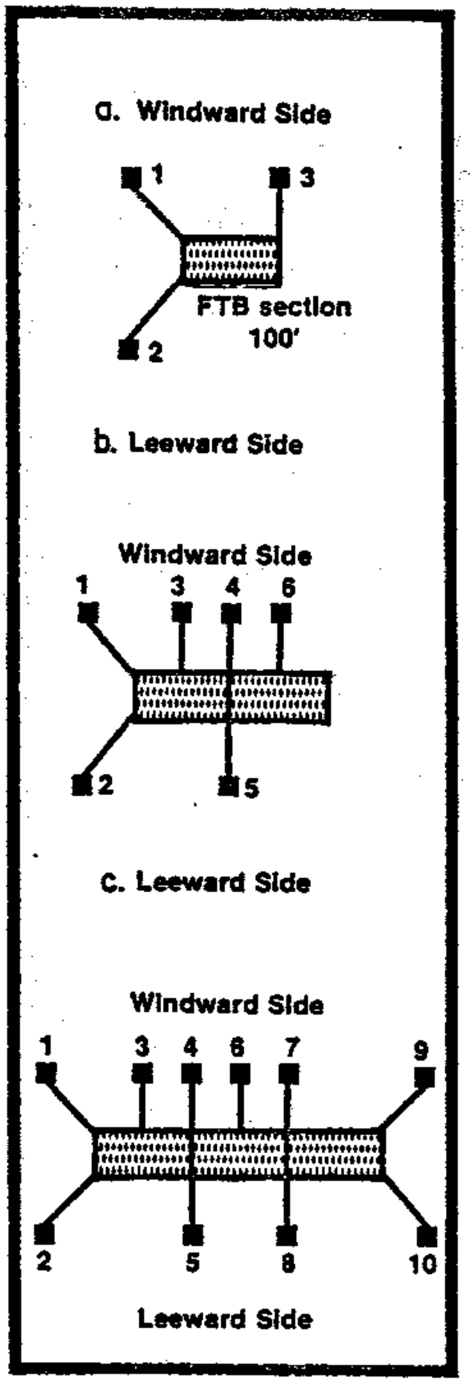


Figure 3.18 FTB Installation
(DeYoung, 1978, p. 24)

The foregoing paragraphs have highlighted the fundamental concerns in the design and installation of FTBs. The cited references should be consulted for additional and more detailed information. In particular, Harms (1979) provides charts and guidelines for Goodyear FTB design. Legal aspects, the ultimate disposal of FTBs, as well as general factors are covered in Ross (1977) and DeYoung (1978). The latter publication also briefly reviews several case histories.

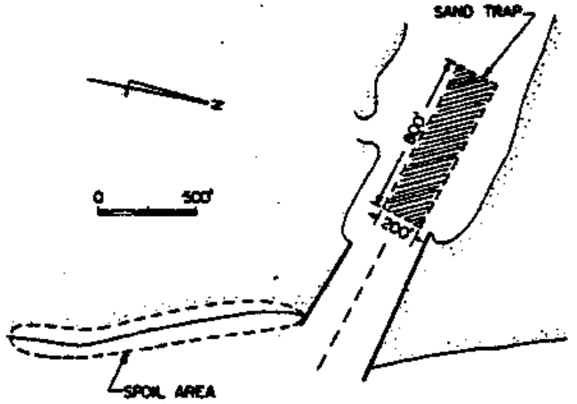
3.3 JETTIES

Inlet entrances provide access from a main body of water to navigable bays, harbors or rivers. The entrance channels, in their natural state of dynamic equilibrium, are often impassable by vessels and therefore not suited to man's purposes. Natural channels may shoal excessively, migrate rapidly and be subject to repeated closure, as described in Chapter 2. Jetties are built at inlet entrances to improve the channels for navigation by reducing or preventing these problems.

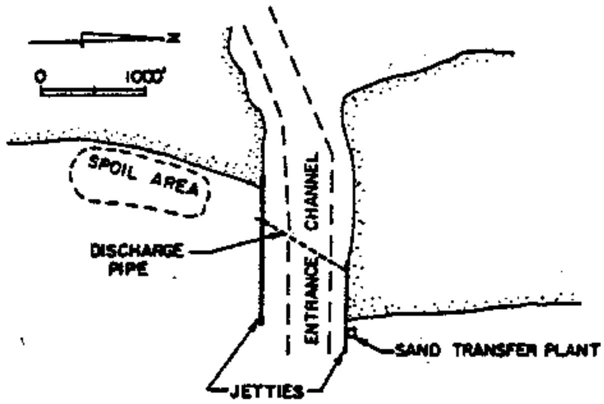
Jetties act as groins to accrete littoral drift on the updrift side of a channel. In this capacity, they prevent shoaling material from entering the inlet. They serve as "training walls" to confine and direct stream or tidal flow into a stabilized, non-migrating channel. Jetties can also be designed to protect the navigable entrance from wave attack.

Jetties are usually installed in pairs. Typical configurations are shown in Figure 3.19. Single jetty protection has been attempted in various cases, but has generally failed to maintain the channels to design specifications.

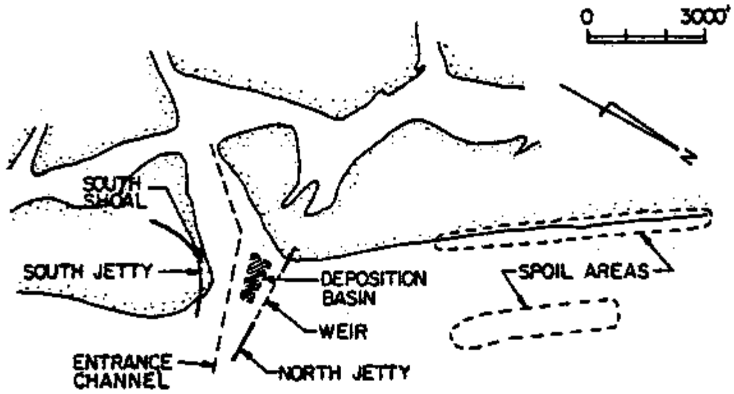
In moderate to severe wave climates, jetties must be massive to



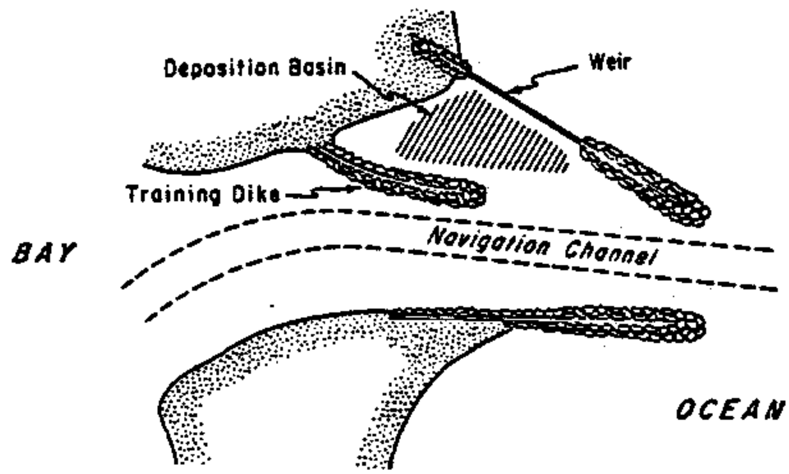
a. Jupiter Inlet, Florida



b. Lake Worth Inlet, Florida



c. Ponce de Leon Inlet, Florida



d. Training dike to control location of navigation channel in a weir jetty system

Figure 3.19 Typical Jetty Pair Configurations (Jones and Mehta, 1977, pp. 54 and 57; Weggel, 1981, p. 24) withstand the attacking forces. Rubble mound or broad-based concrete structures are designed to protect entrances on the open ocean or large lakes. Where waves are not severe, timber, steel and concrete sheet piles have been used successfully in jetty construction. On small lakes, jetties can often be free-standing sheet pile structures (Dunham and Finn, 1974). Details of examples of structural variations are presented in Chapter 4.

The functional design of jetties is considered in the first part of this section. Since the primary purpose of a jetty system is to maintain a fixed navigation channel, navigation requirements and channel design are prime determinants of entrance jetty design. Channel depth and width are governed by the size, type and number of user vessels. Constraints are imposed on the allowable channel cross-section by inlet hydraulics (Weggel, 1981). The reader should refer to CERC (1977) for further information on these and other controlling factors. Methods of harbor planning and channel design are beyond the scope of this work. It is stressed, however, that practical jetty design cannot proceed without an understanding of these topics.

It is necessary that jetties extend through the nearshore region to bar sediment from entering the channel. As a consequence, however, downdrift beaches are deprived of the natural longshore sand supply needed to maintain shoreline equilibrium. The resulting erosion and damages to coastal properties downdrift of a jettied inlet can be devastating and far reaching. In Florida, for example, it is not uncommon to have large stretches of shoreline adjacent to inlets undergoing recession at rates of 10 ft (3.0 m) per year (Walton, 1979). For this reason, all jetty systems should include some plan for artificially bypassing material past the inlet to adjacent shores. Sand bypassing methods are surveyed in the final portion of this section.

Geometrical Components

As previously emphasized, jetty design and placement are wholly dependent on the overall harbor layout and entrance design. Channel width and depth are dictated principally by navigation requirements. The extent to which jetties will be relied upon to maintain this channel design must be evaluated. The ability of tidal currents to flush naturally and preserve the channel (Chapter 2) is an important variable. Similarly, the degree to which it is necessary and practical to bar littoral drift must be assessed, with careful thought to the potential for downdrift erosion. In the following paragraphs, the components of jetty design are reviewed in light of these and other considerations.

Length. Length is the fundamental parameter in jetty planning. Jetties extend at least into the normal breaker zone, the band of longshore movement (Table 2.1). The structures sometimes extend seaward as far as the contour equivalent to the anticipated channel depth (CERC, 1977). Length is often specified with reference to the normal undisturbed distribution of drift in the profile. This distribution is, however, likely to be altered by the presence of the jetties. Modifications must be anticipated and considered in the design phase (Bruun, 1978).

The extreme ends of relatively short jetties are inside the zone of normal uninterrupted drift. Depths seaward of these jetties will have to be maintained either by natural tidal flushing or by dredging. Economic optimization of design requires evaluation of the unit cost (initial and future) per jetty length plus the cost of maintenance dredging for various jetty lengths.

Few benefits are realized by constructing jetties of excessive length. Safety is gained against sudden shoalings, as might occur during severe storms, but properly planned and maintained dredged traps provide a similar measure of security (Bruun, 1978). As jetty length and drift storage capacity are increased, the likelihood for detrimental shore erosion correspondingly escalates. For longer jetties, systems for sand bypassing must be included in planning and in economic analyses.

Alignment. The layout of entrance jetties is planned with regard to the geometry of the navigation channel, inlet and shoals. Because structure costs are minimized in the shallow water over existing shoals, these features usually dictate the most economical alignment. A review of historical data of inlet migration and shoaling patterns will provide useful information on the anticipated behavior of the inlet. The structures should be positioned to take advantage of any beneficial natural processes, provided these will not be altered by construction (Weggel, 1981).

Careful consideration must be given to navigation safety factors. Channel and jetty alignment should be such that small craft entering and traversing the channel are protected from wave action. Ebb tidal currents can cause wave steepening at the seaward ends of jetties. This is a critical consideration, as the steepened waves can break over the ocean bar (Chapter 2), making navigation conditions hazardous (Weggel, 1981).

Familiarity with the site sedimentation characteristics is necessary to evaluate the impounding capacity of a proposed project. Jetties at right angles to the accreted shoreline, approximately parallel to the direction of wave attack, have greater impounding capacity per unit length than structures at acute angles. When the alignment angle is acute, drift can pass around the seaward end of the structure and channel maintenance is necessitated sooner (CERC, 1977).

Height. It is generally not economical to build jetties to a height that prevents overtopping by extreme waves. Jetty crest elevation is chosen to prevent overtopping for some lesser design wave and water level condition. The effects of exceeding design conditions should be investigated. Design is optimized by comparing the initial cost of a higher structure with the higher maintenance and repair costs and decreased benefits of a less substantial jetty (Weggel, 1981).

Since wave energy lessens toward the shore, considerable savings can be realized by progressively reducing the structure cross-section as the water becomes shallower. The leading edge must withstand the full force of waves from any direction. Shoreward, rubble mounds may be decreased in height and size of armor protection, as in Figure 3.20. Similarly, sheet pile structures require progressively less penetration toward the shore (Dunham and Finn, 1974).

The function of weir jetty systems for sand bypassing is discussed later in this section. A low height segment is built into the updrift jetty to form a weir. Sand is transported, by waves and tidal currents, over the segment to a deposition basin and pumped from there to downdrift beaches to halt their recession. A typical plan is shown in Figure 3.21. The low sill is placed seaward of the intersection of the

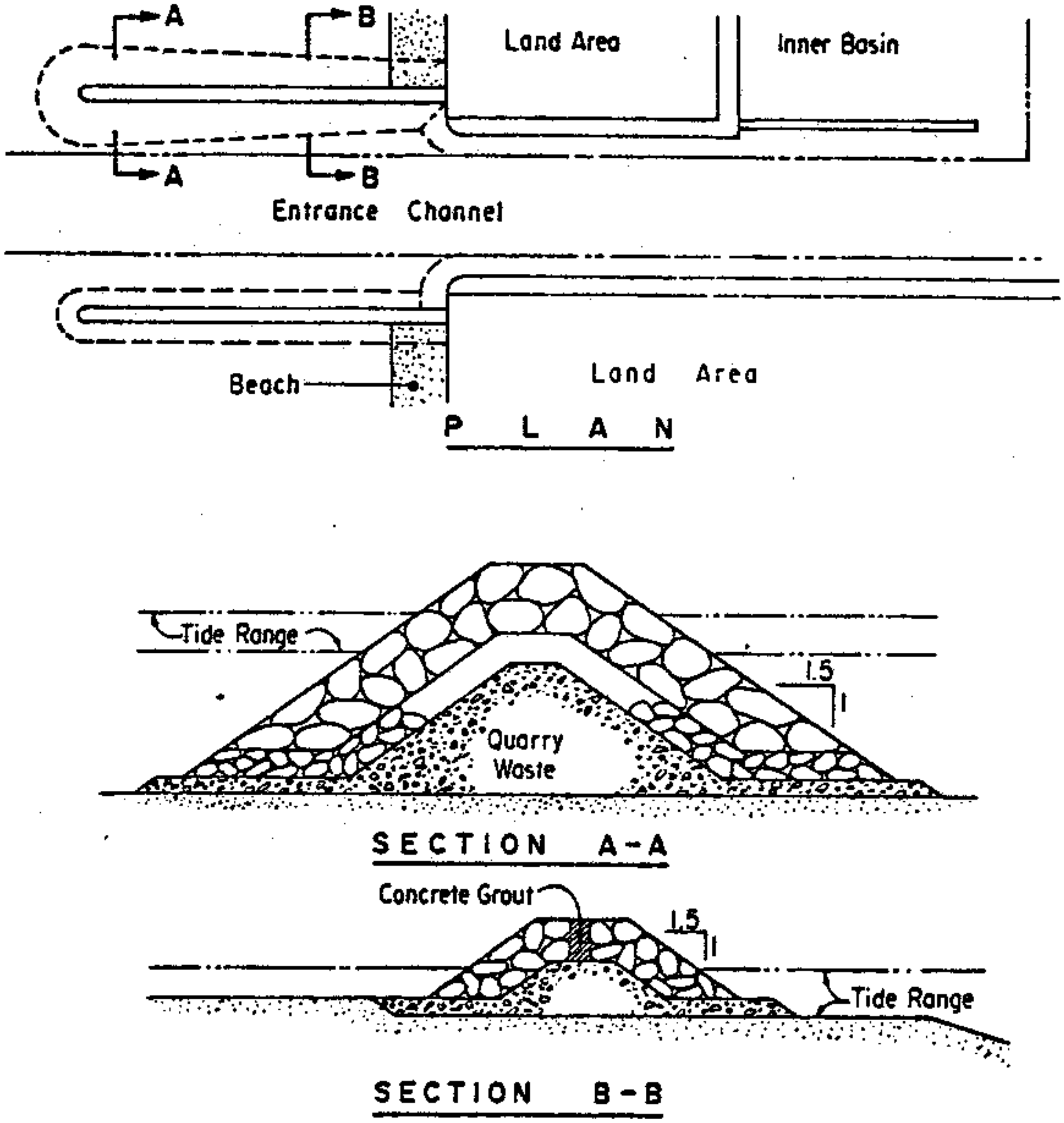


Figure 3.20 Shoreward Decrease in Rubble Mound Jetty Cross-Section (Dunham and Finn, 1974, p. 62)

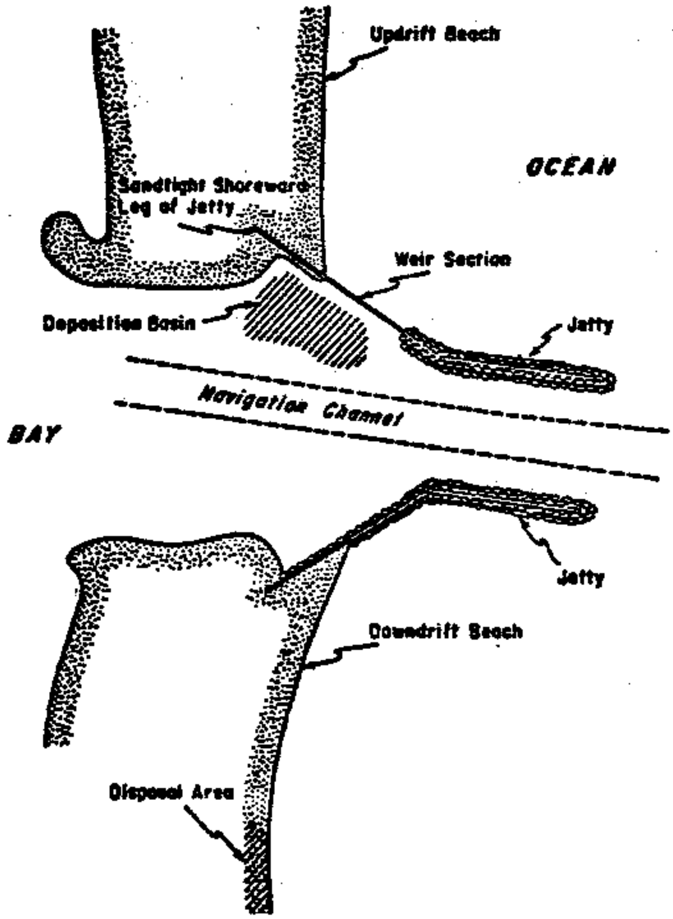


Figure 3.21 Elements of a Typical Weir Jetty System (Weggel, 1981, p. 13)

updrift shoreline with the jetty. Existing weir jetties have weir lengths of 580 to 1800 ft (177 to 549 m).

Weir elevation is a key parameter. To afford wave protection to a dredge operating in the basin the weir crest should be high; however, to control sedimentation the section should be as low as possible. Compromises must be made to achieve an optimum overall design. Generally, the weir crest elevation has been specified as the mean tide level (MTL) in areas with a tidal range of 2 to 5 ft (0.6 to 1.5 m), as the Atlantic coast, and at mean low water (MLW) in low tidal range areas, as the Gulf Coast. Weir jetties are not suitable for coasts with a large tidal range, on the order of 12 to 15 ft (3.7 to 4.6 m), since transport of sand over the weir would be limited to a small part of the tidal period (Weggel, 1981). Details of weir design and hydraulics are reviewed thoroughly by Weggel (1981). Weir jetty inlet improvement case histories are presented by Purpura, et. al. (1977), Jones and Mehta (1977), Magnuson (1965) and Parker (1979).

Permeability. By definition, jetties must be "sand tight" to prohibit littoral drift from passing through them into the inlet. The allowable structure permeability depends on the effects of transmitted waves and shoaling in the channel. At Mission Bay Inlet, California, the permeability of the rubble mound jetties precipitated a major problem. Before 1954, when the jetties were built, it was the local practice to place core rock ("C" rock) up to MLLW and to continue with cap rock only through the wave and tide zone. The large voids (35 to 40 percent) inherent in this placement allowed an excessive amount of sand to pass through the Mission Bay jetties into the entrance channel.

Sealing portions of the jetties with a mixture of concrete and bentonite effectively reduced this objectionable shoaling (Herron, 1972).

Single vs. Paired Jetties. Jetties can be single or double. The response of U.S. tidal inlets to the construction of single jetties is documented by Kieslich and Mason (1975). From a navigation standpoint, entrance channels were not improved by the installation of single updrift jetties. The structures generally resulted in undesirably narrow channels in proximity to the jetties (Figure 3.22). Maintenance dredging was usually required within a few years of construction, followed by the addition of a second jetty at a later date.

Occasionally, a rocky headland or other natural formation can serve as a jetty on one flank of a channel. The first weir jetty system (Figure 3.23) resulted from exploitation of a conveniently placed low reef at Hillsboro Inlet, Florida (Parker, 1979). Similarly, a suitably located offshore reef (Figure 3.24) can provide a natural inlet bypassing system, as on India's east coast (Brunn, 1978). In these special cases, it might be acceptable to build only one jetty.

A jetty is normally required on each flank of the inlet entrance. Spacing of the jetty pair is specified in the overall harbor design with consideration to navigation and hydraulic factors. The size of the tidal prism largely determines the cross-sectional area of the channel. Jetty spacing has an influence on the relative dimensions of the channel, the width to depth ratio. Structures spaced too far apart may allow shoaling, resulting in inadequate water depths. Too close a spacing can result in channel scouring, which can undermine jetty foundations and, in the extreme, cause complete structural failure (Weggel, 1981). Jetty spacing should allow for protective berms on both

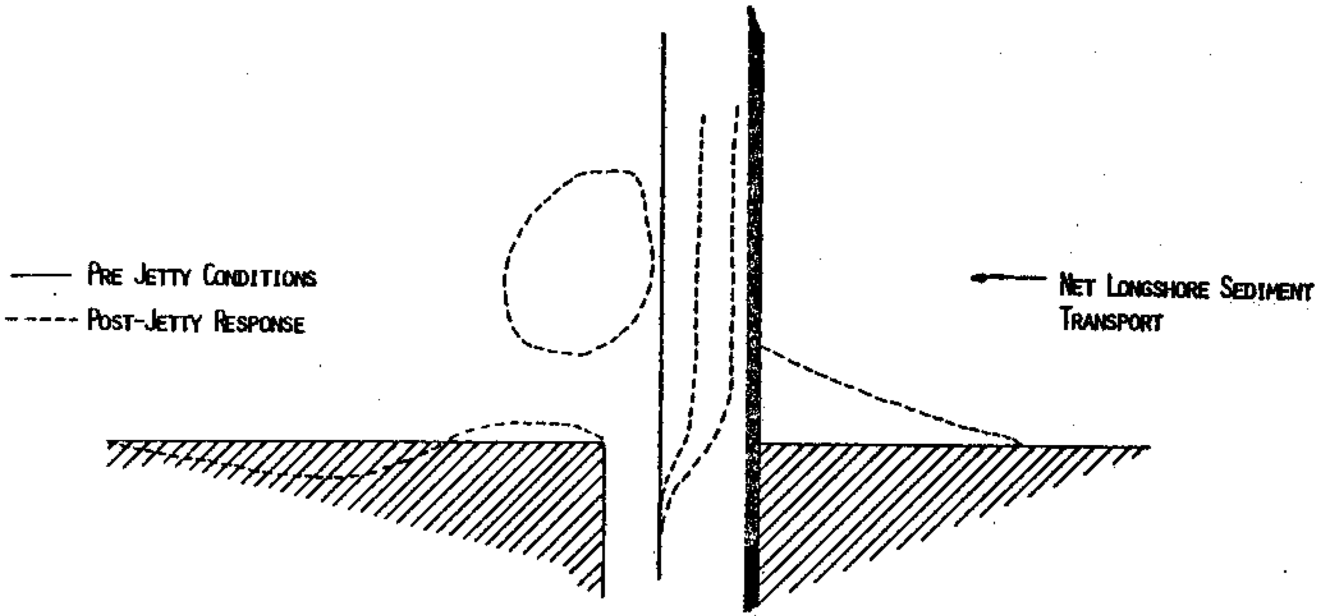


Figure 3.22 Channel Migration Toward a Single Updrift Jetty (Kieslich and Mason, 1975, p. 703)

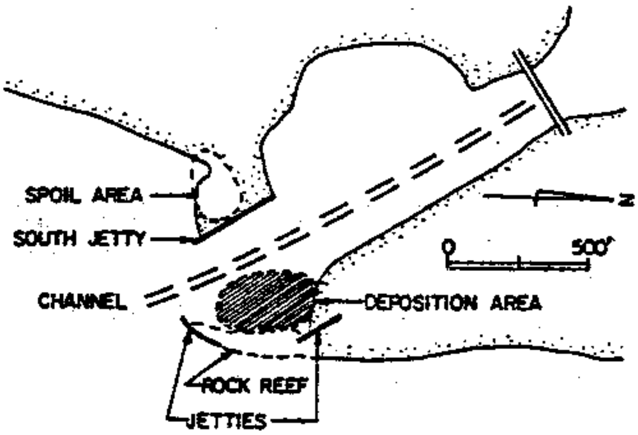


Figure 3.23 Natural Reef in Weir Jetty System, Hillsboro Inlet, Florida (Jones and Mehta, 1977, p. 54)

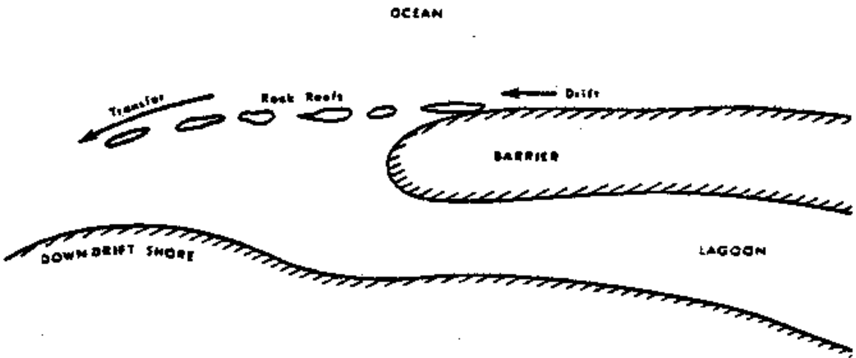


Figure 3.24 Natural Inlet Bypassing on Reef at Sapati, The Arabian Sea, India (Bruun, 1978, p. 428)

sides of the channel (Figure 3.25) to prevent undermining of the structures (Dunham and Finn, 1974).

Erosion and Sand Bypassing

The construction of a partial or complete barrier to littoral drift movement may have a significant and adverse effect on adjacent shorelines. Material that was originally transported alongshore is trapped in the updrift fillet of the structure or diverted offshore. Natural processes for bypassing inlets (Chapter 2) are altered or destroyed. There are numerous cases in which the erosion downdrift of a breakwater or jetty became critical before corrective actions were taken. In some cases, the total cost of remedial measures equalled or exceeded the initial project costs. The negative effects of construction must be anticipated and evaluated from the start. Even in the first phases of design, the development of a satisfactory method to restore and maintain shoreline stability should be a major concern (Watts, 1965).

Sand transfer systems artificially transport material from the updrift to the downdrift side of a jettied inlet. They serve, in this way, to abate downdrift erosion. An equally important function of sand bypassing is to maintain a navigable channel by minimizing shoaling. The major function of a proposed bypass project must be defined at the outset. The system can then be tailored to the site conditions. Where both channel shoaling and downdrift erosion are problems, the quantities and causes of each are often closely related, and one system can be installed to benefit both (Richardson, 1977).

System design factors are presented below in general terms.

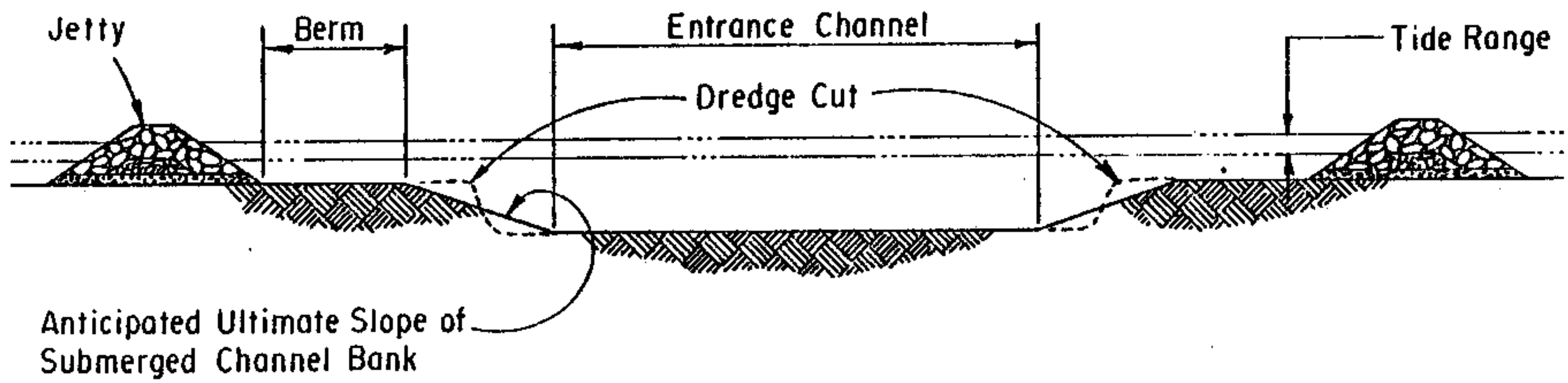


Figure 3.25 Protective Berms in Harbor Entrance Channel (Dunham and Finn, 1974, p. 63)

Reviews of project histories provide the best illustrations of the principles outlined. Reports on a range of projects are included in Richardson (1977), Jones and Mehta (1977), CERC (1977), Weggel (1981) and in the proceedings of the biennial ASCE Coastal Engineering Conferences.

Sand Pickup and Discharge. Sand can be collected from several areas, depending on the harbor configuration, littoral characteristics, and the major purpose of the bypassing system. The various general pickup sites are marked in Figure 3.26. First, consider systems intended primarily to bypass sand across an inlet. For a channel protected by a simple jetty pair, the updrift fillet D is the logical sand pickup location. If the updrift jetty includes a low weir section, the prime sand source is the impoundment basin C. Sand is accreted in two locations at an updrift shore-connected breakwater, areas D and E. When an offshore breakwater assists in entrance protection, areas B and D are logical placement areas for pickup apparatus. Alternatively, if the major purpose of the project is channel maintenance, the sand must be picked up from the channel shoals or along the paths of shoal formation, zones A1, A2 and A3 (Richardson, 1977).

The point of discharge on the downdrift side must be given similar consideration. Tidal currents and littoral drift reversals may tend to move the expelled sand back toward the inlet. The discharge point should be beyond the sphere of influence of the downdrift jetty and littoral forces tending to move the material in an updrift direction (CERC, 1977).

System Types. Bypassing systems can be classified by their on-site mobility, as suggested by Richardson (1977). A system which utilizes a

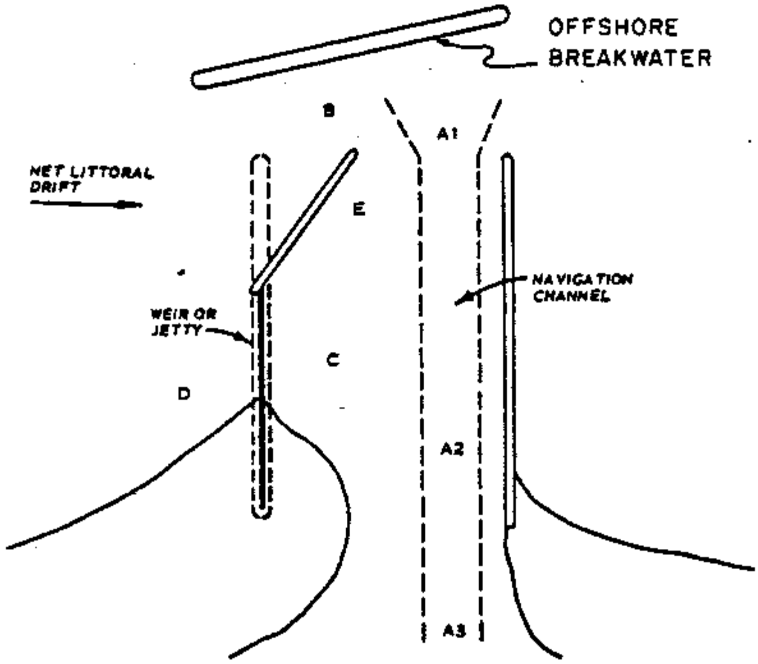


Figure 3.26 Bypassing Systems - Possible Sand Pickup Locations (Richardson, 1977, p. 69)

dredge pump operating from a platform is fixed. A conventional floating dredge is classed as mobile. Semi-mobile systems establish a fixed location for the major pumping apparatus while part of the equipment is mobile.

In most fixed bypass schemes, sand is picked up through a suction pipe and pumped through a discharge line to the outlet point. The dredge pump and other equipment are mounted on a platform or on the updrift jetty (Figure 3.27). Fixed plants are limited in the amount of sand they can intercept. Except in optimum locations, all the sand which is affected by the structures cannot be bypassed with this method. Sand which escapes around the fixed plant may "landlock" the system by a growing accretion fillet. However, operating costs, maintenance and manpower requirements are usually low. Their performance is relatively unaffected by the wave climate, allowing continual operation. They also pose no hazard to navigation, nor vice versa.

Mobile systems commonly comprise a hydraulic pipeline dredge and auxiliary equipment. They are used for channel maintenance, operating from the shoals, and to empty impoundment basins, as areas B, C and E in Figure 3.26. The pipeline dredge is highly susceptible to damage by wave action, and operating costs are relatively high. Their use may interfere with or totally block the navigation channel. The ability of mobile plants to clean large areas at a time is, however, a distinct advantage. A mobile dredge used periodically may intercept more sand than a fixed plant.

A semi-mobile design developed at the Waterways Experiment Station is shown in Figure 3.28. When sunk, the mobile jet pump module excavates a series of craters in the bottom, forming a littoral drift

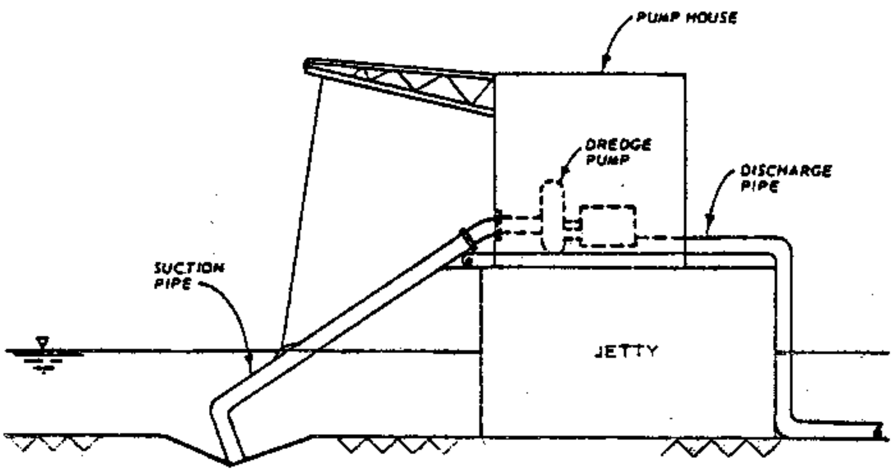


Figure 3.27 Fixed Bypass System - Schematic Layout (Richardson, 1977, p. 71)

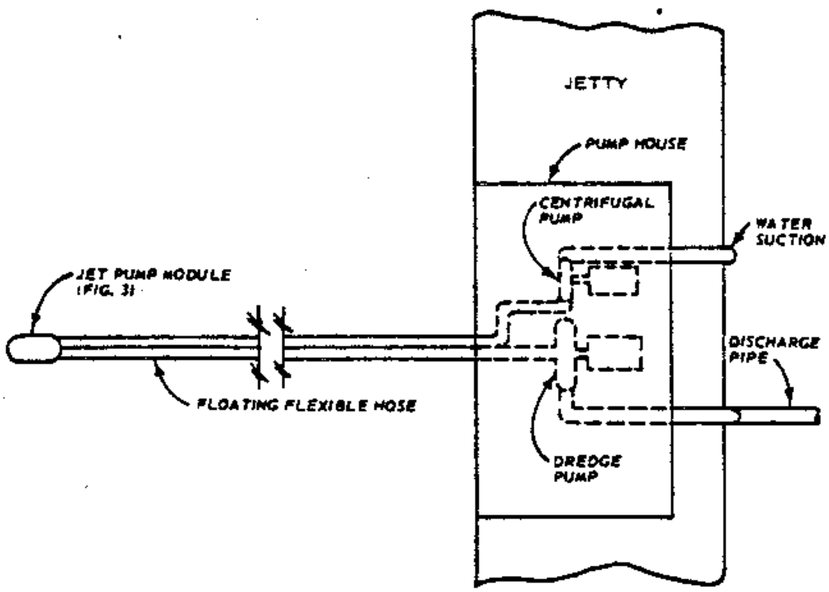


Figure 3.28 Semi-Mobile Bypass System - Schematic Layout (Richardson, 1977, p. 73)

trap. The centrifugal pump, dredge pump and motors are the fixed components of the system. The distance between the jet pump and fixed portion must be limited for efficient operation. The foundation materials must be conducive to excavation by the jet; deposits of cobbles, large shells, peat layers and cemented materials would not suit this purpose. The proposed scheme is usable in exposed waters, as in the accretion wedge. The semi-mobile plant is quite adaptable to both project functions and various inlet configurations (Richardson, 1977).

System Capacity. The required capacity of the bypassing equipment involves many variables. The system may operate regularly or periodically, a function of the sand pickup location. Channel shoal removal is usually done periodically, to minimize interference with navigation. Bypassing from an accretion fillet must generally be continuous. Operation from an impoundment basin, behind a breakwater or weir, can be carried out on either schedule.

For periodic operation, the system capacity depends largely on equipment availability and economics. For continuous plants, the capacity must be sufficiently large to handle site conditions, but small enough to be economical (Richardson, 1977).

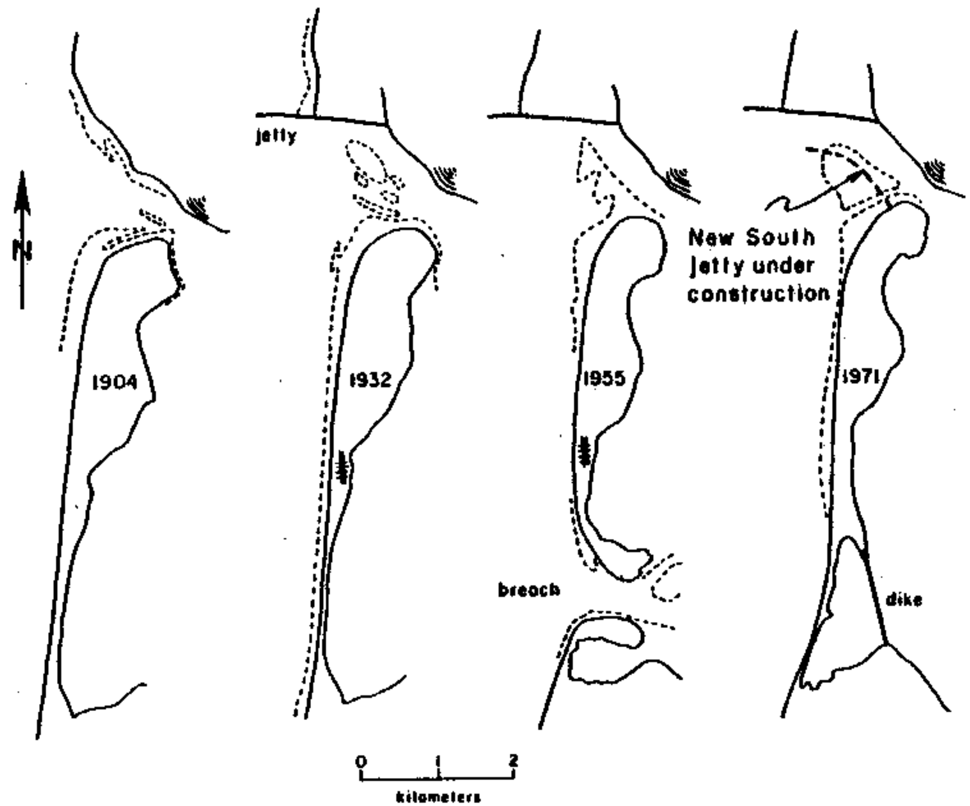
Erosion on Zero Net Drift Shores. It seems reasonable to infer that on coasts with zero net drift, nearshore structures will accrete no material and induce no downshore erosion. This is generally a valid assumption. Jetties which have not caused erosion are, in most cases, on such shores. Occasionally, however, coastal erosion can result from jetty construction even in areas of zero net littoral drift.

The damages sustained at the entrance to Tillamook Bay, Oregon, serve as an example. A single north jetty was constructed at the mouth of the bay and was later extended. The north shoreline advanced in immediate response to the structure (Figure 3.29a) even though the Oregon coast has a long term zero net littoral drift. As shown in Figure 3.29b, a large shoal formed at the inlet mouth, making the entrance nearly impassable. Dune and property erosion on Bayocean Spit, to the south of the entrance, became progressively severe. Homes and other buildings were destroyed. In 1952, the spit was breached, and ocean water and beach sand washed into the bay.

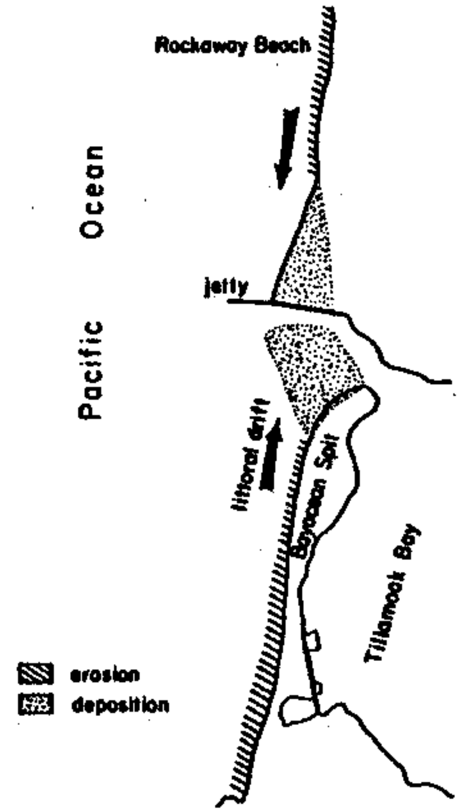
Explanations for the deposition-erosion pattern at Tillamook Bay were proposed by Komar (1976) and Komar and Terich (1976). To maintain an equilibrium configuration, sand moved alongshore to fill the pocket formed between the jetty and pre-jetty shoreline. Material also accumulated in the sheltered areas created by jetty construction. Accretion of these materials caused the corresponding unanticipated recession of the spit and adjacent beaches. Because the length of beach involved was relatively small, the erosion per unit length was especially severe. As demonstrated, jetty construction can radically alter wave refraction patterns and upset the natural shoreline equilibrium, even on zero net drift coasts, with damaging consequences.

3.4 GROINS

Groins originate at the backshore and extend, usually perpendicular to the shore, into the littoral zone to intercept longshore transport, as shown in Figure 3.30. Groins can provide or widen beaches by retaining littoral drift or can stabilize and control erosion of existing shoreline areas by reducing the rate of loss of sand. Terminal



a. Erosion of Bayocean Spit



b. Deposition-Erosion Pattern
around Tillamook Bay

Figure 3.29 Effect of Jetty Construction at Tillamook Bay, Oregon (Komar, 1976, pp. 334-335)

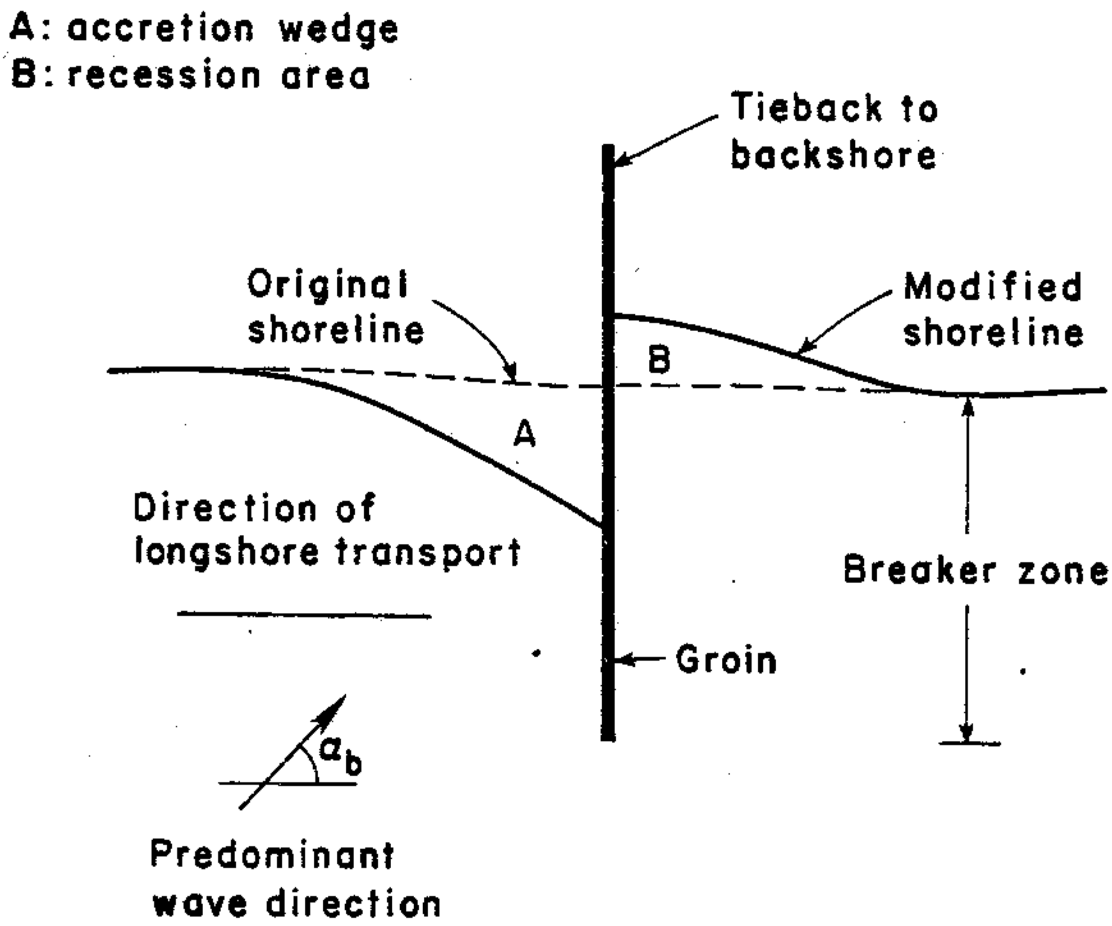


Figure 3.30 Individual Groin Configuration, Plan View

or barrier groins block more completely the longshore movement of sediment and are useful where it is desired to prohibit downdrift accretion.

Groins are the most widely used, most controversial and the least understood of shore protection structures. In many cases, groins have been successful in bringing about the specified shoreline changes with little negative impact. Conversely, many groins have failed or produced no effect. Too often, they have actually worsened conditions at or adjacent to the site. Failed designs result primarily from an inadequate understanding of groin operation (Sanko and Smith, in preparation).

The mechanics of operation, as related to the site littoral processes, are detailed below. Foremost among the natural parameters input to groin design are the magnitude and direction of wave attack, the shape of the beach profile and the intensity of longshore drift. Environmental effects, such as the potential recession of downdrift shores, must be considered as well. These elements vary from site to site on a coast and differ widely for various coastal regions. Even when consideration is limited to a specific location, there are a number of design alternatives available to the planners. There is no one absolute groin design, no panacea which will apply in every case. Rather, recommendations can be made, based on field observations, model test results and largely on past experience and engineering judgment. Functional design guidelines for groins are presented in the final portion of this section.

Mechanics of Operation

Groins disrupt wave motion and break the continuity of longshore transport. As a result, beach material is accreted on their updrift sides. The supply of sand to the downdrift shore is correspondingly reduced and this area undergoes recession. When the sand-holding capacity of a groin is reached, further accretion is not possible and subsequent drift bypasses the groin and resumes its normal course. Figure 3.30 illustrates a plan view of an individual groin with the typical shoreline modification it produces.

This sequence may similarly be traced in its effect on the shore profile (Figure 3.31). The beach slope on the downdrift side of the structure lacks nourishment and becomes progressively flatter. The updrift slope grows steeper as the movement of sand is interrupted and the particles are accreted. The sediment gathered most readily is the coarser fraction of the beach sand in transit, as the finer material remains in turbulent suspension. When the groin is "full", either in surface profile or areal pattern, impoundment ceases. Sand then passes over its top or around its seaward end, or both.

Groins are most often installed in a series, a groin field, to affect a larger segment of the coast. The action of each groin in the group is essentially the same as that of an individual groin. Figure 3.32 demonstrates the typical scalloped shoreline which results from this system. The spacing of individual units in the field and the order of groin construction, both critical factors, are discussed subsequently.

The decrease in the rate of longshore transport caused by groins can be explained in terms of wave energy. As discussed in Chapter 2,

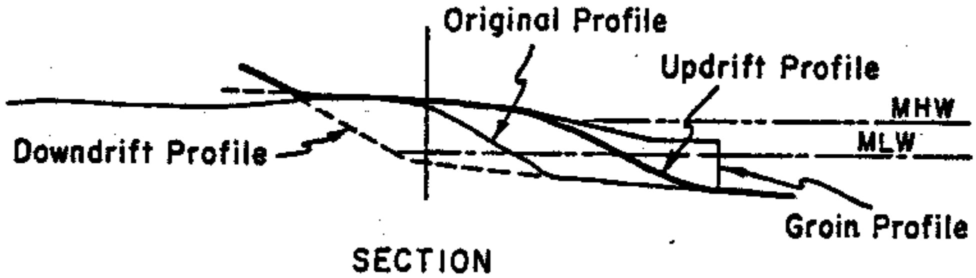


Figure 3.31 Typical Groin Section (CERC, 1977, p. 5-35)

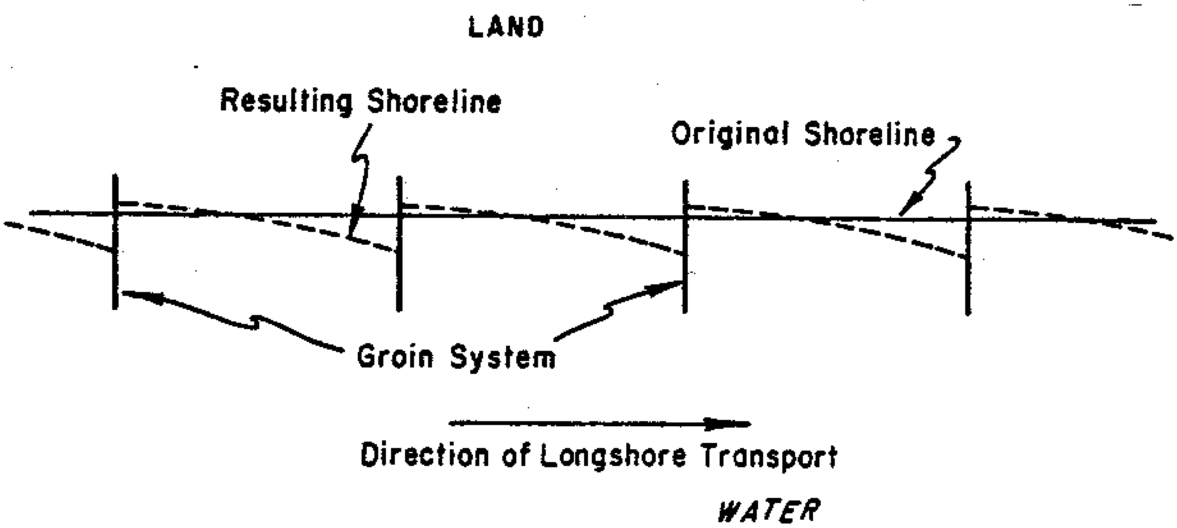


Figure 3.32 Typical Groin Field Shoreline Configuration (CERC, 1977, p. 5-35)

breaking waves transmit energy to the coast, facilitating the transport of sediment within the surf zone. The transport rate is directly proportional to the lateral or alongshore component of wave energy. As the breaker angle, α_b, decreases, the longshore component and therefore the lateral transport of drift are lessened. At the limit when $\alpha_b=0$, wave attack is normal to the shore and the wave energy available for littoral transport is theoretically zero. A well-engineered groin system capitalizes on this principle. The modified shoreline between adjacent groins tends toward a stable alignment normal to the predominant direction of wave attack. Thus, the lateral dissipation of energy and the corresponding rate of transport along the coastline are minimized (Inman and Frautschy, 1965).

There is evidence that sediment accretion is not confined to the region shoreward of the groin end. In a prototype investigation of impermeable groins (Hawley, 1976), accretion extended offshore after initial filling of the system. The accreted profile refracted waves breaking further offshore, thereby reducing wave energy reaching the coast. This process appears to be an additional mechanism which encourages a stable foreshore topography. A similar phenomenon was noted by Allen and Nordstrom (1977) in review of the effect of groins at Sandy Hook, New Jersey. The groins tended to deflect a portion of the drift seaward, causing the growth of longshore bars. The bars act effectively as wave energy filters. Where there is a high volume of littoral drift, longshore bars may build up vertically into small spits, recurve into the beach and, with overwash, prograde downdrift shores. This subaqueous spit growth model is illustrated in Figure 3.33.

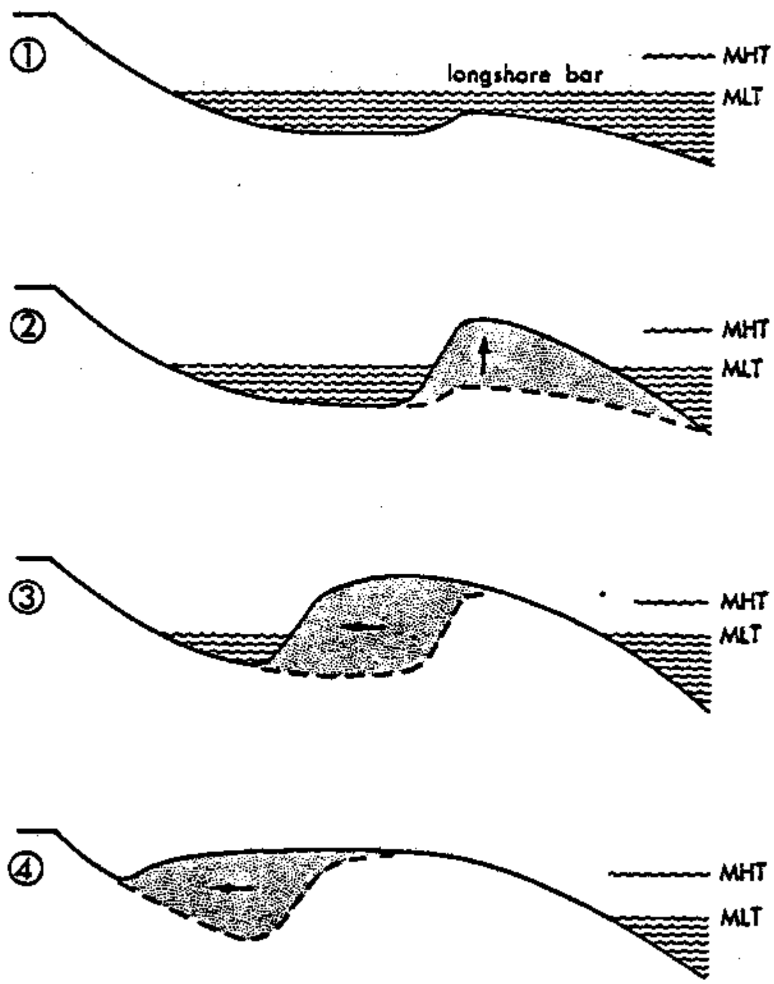


Figure 3.33 Subaqueous Spit Growth Model
(Allen and Nordstrom, 1977, p. 45)

Groins cannot serve their intended function where there is insufficient sediment flow. In environments of comparatively little drift the structures may well be a detriment to the region, causing significant recession of the shoreline. The installation of groins must be restricted to areas of substantial sand movement.

Some recession of downdrift shores may be expected at a successfully operating groin system. However, severe downdrift erosion can quickly result from an improperly designed system. Corrective actions must be planned and enacted to prevent such damage. A commonly recommended and viable remedy is artificial nourishment, a replenishment of the beach material with fill from another location. By placing sand at the updrift end of a groin system, the amount of material transported by longshore currents is increased. While the groins will subsequently entrap a portion of the drift, the sand supply to neighboring beaches is relatively uninterrupted. Nourishment may be carried out periodically, as a maintenance measure, or solely during the construction phase.

Geometrical Components

Despite differences in design and appearance, all groins comprise certain geometrical components. These are length, height, alignment relative to the shore and, for groin fields, spacing. The order of groin field construction should also be planned during the design phase. The state-of-the-art of groin design does not comprise a wealth of theoretical principles; design is based largely on empirical rules and local experience and practice. Design guidelines and recommendations set forth in the literature are summarized below. The typical groin configuration proposed by CERC (1977) is highlighted.

The key and initial criterion in the design of an individual or group of groins is the extent to which it is intended to hinder longshore transport. The measure of sand it is desired to entrap, the minimum beach width, must be quantified. Careful study and judgment must be used in formulating this decision.

As a design aid, a refraction diagram should be plotted for the mean wave condition, that which produces the greatest longshore transport rate. The expected shape and position of the accreted shoreline are evaluated on this schematic as approximately normal to the wave orthogonals (Figure 3.34). Alternatively, and more simply, observations of the shapes of beaches impounded at existing groins with like alignment and wave exposure can be used to approximate the stabilized shore alignment (CERC, 1977).

Quantitative approximations of the rate and volume of drift at the site are also useful. It may be discovered that the transport rate and volume are too low to support the proposed groin field without excessive external nourishment. Such a project might wisely be abandoned after preliminary analyses.

Compromises must be made between the advantages and drawbacks of groin construction. For example, the beach width might be dictated by the requirements for use as a recreational area. The proposed beaches might be designed as smaller than initially planned to lessen downdrift erosion.

Length. Groin length is qualitatively described as long or short, the former entrapping more sand than the latter. Length has also been defined relative to water depth and the distance seaward to the breaking point of plunging breakers (Balsillie and Berg, 1972).

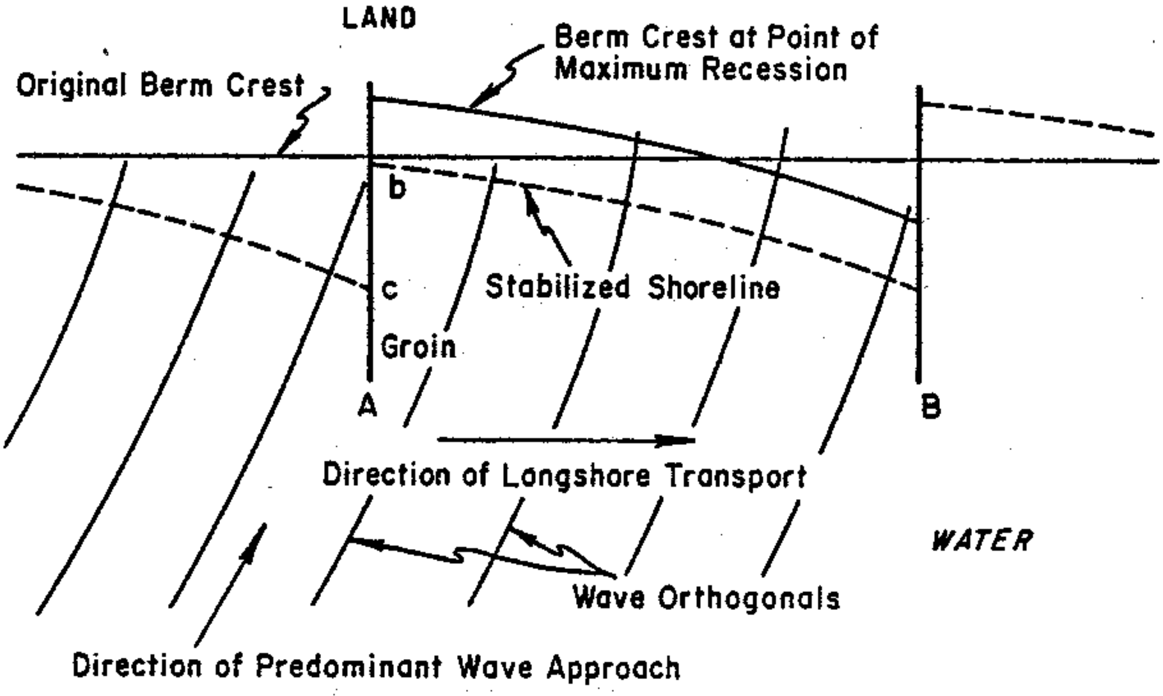


Figure 3.34 Schematic Diagram of Groin Accretion (CERC, 1977, p. 5-37)

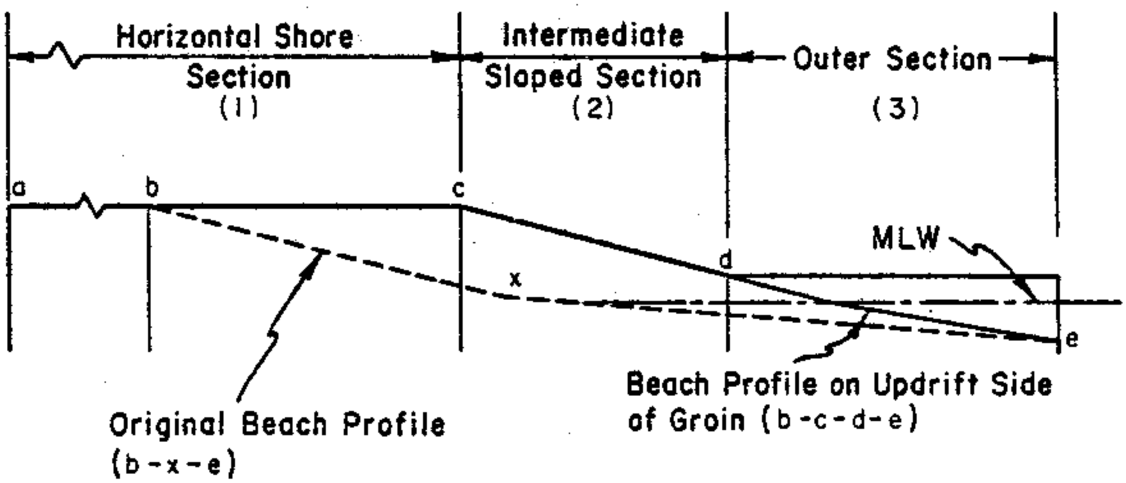


Figure 3.35 Three Section Groin Design (CERC, 1977, p. 5-40)

It is necessary that groins reach seaward into the region of littoral transport to impede the passage of drift. As this band of movement is largely contained within the surf zone, the extension of conventional groins to this zone is sufficient. The extent of normal breaker zones (NBZ) for various coasts are approximated in Table 2.1. The beach profile throughout the NBZ is an influencing factor in the ability of the groin to accumulate sediment. If there are one or more bars relatively close to the shoreline, much of the littoral drift in the area will be concentrated in their immediate vicinity. Most of the drift will be "carried" on bar crests and a smaller portion in the troughs. Groins will trap more material if they span these bars. Similarly, groins terminating just inside the bar will be less effective (Brunn and Manohar, 1963).

Another important factor in choosing the design length is the landward extension of the groin. It is imperative that the structures be tied into a bluff or otherwise keyed into the beach. Flanking and undermining of the landward end of the groin, the possible consequences of omitting this design feature, often result in structural failure and extensive erosion of the beach (Hanson, Perry and Wallace, 1978).

A standard groin design proposed by CERC (1977) comprises three portions (Figure 3.35): horizontal sections landward (1) and seaward (3) joined by a sloped intermediate section (2). The length of each section can be evaluated as follows (CERC, 1977):

1. The length of the shore section, section 1, is represented by the distance ac in Figure 3.35. Distance bc is taken from the sketch of the modified shoreline, Figure 3.34. The length ab is required to key the groin into the backshore, as described previously.

2. The intermediate portion, segment 2, should be angled such that it parallels approximately the foreshore slope developed on the updrift side of the groin. The accreted slope will generally be steeper than the original beach slope, as the sediment likely to be trapped is the coarser fraction of the beach material. These larger particles can naturally sustain steeper slopes. The extension of the seaward end of section 2, at point d, depends on the construction methods used, the degree of drift obstruction desired, or the requirements of swimmers and boaters.

3. The outer section, part 3, comprises all of the groin extending seaward of the intermediate portion. Again, this length depends on the amount of littoral drift to be checked and on local structural practice. The design length should be such that the original beach profile, bxe, and the new updrift profile, line bcde in Figure 3.35, intersect within the toe of the groin, as shown.

Height. Like groin length, groin height is often referred to relatively, as low or high. High structures accrete more material, which creates a greater reduction in the downshore sand supply. Also, scour in the immediate vicinity of a high groin may occur. Scour is caused by overtopping waves which spill over the groin crest and remove material from along the downdrift side, enabling flanking and eventual failure of the member. At low groins, the effect of such scour is much less severe (Balsillie and Berg, 1972). Low groins allow a portion of the littoral drift to pass over the structure, even immediately after construction. The supply of sediment to downdrift shores is not totally curtailed and the recession of these beaches is likely to be less critical. To minimize the potential for erosion, groins should be designed as low as possible.

The height of each groin section in Figure 3.35 may be determined by the criteria recommended by CERC (1977):

1. The minimum height of the shore section is the height of the desired berm, usually the height of maximum high water, plus the normal wave uprush. If it is undesirable to allow sand to overtop the structure and travel to beaches downdrift,

a high groin may be erected. The maximum height of section 1 is then the height of maximum high water plus the height of storm wave uprush. The height of the shore tie-in, ab in Figure 3.35 is usually determined by the anticipated use of the area. On a recreational beach, for example, high shore sections could pose safety hazards and unattractively segment the backshore.

2. The intermediate portion parallels the foreshore slope. The elevation of the lower end of section 2, point d, is contingent on the construction methods utilized and the degree of obstruction the groin is to provide.

3. The horizontal seaward portion should be as low as possible while maintaining useful operation of the structure. Many existing groins are about 2 ft (0.6 m) above mean sea level (Brunn and Manohar, 1963).

The CERC groin design described is a rational model based on extensive experience and is the prototype for many groins constructed in the United States. However, it is not implied that this design need be duplicated exactly or in every case. It is reemphasized that the design methods presented are not rules to be strictly adhered to, but reasonable procedures to be varied and expanded on as judged suitable in each situation. For example, groins erected in Europe often have profiles which are mildly sloped along their entire length to fit more precisely the cross-sectional geometry of the beaches (Brunn and Manohar, 1963).

Groin height and length must be considered concurrently to achieve the optimum design. Various combinations of length and height cause different changes in the shoreline configuration. For example, barrier groins have been used to block completely the passage of littoral drift. These structures must be high to prevent overtopping, of substantial length to prohibit migration of sand around the seaward end, and also be sufficiently impervious. They may be used to trap sediment on the updrift side of an inlet or submarine canyon and serve, in such instances, as artificial headlands (Dunham, 1965). In Table 3.4 the entrapment capabilities of groins of various proportions are presented. The representative values tabulated, considered as conservative, are based on a normal breaker zone extension to the 6 ft (1.8 m) contour, as on the Atlantic coast (CERC, 1977).

Spacing. Correct determination of the distance between groins is essential to the design of a groin series. If the spacing is too considerable, the groins will fail to function as a group and cause excessive erosion. Too small a separation between units is also disadvantageous. In this case, sediment passing the seaward end of the groins will tend to be irretrievably diverted offshore.

Spacing is defined as a function of groin length, as a ratio of length to spacing. For instance, 1:3 signifies a spacing of three times the length, assuming the groins are of one length. In past practice, the common range of ratios has been 1:1 to 1:4. A more recent criteria, recommended by CERC (1977) specifies a spacing of 1:2 to 1:3, where the groin length is measured from the berm crest to the seaward end.

Proper spacing results in accretion fillets which extend from the updrift side of each groin to the base of the adjacent updrift groin (Figure 3.34). Determination of suitable spacing should be based on a study of the site conditions as well as economic factors. When an adequate study cannot be made, it is conservative to space the series too widely rather than too closely. It is easier to add intermediate groins, if necessary, at a later date (Sanko and Smith, in preparation).

The sequence of groin construction must be planned during the design phase. A scheme which will minimize downdrift erosion should be recommended. Artificial nourishment is often used to fill the new

Table 3.4 Entrapment Capacities of Groins (after CERC, 1977, p. 5-41)

Height	Length-Depth of Extensions Below MLLW		Percent of Total Longshore Transport Interrupted*
	Feet	Meters	
High	≥10	≥3	100
High	4-10	1.2-3	75
Low	>10	>3	75
High	0-4	0-1.2	50
Low	<10	<3	50

*Includes the drift accreted by the groins as well as that portion diverted offshore or otherwise prevented from reaching downdrift shores.

groins to capacity, so that the littoral drift is not trapped for this purpose. The fill is usually placed most economically in a continuous operation. All groins should be constructed simultaneously so that the final fill may be deposited as soon as possible.

When artificial nourishment is not used, the natural processes must fill the groins. Each groin can commence filling only when its updrift neighbor is filled to capacity. Filling of the entire field can take so long that severe downshore erosion is caused. To avoid this problem, groins should be constructed one at a time, beginning with the groin at the downdrift end. Only when this groin is full and the shoreline has stabilized can the next updrift groin be built. This construction sequence reduces damages and also provides field verification of design spacing (CERC, 1977).

Alignment. The orientation of groins relative to the coastline has been the subject of some study. If the depth to which the groin must reach was the only determining factor in groin design, a savings in cost could be realized by the sole use of groins normal to the shoreline. However, alignment specifications are also contingent on the angle of wave incidence. It has been proposed that when the angle of wave approach is constant, groins aligned slightly updrift from the perpendicular have an increased impounding capacity (Balsillie and Berg, 1972). When the shore alignment will change considerably after construction, the groins can be placed such that they will be normal to the anticipated shoreline rather than the existing beach (CERC, 1977). In regions with variable wave conditions, construction normal to the coast is advised (Balsillie and Berg, 1972).

Although the majority of groins are linear in plan, variations have been introduced at numerous installations. Special configurations include curved, angled, Z-, L- and T- groins (Figure 3.36). Most of these alterations are devised in an attempt to mitigate downshore recession. While certain of these features do effect some accretion on the immediate downdrift side, erosion of the beach further downdrift is often unchanged. Structures with usual geometries may also be subject to increased scour and are generally more costly to erect than straight groins (CERC, 1977). Because of the many variables involved and the somewhat limited data base, it is difficult to predict accurately the effect of these devices on the coastal zone.

Structural Variations

The basic layout of a groin is defined through quantification of its geometrical components. The structure is further described by its characteristics of permeability and adjustability, dealt with in the following paragraphs. Finally, a groin is individualized by the choice of construction materials. In many cases, the selection of economical, available materials is made prior to or concurrent with other design decisions, thereby governing the resulting design. In all cases, consideration of feasible materials and fabrication methods must be an integral part of the design phase. The various materials used in groin construction are surveyed in Chapter 4.

Permeability. In the same manner as a low groin invites overtopping by littoral drift and a short groin allows sediment to circumvent its seaward end, a permeable groin permits deposition in its lee. Permeability, in the form of apertures in the structure, enables

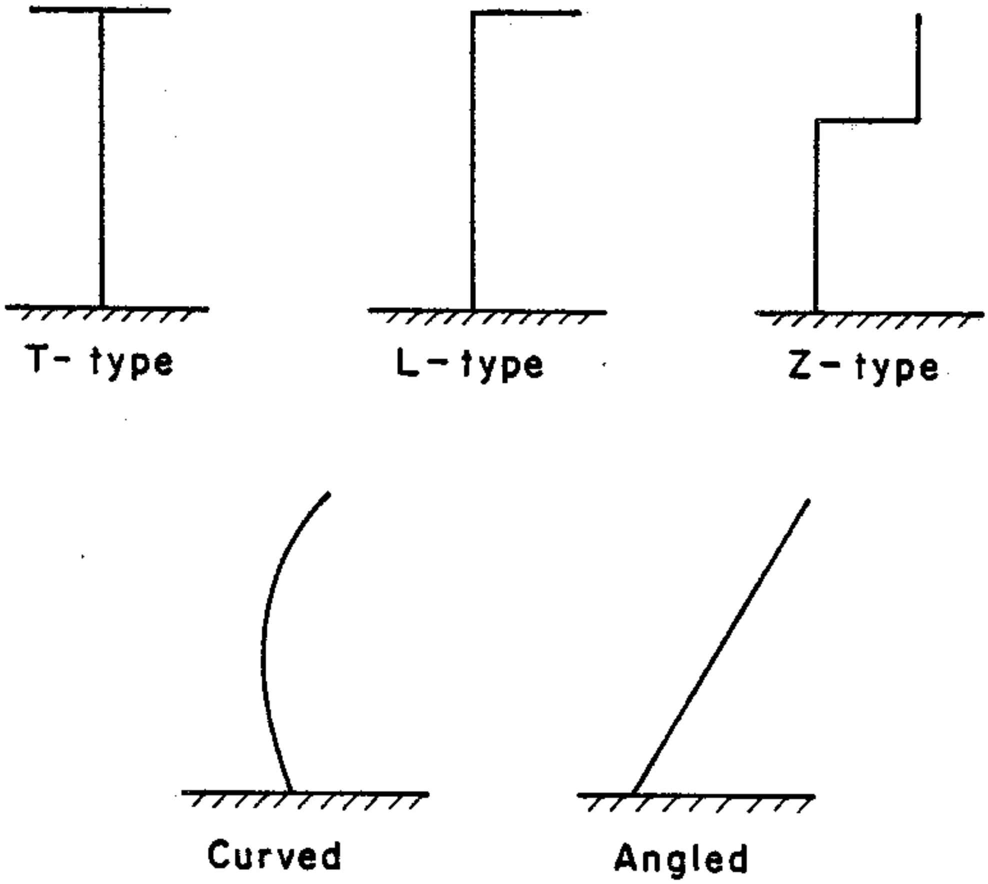


Figure 3.36 Groin Plan Forms (after Horikawa, 1978, p. 330)

the transmission of wave energy and appreciable drift through the groin. The flow of material alongshore, then, is reasonably continuous despite the presence of the groin and the threat of downshore recession is minimized. Nearshore topography is altered less radically. The sharp transition, or scalloping, which is likely to occur otherwise in the immediate vicinity of the groin is also lessened.

The present state of knowledge is insufficient to present rules regarding the design of permeable groins, although study in this area is ongoing. Most investigators recommend that these structures be used only in groin fields and not as individual groins. Further, the advantages of permeable groins can usually be duplicated by appropriately planned low, impermeable groins (Balsillie and Berg, 1972).

Adjustability. Changeable groin height is a design feature which has been tested to a limited extent, in Florida and England. Groins of this type may be heightened as beach accretion progresses or lowered if it becomes necessary to bypass sediment to downdrift areas. Proponents of this plan emphasize that groin height can be reevaluated and altered whenever necessary, in accordance with the actual modification and development of the coast. Also, as adjustable groins can be maintained at 1 or 2 ft (0.3 to 0.6 m) above the beach level, they present less of a barrier to beach traffic and recreational activities than do permanent structures, which often segment the backshore region (Brunn and Manohar, 1963).

Many systems of adjustable groins have been built on the lower east coast and gulf coast of Florida. One such field, constructed at Madeira Beach in 1957, comprises 37 groins spaced at 300 ft (91 m) intervals.

Each 210 ft (64 m) long groin consists of 21 king piles on 10 ft (3 m) centers. Concrete barrier slabs, 18 inches (0.4 m) in width, fit into 4 inch (101 mm) slots in the piles. This typical adjustable groin configuration is shown in Figure 3.37. The design specifications provided that no more than one slab width above the beach be added at any time. This procedure has been followed, and the system has functioned reasonably well (Eldred, 1976).

Underscour of the slabs may necessitate rearrangement and lowering of the boards. The top slabs must be held stationary by locks or wedges (Brunn and Manohar, 1963). The major problem with the system is that movement of the king piles renders impossible any subsequent adjustment or addition of slabs. A secondary cause of failure is deterioration of the panels themselves (Jones, 1980).

3.5 SUMMARY

Breakwaters, jetties and groins are process alteration structures. They are similar in that they all impede the flow of littoral drift and attenuate wave energy. The precise purpose and the degree to which they upset the natural equilibrium of shore processes is different for each structure. Functional design characteristics are combined in each case to devise a structure unique to the purpose and site under consideration.

Bottom-resting breakwaters can provide shore or harbor protection, or both. They are most effective in providing an area of calm water on their leeward sides. Accretion of littoral material in the "wave shadow" is a secondary result. Breakwaters can be shore-connected or offshore structures depending on the objectives of the project.

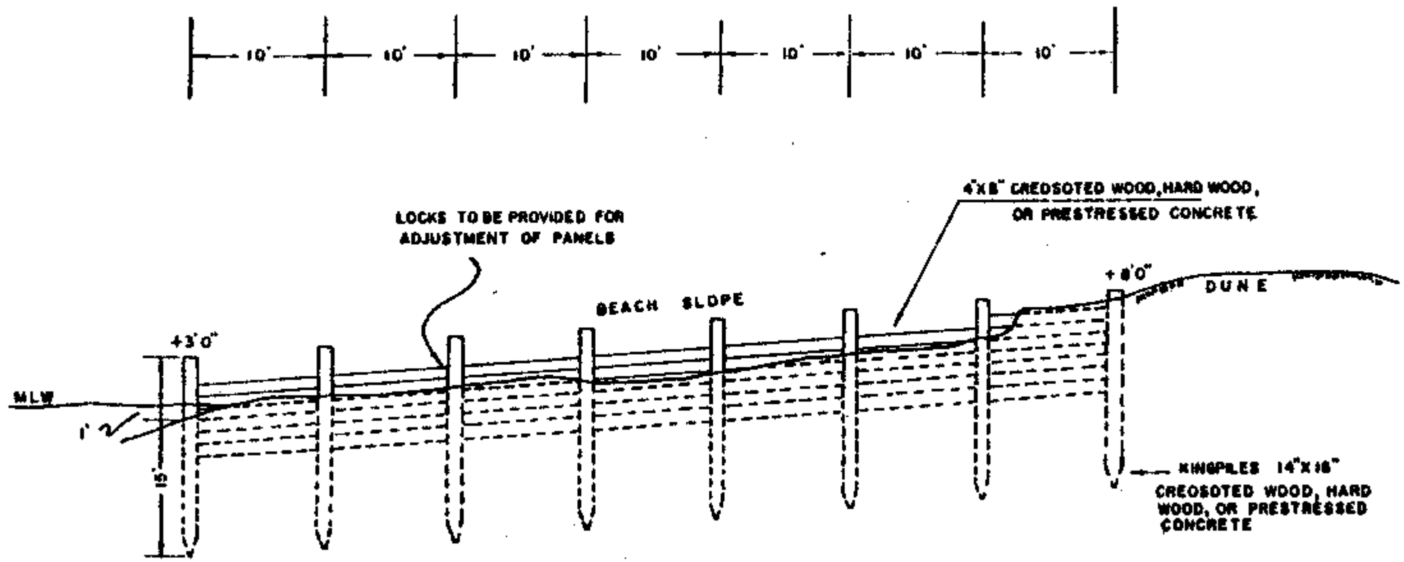


Figure 3.37 Adjustable King Pile Groin (Bruun and Manohar, 1963, p. 14)

Structural type also influences the mechanisms of breakwater operation. Siting is a complex design process, facilitated by diffraction and refraction analyses. Length, offshore distance, height and alignment must be properly specified in this phase to optimize breakwater functional design.

Floating breakwaters, for harbor protection purposes, belong to the same functional group as bottom-supported breakwaters. However, they are operationally and structurally quite different from the conventional structures. Floating breakwaters can attenuate waves by a number of mechanisms; the precise combination of these depends on the structural behavior, i.e., whether rigid or flexible. Many floating breakwater designs have been proposed, but most are economically impractical. The Goodyear floating tire breakwater (FTB) is a noteworthy exception which has proved to be a feasible means of shore protection. Salient FTB design characteristics are presented in Section 3.2.

Jetties protect inlet entrances from shoaling and migrating and attenuate excessive wave action. Navigation requirements and the hydraulic stability of the channel are prime determinants of entrance jetty layout. Functional components of jetty siting include length, height, alignment, spacing and permeability. Jetty construction and operation can totally destroy the natural balance of littoral and inlet processes and initiate severe downdrift erosion. Sand transfer systems are installed to abate damaging effects and must be included in jetty project planning.

Groins are intended to protect or stabilize shorelines. These are the most unpopular of the shore protection structures as their use has in many cases caused major downdrift erosion. Their operation is understood conceptually, but quantitative design rules are lacking. Groin length, height and spacing must be carefully devised, using the recommendations presented in Section 3.4, to minimize negative impacts. When groin construction is not clearly a viable protection mode, the project should be reevaluated and redesigned, or abandoned.

CHAPTER 4

STRUCTURAL VARIATIONS

Breakwaters, groins and jetties are different in purpose, size, orientation and exposure to waves and other environmental forces. They all act, in some degree, to reduce wave forces and bar littoral drift in the nearshore zone. Because they share this general function and milieu, they also share structural configurations. The two conventional structural groups are the mound and wall types of shore stabilization devices. A third category, low cost shore protection, reflects the recent trend toward developing protection alternatives which are economically feasible for private landowners. Subsets within each classification are identified more commonly by their material components, as rubble mound and steel sheet pile wall. Common structural methods are described in this chapter. General comments on design principles and illustrations of various devices provide a fuller understanding.

The behavior and performance of coastal construction materials is discussed by Hubbell and Kulhawy (1979a). Established materials, such as steel, concrete and wood, and some of the newer choices, as gabions and synthetic fabrics, are covered in this work, so no attempt will be made to repeat this information. Rock was not considered in that study; since it is the main material used for construction of breakwaters, groins and jetties, it will be dealt with herein. The durability and availability of rock are described in Chapter 6.

The purpose and scale of the proposed project has a major impact on selection of structural type. Larger-scale structures, as jetties and breakwaters associated with major harbors, are founded in deeper waters and are subject to more complex and severe environmental loadings. Consequently, they must be massive structures and generally are of conventional design, such as rubble mounds or cellular sheet pile walls. Smaller-scale, shallow water structures, including inshore breakwaters, small lake jetties and groins, are suited to a wider range of materials and structural configurations. These may be adaptations of large-scale methods, such as rubble mounds, or examples of innovative, less tested designs, as the low cost devices. Other factors to consider in material selection are discussed in Hubbell and Kulhawy (1979a).

The emphasis of this study is on the engineering of smaller-scale shore stabilization structures. The design of rubble mounds is presented in Chapter 7. Wall structure design procedures are described by Saczynski and Kulhawy (in preparation). Some variations, notably cellular sheet pile walls and concrete caissons, are typically used in the larger installations. The general design considerations set forth in Chapter 5 apply to these, but presentation of precise technical design procedures is outside the scope of this work because they require detailed engineering studies and design.

4.1 MOUND STRUCTURES

Nearshore structures are often formed by dumping or placing construction materials on the seabed in a mound shape. Mounds are gravity structures which depend for their stability on their own weight and massiveness rather than on foundation preparation. They effectively attenuate wave energy through runup on their sloped faces and dissipation within the voids of their rough surfaces.

Rubble mounds, described below, are the most familiar members of this group. There is a large body of knowledge concerned solely with the design and behavior of rubble mound structures. Stepped face gabion mounds are a relatively recent variation on the standard rubble mound. Any material components which can interlock and maintain a stable mound theoretically can be used for mound construction.

Rubble Mounds

By far, the most common structural configuration of breakwaters, jetties and groins is the rubble mound, composed of layers of natural quarried rock. The three general zones of a rubble mound profile are illustrated in Figures 4.1 and 4.2. The core of small rock, referred to as quarry-run or quarry waste, generally comprises more than 50 percent and up to 80 percent by volume of the rubble mound (Fookes and Poole, 1981). One or more intermediate layers, termed underlayers or filter courses, overlay the core. These layers are graded according to filter design principles to prevent erosion and loss of core material. The primary cover or armor layer ultimately shields and stabilizes the mound with large rock or concrete armor units. Although there may be variations in practice, such as the elimination of underlayers or the omission of core material in an all-armor rock mound, conventional design of larger rubble mounds includes all three zones (Quinn, 1972).

The structural integrity of a rubble mound is highly dependent on the weight and shape of armor rocks which envelope the mound. The armor unit weight required varies directly with structure side slope, i.e., steeper slopes require heavier rock. The relationship of other contributing parameters and the precise determination of design

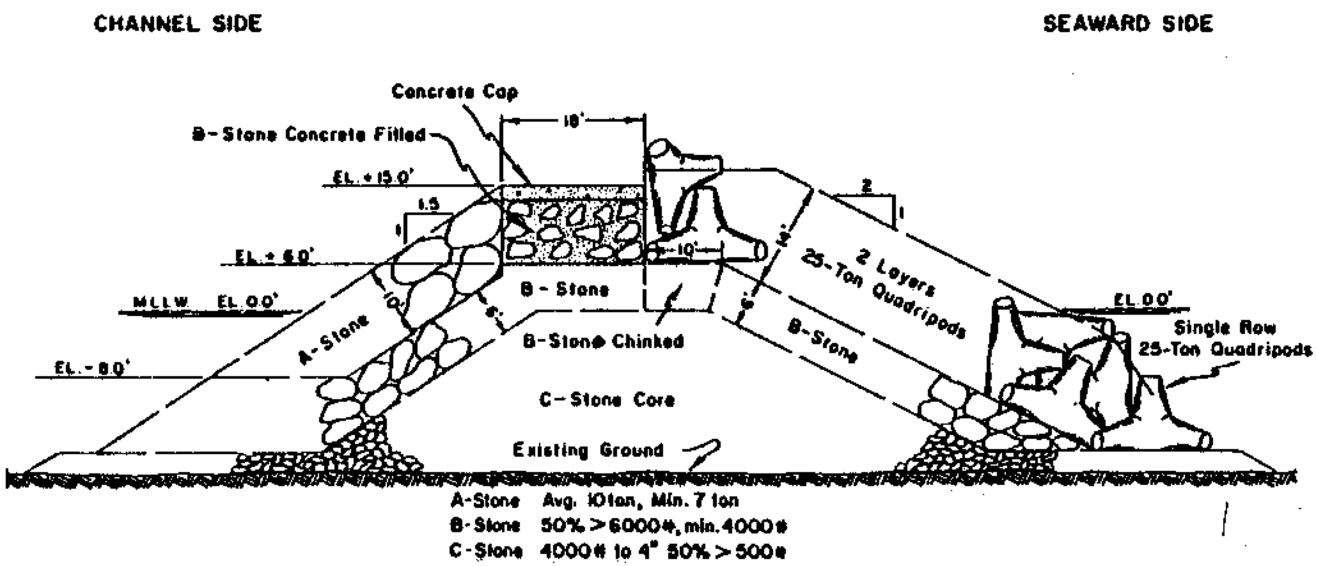


Figure 4.1 Rubble Mound Jetty (CERC, 1977, p. 6-85)

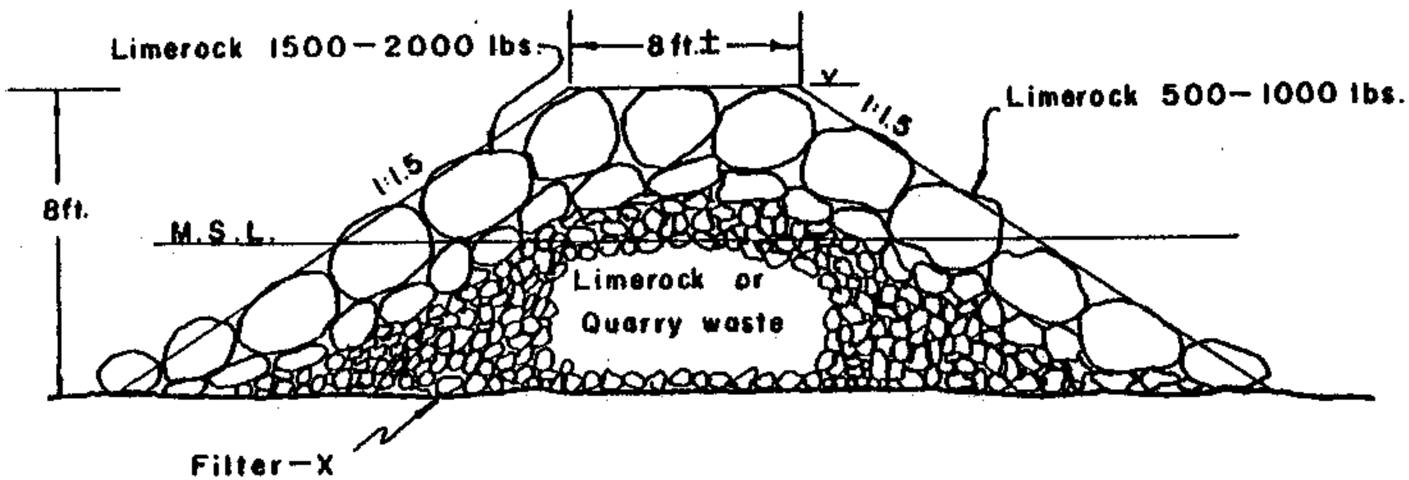


Figure 4.2 Rubble (Limestone) Mound Groin (Bruun and Manohar, 1963, p. 29)

specifications are detailed in Chapter 7. The availability of durable rock must be evaluated as an adjunct to the design phase. Investigative and laboratory methods to perform this task are presented in Chapter 6. When armor rock of the required size is unavailable, concrete shapes may be specially formed to serve in their place; the characteristics of concrete armor units are also described in Chapter 6.

Rubble mound jetties and breakwaters have been topped with poured-in-place concrete caps, as shown in Figure 4.1. Concrete use ranges from simply filling in the voids between armor layer units, to the much larger-scale casting of monolithic seawalls atop the mound crest. Caps are designed to strengthen the crest, increase its height, or provide a roadway along the crest for construction or maintenance access (CERC, 1977). These purposes are most applicable to the construction of large-scale shore protection structures.

There are several advantages to using rubble mounds. They are adaptable to any water depth and most foundation conditions. Settlement of the mound under wave action usually results in readjustment of the rock components to a more stable configuration, rather than in structural failure. Structural damage is progressive, when it develops, rather than sudden and potentially catastrophic. Damages are generally easily repaired. As noted in Chapter 3, rubble absorbs rather than reflects wave energy, a beneficial characteristic. On the negative side, excessive transmission of wave energy may occur if the rubble mound core is too low and porous. An additional disadvantage is the large quantity of material required, an amount which increases considerably for small increases in water depth. The initial project cost is likely to be high if suitable construction materials are not available locally (CERC, 1977).

The ability to produce large quantities of rock economically, and the improvement of rubble mound design methods, have led to their extensive use as shore protection elements. In view of their importance, the design of rubble mound structures warrants particular attention. Chapter 7 is devoted to presentation of rubble mound design technology.

Gabions

The adoption of polyvinyl chloride (PVC) coated wire, more than 20 years ago, for the manufacture of gabions enabled their use to be extended to the coastal environment. The rock-filled wire baskets and mattresses have been formed into mounds and incorporated into rubble mounds to provide coastal defense works. Dimensions and other features of gabions are included in Hubbell and Kulhawy's (1979a) survey of coastal construction materials. The advantages of gabions, with respect to this application, are: 1) they are highly flexible and will adjust to differential settlement, as caused by undermining from wave and current scour, 2) they can be filled and placed underwater with minimal problems, 3) hydrostatic heads do not develop behind the permeable gabions, and 4) they are often an economically attractive alternative. Wave energy is absorbed within the interstices of the stones and, unlike riprap, the rocks remain securely encased.

Gabions are well-suited to the construction of groins. The individual building components are easily added or removed, so that the groin configuration can be altered in accordance with its effect on the shoreline. The permeable gabions allow penetration of littoral drift through the structure, a desirable feature which results in more uniform beach accretion. The groin illustrated in Figure 4.3 is designed of rock-filled wire mesh mattresses over a core of stone or sand fill. Groins similar to the stepped mound design in Figure 4.4 may be employed for shoreline stabilization. A wide apron around the structure ensures stability. The ample flanks can settle and adjust to undermining by erosion without threatening the structural integrity and usefulness.

On rubble mound breakwaters and jetties, gabions are used to cap and protect the underlayers (Figure 4.5). In an innovative project, gabions were used to form the breakwaters built at Tristan da Cunha, in the South Atlantic, circa 1964, when the islanders returned following a volcanic eruption. The two shore-connected breakwater arms comprise rockfill founded on lava, overlain by a sloped facing of gabions. Though the small harbor protected is exposed to extremely violent wave action, damages to the gabions have been limited (Crowhurst, 1981).

Along the coast of Bedok, Singapore, offshore breakwaters were constructed entirely of gabions. These reached to just below the low water mark, to encourage the deposition of sand on the beaches immediately in their lee. A disadvantage of the chain link mesh used is that breakage of single strands of wire can lead to unravelling and the eventual collapse of the gabions. To date, these structures remain in reasonable condition and have fulfilled the design objectives. In this case of relatively light wave action, a vertical stepped face was used. Where heavy wave action is anticipated, it is essential to use sloping faces to allow additional energy dissipation in runup (Crowhurst, 1981).

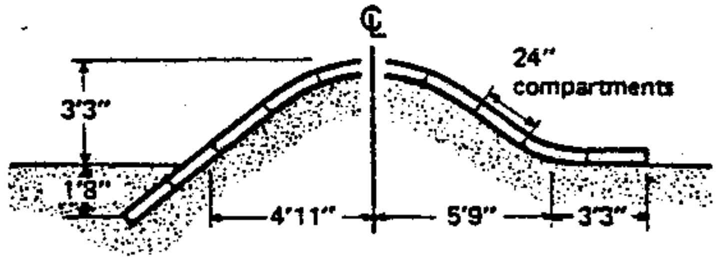


Figure 4.3 Revet Mattress Groin (Maccaferri Revet Mattress Catalog, undated, p. 11)

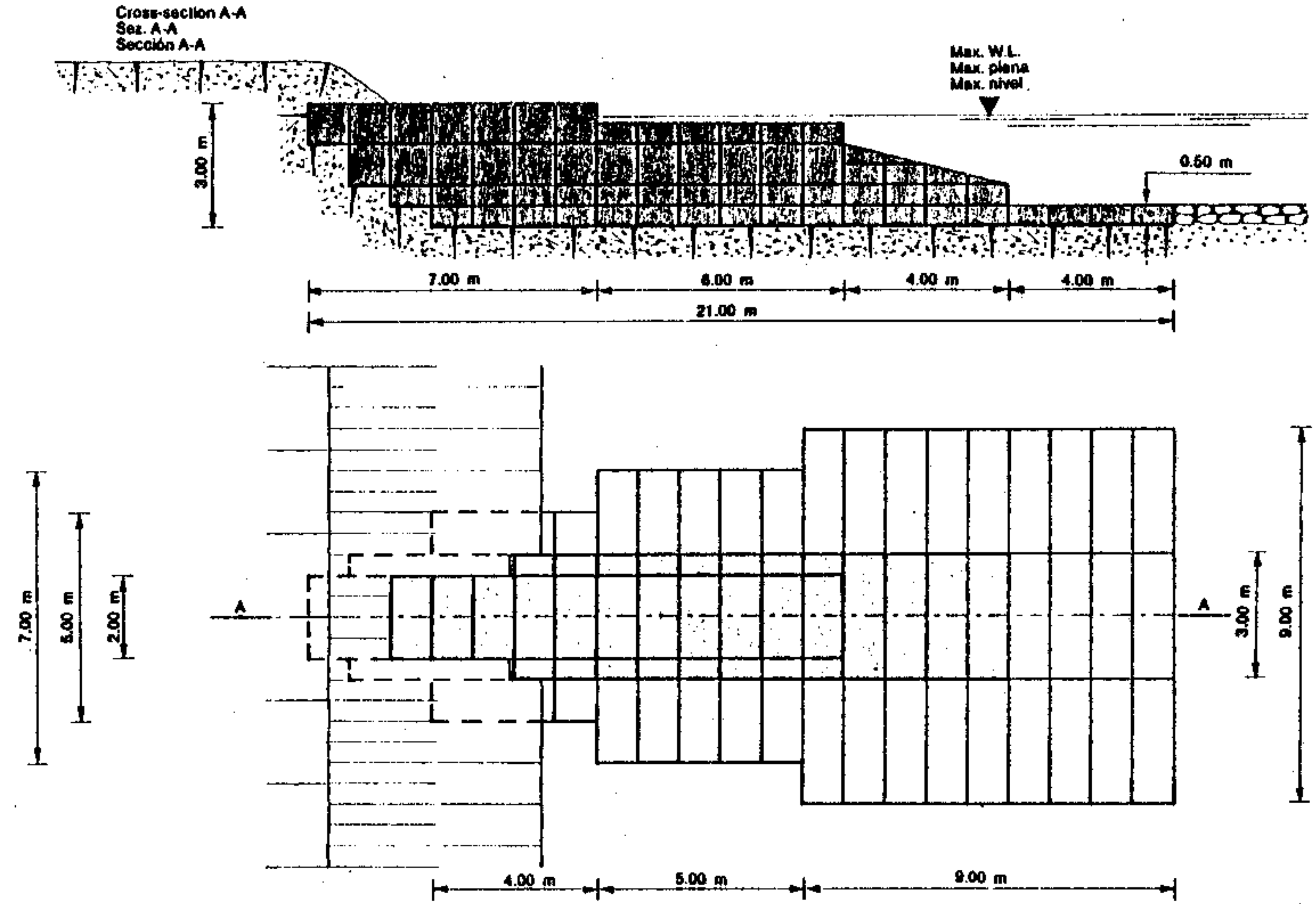


Figure 4.4 Stepped Mound Gabion Groin (Maccaferri Gabions Inc., 1979, p. 40)

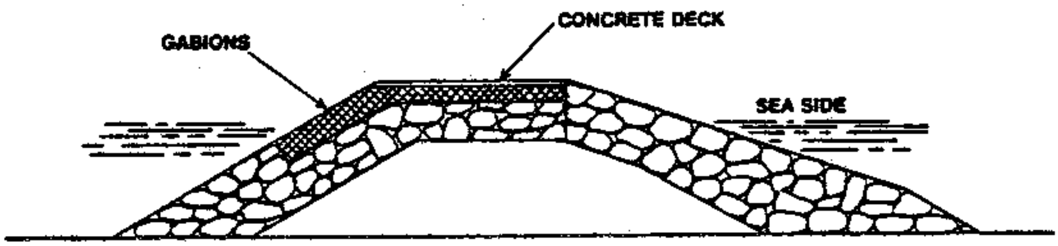


Figure 4.5 Gabion Reinforcement on Shoulder of Rubble Mound Breakwater (Bekaert Gabions, 1977, p. 54)

4.2 WALL STRUCTURES

Straight walls dissipate energy largely by reflection rather than by absorption. They also differ from mounds in that they may fail or be severely damaged by a single wave of more than design proportions (Dunham and Finn, 1974). Sheet pile structures consist of lines of piles interlocked to form a continuous wall. Piling materials include steel, timber and, less commonly, concrete. Configurations range from single walls, for small structures and low wave climates, to double and cellular walls for more massive structures with more severe exposures. Caissons, piles and cribs are other structural variations within the wall group.

Regardless of the configuration used, attention must be given to foundation considerations (See Chapter 5). Piles must penetrate to a sufficient depth to attain structural stability against overturning. Wall structures cause waves to generate scouring currents, which can erode unconsolidated foundation materials and result in severe undermining. Sheet piles have sometimes lost so much embedment as to threaten their structural integrity. Cellular walls and caissons, which rest on the bottom rather than penetrate to depth, are particularly vulnerable; they have occasionally toppled seaward into their own toe scoured trenches (Dunham and Finn, 1974). To protect against damaging erosion, riprap must be placed along the toes of wall structures.

Sheet Pile Structures

Steel Sheet Piles. Single wall steel sheet pile structures are used in low wave areas. In accordance with this constraint, they are most successfully employed as groins, onshore breakwaters and other shore protection elements subject to low structural loads. These systems may be designed as described by Saczynski and Kulhawy (in preparation). The wave and soil forces to be resisted are evaluated to determine the required depth of penetration of the sheet piles. This value varies considerably with the nature of the foundation material and, for this reason, a careful foundation study is warranted. The stability of the single wall depends on its strength as a cantilever beam. Where the imposed bending forces are small, straight web piles may be sufficient. To resist greater forces, deep web sections should be used. The structural members of the groin illustrated in Figure 4.6 are deep web Z piles, restrained at the top by a steel channel.

When the combined design wave and soil forces exceed the cantilever strength of the sheet pile wall, bracing must be incorporated to prevent overturning. The single wall can be simply buttressed, as in Figure 4.7, by short lines of piles driven perpendicular to the main structure. Bracing is similarly obtained by double wall construction. Two parallel rows of sheet piling are connected and braced against each other with tie rods and crosswalls, as shown in Figure 4.7. Each wall is stiffened with inside wales. For added stability, the structure is filled with granular material and capped with concrete, asphalt or heavy rubble (USCOE, 1963).

The third steel sheet pile structural variation is the cellular configuration. The groin illustrated in Figure 4.8 is of the diaphragm type, a series of arcs connected to cross diaphragm walls. Granular fill and capping provide added weight for structural stability. The outward pressure from the fill results in circular or hoop tension in the walls, contributing to resistance against tilting and overturning.

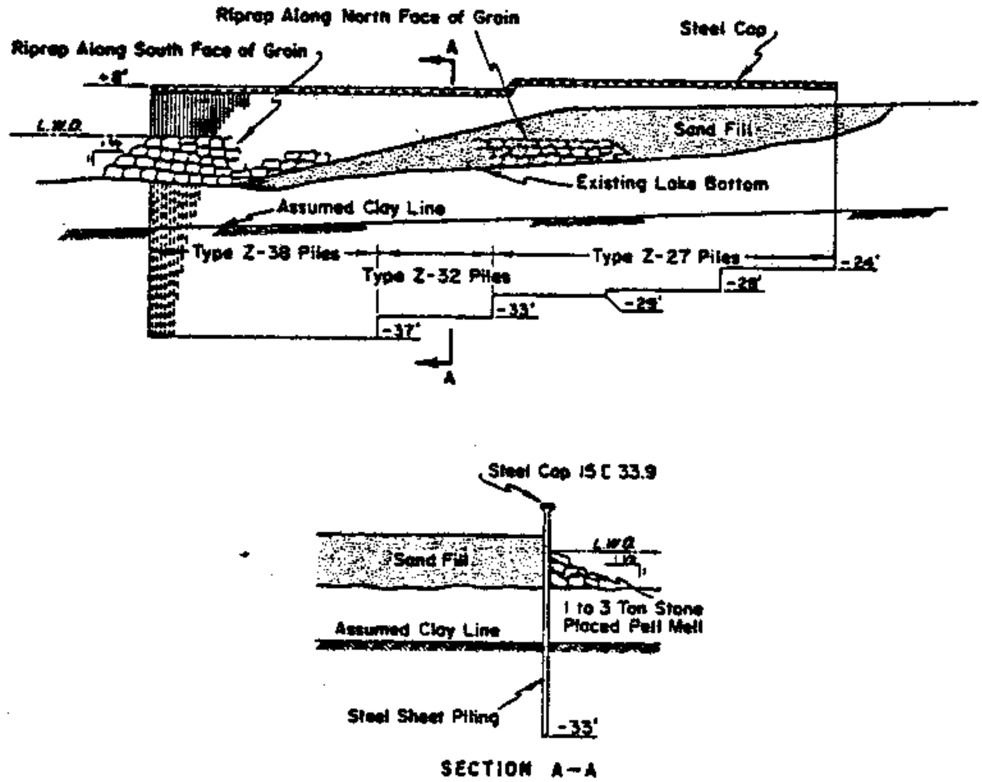


Figure 4.6 Cantilever Steel Sheet Pile Groin (CERC, 1977, p. 6-79)

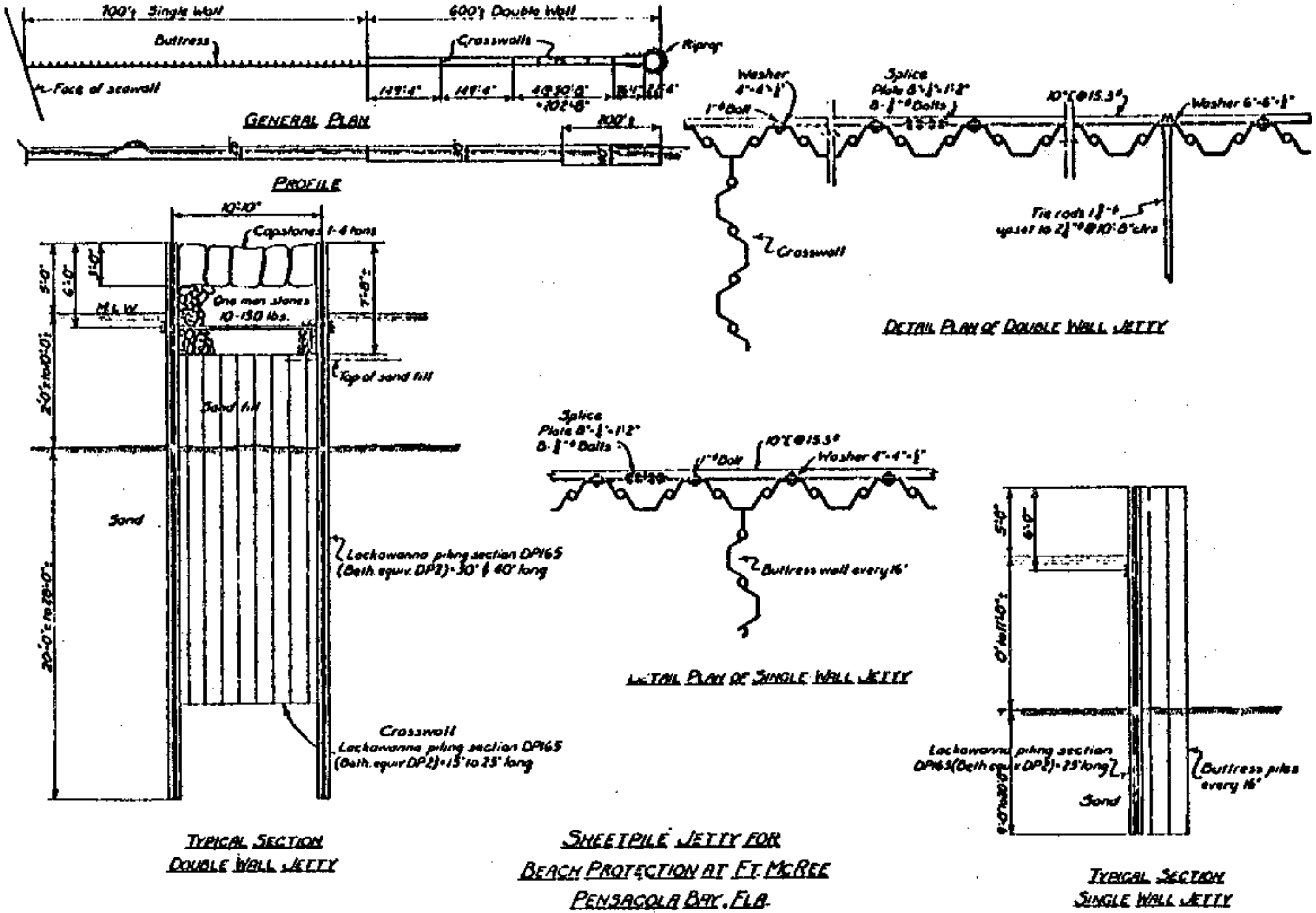


Figure 4.7 Buttressed Single Wall and Double Wall Steel Sheet Pile Jetty (USCOE, 1963, pl. 31)

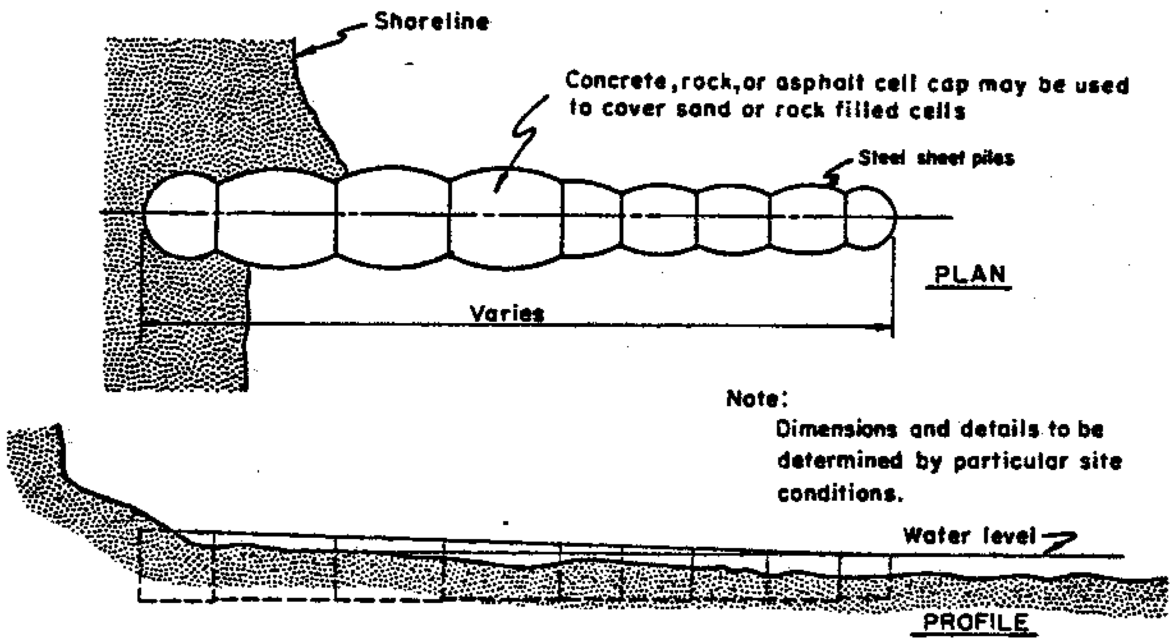


Figure 4.8 Diaphragm Type of Cellular Groin (CERC, 1977, p. 6-80)

The circular type (Figure 4.9) consists of complete circles connected by shorter arcs. Figures 4.10 and 4.11 typify the designs of two large-scale structures. Each cell must be stable against sliding, overturning, and rupture in the web and interlocks. Rupture is often traced to driving the piles out of interlock, which can result from overdriving through hard material or deflection of the piling by boulders (USCOE, 1963).

Cellular sheet pile structures may serve in moderate wave climates where storm waves are not too severe. Cellular breakwaters, jetties and groins have been built with considerable success on the Great Lakes. They can be used in a wide range of foundation conditions and are suitable where adequate pile penetration cannot be obtained. They can be installed in water depths up to 40 ft (12.2 m) and require little ongoing maintenance (CERC, 1977). A major drawback to their use is construction difficulty. The cells are economical and quick to erect, but are extremely vulnerable to wave and storm attack during construction. The diaphragm wall is filled in stages, keeping the height in adjacent cells nearly equal to avoid distortion of the piling. The cells of the circular type are filled as soon as the piles are driven. Until the circles are completely closed, however, the structure has virtually no stability and, correspondingly, no defense against damage. Only in areas like the Great Lakes, where there are periods of good weather and calm water, is the use of sheet pile cells practical (Quinn, 1972). Another limitation to their widespread use is that of material corrosion, discussed by Hubbell and Kulhawy (1979a).

Timber Sheet Piles. Timber sheet piling is suitable for structures subject to moderate wave action in relatively shallow depths. For this

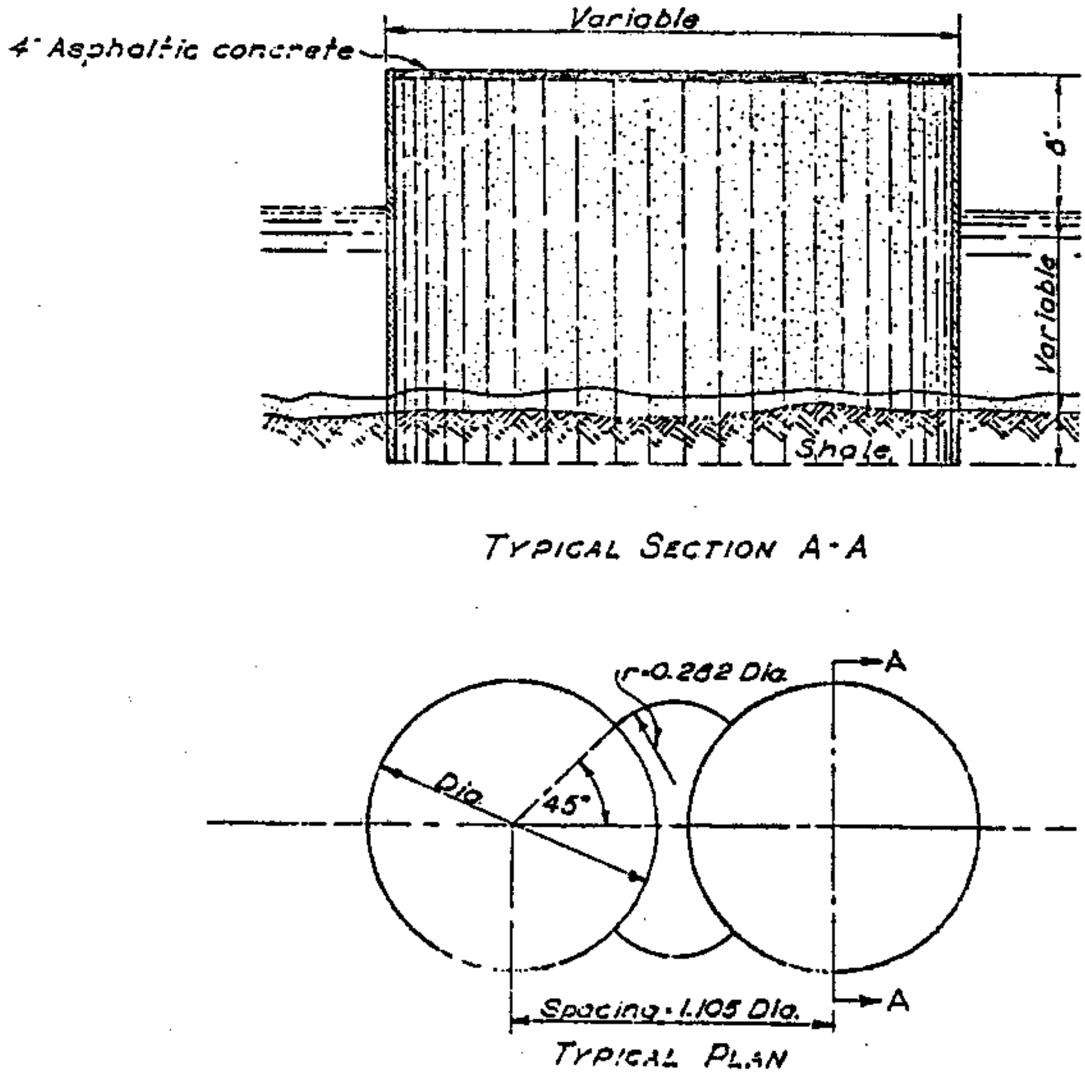
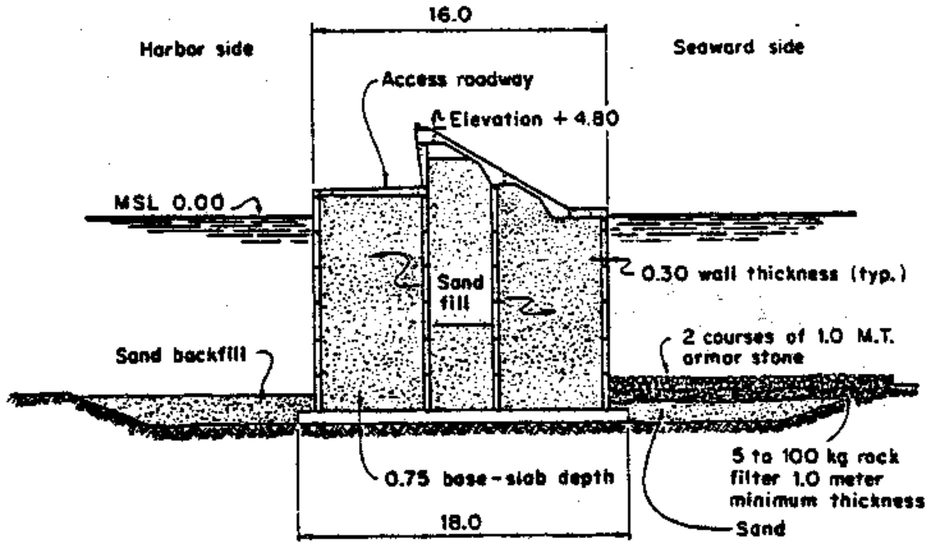


Figure 4.9 Circular Type of Cellular Breakwater (USCOE, 1963, pl. 15)



(dimensions in meters)

Figure 4.10 Cellular Steel Sheet Pile Breakwater at Marsa el Brega, Libya (Quinn, 1972, p. 250)

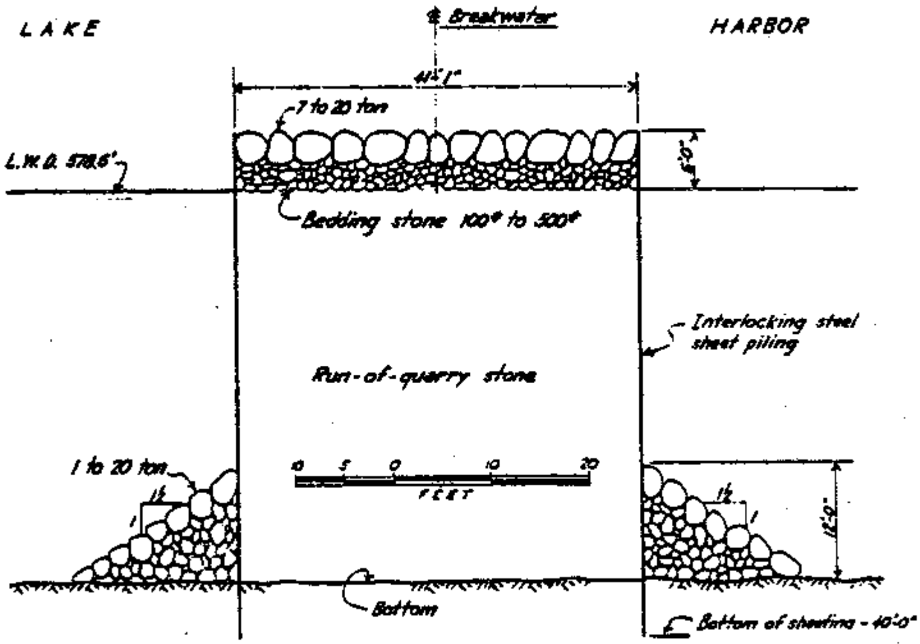


Figure 4.11 Cross-Section through Cellular Sheet Pile Breakwater at Calumet on Lake Michigan (Quinn, 1972, p. 252)

reason, timber groins are much more abundant than timber breakwaters and jetties. In any application, timber piling is not appropriate for use on open, exposed shores. In view of the high cost, maintenance costs and somewhat low life expectancy, timber should be considered only where the purpose and local conditions warrant its special use (USCOE, 1963).

Figure 4.12 demonstrates the use of timber in a typical groin configuration. Timber sheet piles are made of two 3 inch (76 mm) thick timber boards staggered in a shiplap joint. This vertical wall is framed into a system of horizontal wales or stringers. Primary structural support for the unit is derived from penetration of the round timber piles. The wales and round piles also distribute the wave loads and limit wall deflection and the opening of joints between adjacent sheet piles (Ayers and Stokes, 1976).

A low cost variation of this timber groin is shown in Figure 4.13a. Piles are driven into the bottom in pairs, with planks sandwiched between them. Because the planks cannot be embedded deeply when working underwater, this method is limited to areas of wide tidal range where construction can proceed during low tide. Rubber tires on timber piles (Figure 4.13b) comprise another low cost configuration, effective where adequate pile penetration is obtainable. Horizontal timber crosspieces keep the tires from floating off the tops of the piles in high water (Rogers, Golden and Halpern, 1981).

<u>Concrete Piles</u>. Concrete is one of the less common pile materials employed in the construction of shore protection structures. A concrete groin system constructed on the east coast of Niigata, Japan, is shown in Figure 4.14. A bulkhead type breakwater (Figure 4.15) may be suitable where soft bottom material extends to considerable depth and

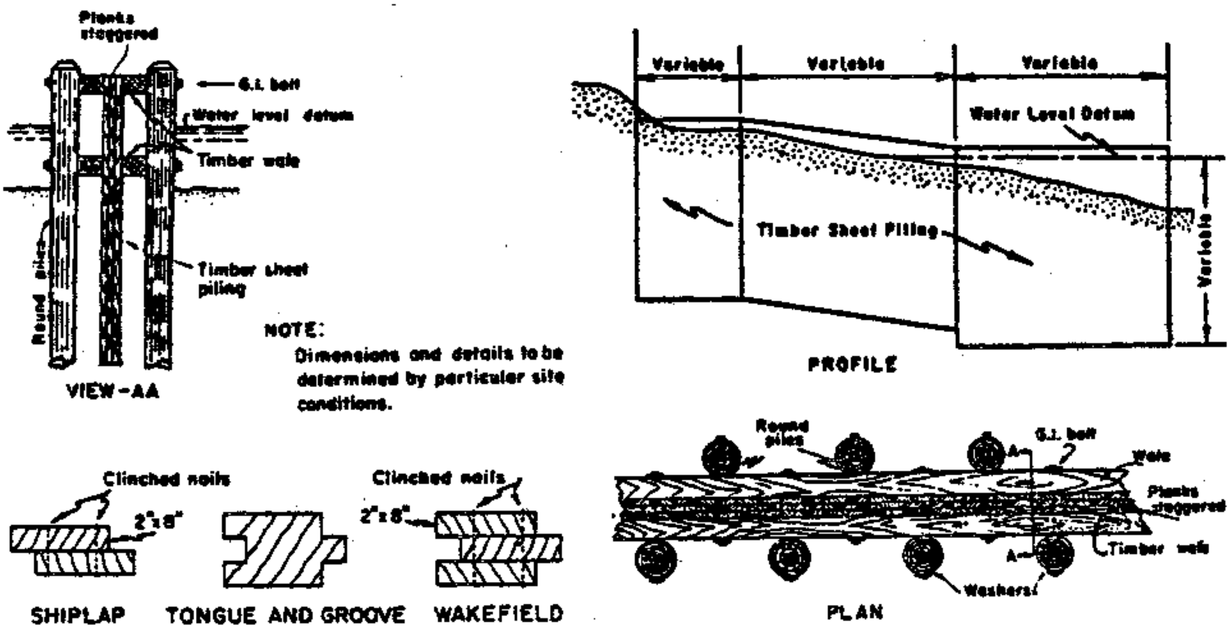
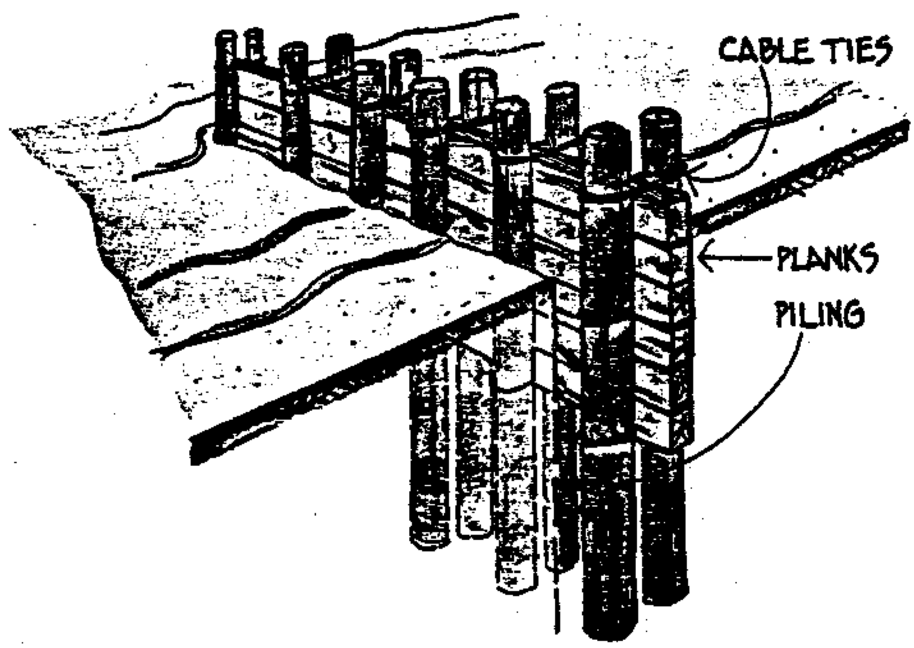
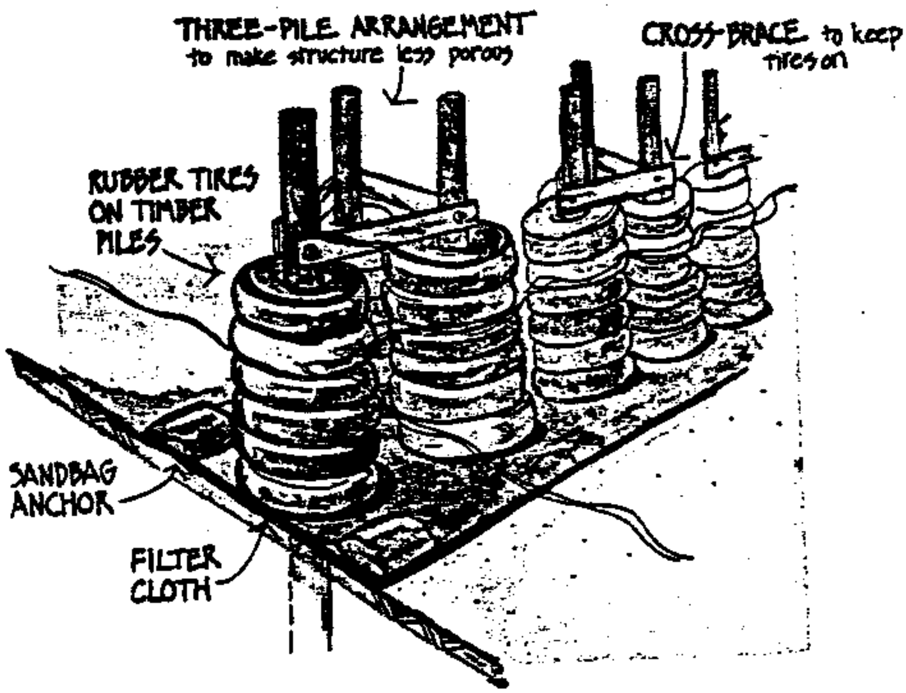


Figure 4.12 Typical Timber Sheet Pile Groin (CERC, 1977, p. 6-77)



a. Timber Groin



b. Timber Breakwater

Figure 4.13 Low Cost Timber Shore Protection (Rogers, Golden and Halpern, 1981, pp. 17 and 20)

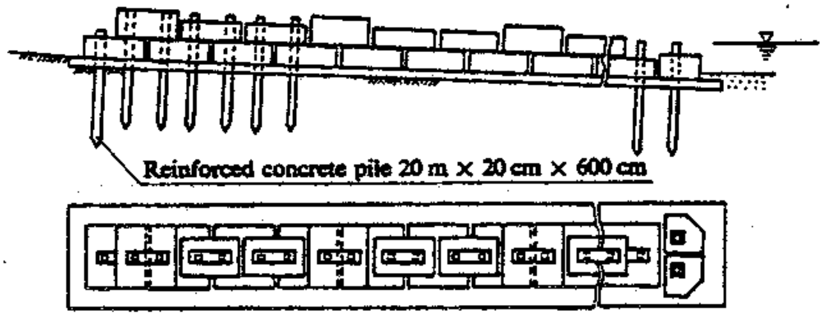


Figure 4.14 Concrete Block Groin, Niigata, Japan (Horikawa, 1978, p. 331)

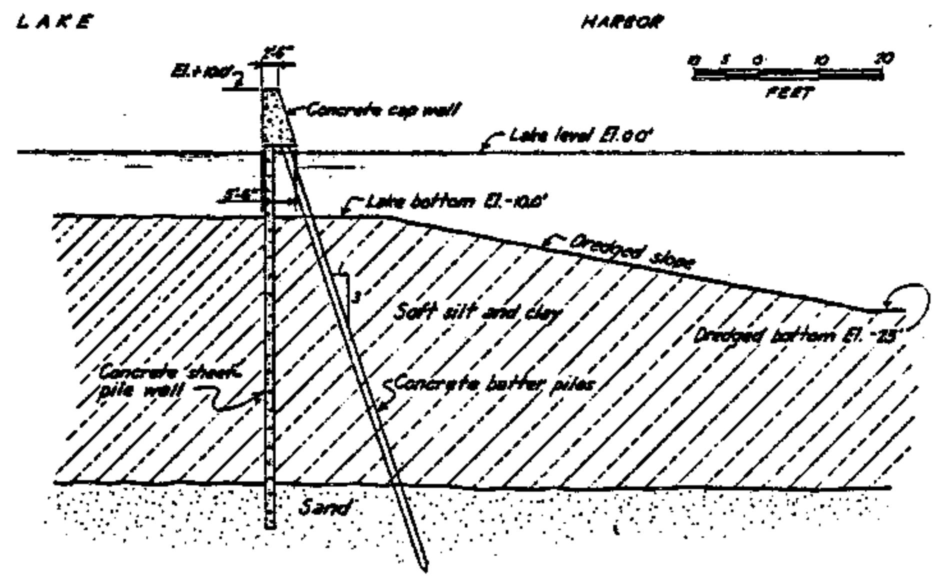


Figure 4.15 Concrete Sheet Pile Breakwater (Quinn, 1972, p. 256)

the wave height does not exceed 10 ft (3.0 m). Concrete sheet piling and batter piles are driven through the soft stratum into the underlying bearing material. These are capped above low water level with a poured-in-place wall (Quinn, 1972).

Concrete Caissons

Caissons used in coastal construction are reinforced concrete shells with diaphragm walls which divide the box into several compartments (Figure 4.16). The units are floated into position and settled on a prepared foundation, either a rubble mound or piles. The structure is filled with stone or sand and capped with concrete or armor units for stability. A cast-in-place parapet wall may be added to protect against overtopping. Heavy riprap placed along the base of the caissons protects against scour and weaving on pile foundations, and adds resistance to horizontal movement (CERC, 1977).

This type of construction has been used for breakwaters in the Great Lakes and for harbor protection in Europe. This scheme permits a large amount of work to be done on land, an advantage where the sea is rough and the working time of floating equipment is constrained (Quinn, 1972). Caissons can be used in depths of 10 to 35 ft (3 to 11 m). Their use is limited to breakwater and jetty construction; groins are rarely subjected to forces that would justify usage of concrete caissons.

Cribs

Cribs built of timber or precast concrete elements are utilized in much the same manner as concrete caissons. Floored cribs are settled on a prepared foundation and filled with stone. Timber, concrete or cap

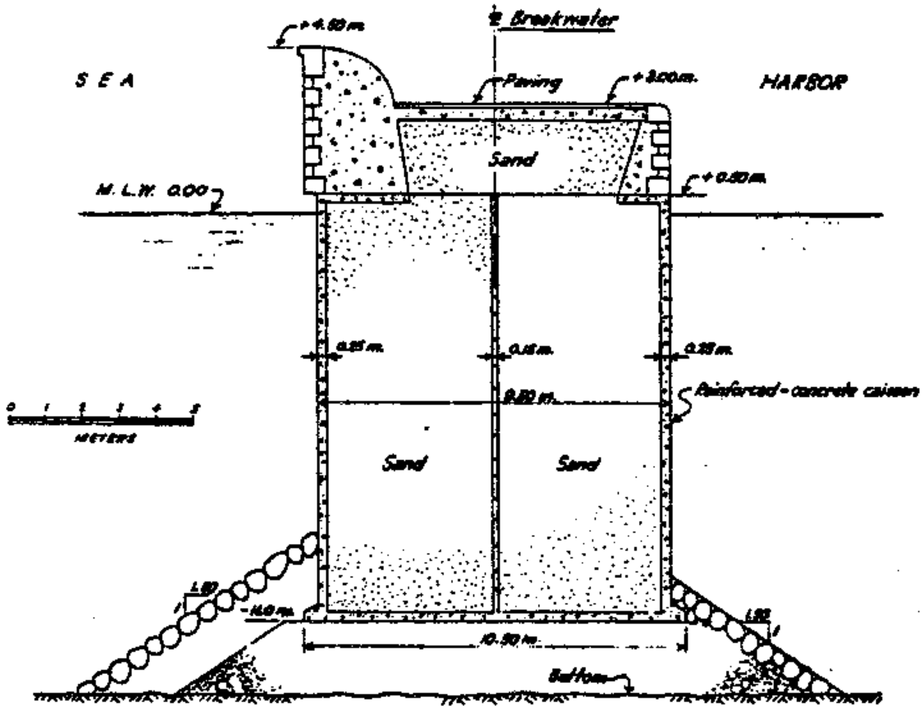
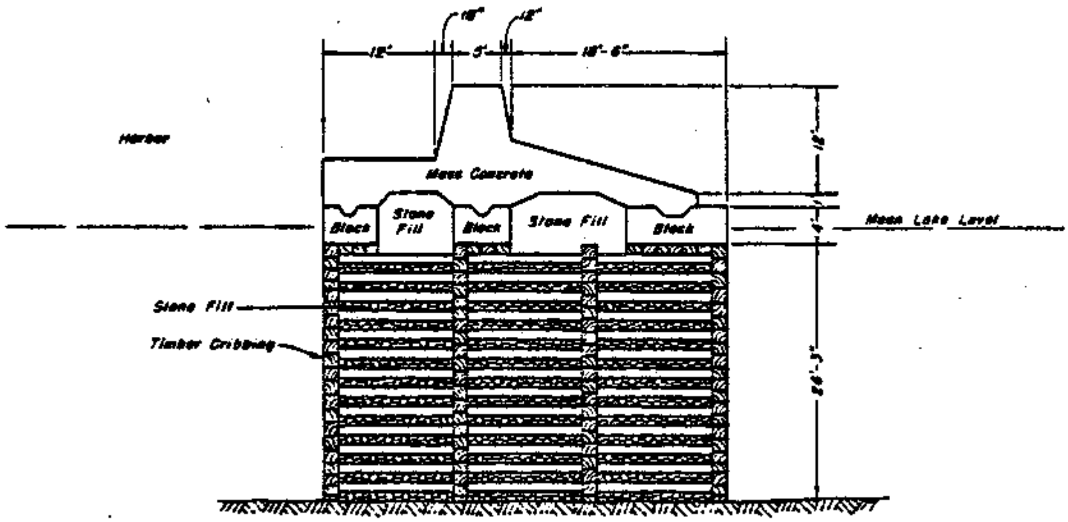


Figure 4.16 Concrete Caisson Breakwater, Helsingborg Harbor, Sweden (Quinn, 1972, p. 249)



BREAKWATER AT HARBOR BEACH, MICHIGAN

Figure 4.17 Timber Crib Breakwater (USCOE, 1963, pl. 16) stones provide, by their weight, additional stability. Rock-filled timber cribs can withstand considerable racking and settlement without rupture (USCOE, 1963). These have been used most extensively on the Great Lakes, particularly in the past when timber was relatively cheap in the area. A typical timber crib breakwater is illustrated in Figure 4.17.

4.3 LOW COST SHORE PROTECTION

The state-of-the-art of shore protection has been largely directed at the protection of public and commercial property. However, 75 percent of the United States shoreline, excluding Alaska, is privately owned (Cousins and Lesnik, 1978). Extensive and costly annual property loss is due, in part, to the private landowner's use of poorly conceived and improperly executed shore protection techniques. There is a great need for information about low cost and usually smaller-scale protection devices that can be successfully implemented by individual property owners. In response to this need, Congress passed the Shoreline Erosion Control Demonstration Act of 1974. The legislation authorized the Corps of Engineers to conduct a five year, eight million dollar program to develop, demonstrate and evaluate low cost erosion control methods and disseminate conclusions and guidelines to the public. The final project report, presently in press, promises to provide important technical assistance to private landowners. Sources of further project information are listed in Appendix A. An outline of the project framework follows.

Sixteen demonstration sites were chosen in the Delaware Bay, Atlantic, Pacific, Gulf, Alaska and Great Lakes coastal regions. The erosion control projects installed were governed by the low cost criterion, defined as \$50 and \$125 per front ft (\$164 and \$400 per m) of device. The former figure is for materials only, assuming the landowners install the device, and the latter is for materials and labor, assuming a contractor and heavy equipment would be necessary for installation. The measures studied were intentionally of simple design and intended to perform only on low energy coasts, with a maximum wave height of 6 ft (1.8 m). Protection was designed for a ten year life with minimum maintenance requirements. Materials and techniques were selected to be compatible with the geographical region of each project (Housley, 1978; Cousins and Lesnik, 1978).

A sampling of the techniques proposed, in 1974, to be studied is given in Table 4.1. Some of these methods are previously tested techniques on which better performance and cost data are needed; some are innovations being tried for the first time. Many are adaptations of larger-scale shore protection technology while others seem particularly suited to low energy, low cost, small-scale applications (Housley, 1978). Mounds, sheet pile walls and floating breakwaters are potential low cost methods which have already been presented as structural variations. Other breakwater and groin construction materials and configurations cited in Table 4.1 are discussed briefly in this section. The final report of the Shoreline Erosion Advisory Panel (Appendix A) should be consulted for general conclusions and design guidelines regarding these methods.

Table 4.1 Low Cost Shore Protection Techniques

Material	Erosion Control Structure*			
	Breakwater	Groin	Revetment	Bulkhead and Seawall
Rubble	✓		✓	
with Asphalt Mastic		✓		
Sheet Piles	✓	✓		✓
Gabions	✓	✓	✓	
Fabric Bags	✓	✓	✓	✓
Longard Tubes	✓	✓		
Rubber Tires	✓		✓	✓
Cribs			✓	✓
Z Wall	✓			
Concrete Blocks	✓		✓	
Corrugated Pipes		✓		
Steel Fuel Drums		✓		

* Additional tested methods include: coastal vegetation, beach fill, perched beaches.

Longard Tubes

The Longard tube is manufactured by the Aldek Company of Denmark and distributed in the United States by the Edward Gillen Company of Milwaukee, Wisconsin. The Longard tube is essentially an envelope of material given structural capability by sand filling. The tube is a polyvinyl-coated outer shell of woven material lined with polyvinyl sheeting. Sand pumped as a slurry into the tube provides the shell with weight and strength. A trap of filter cloth at one end retains the fill while allowing water to drain out.

Longard tubes have been used as groins in Michigan's Demonstration Erosion Control Program, a study similar in purpose to the federal program. The 42 and 69 inch (1.1 to 1.7 m) diameter tubes were installed singly and stacked, one on two, in a pyramid configuration. To keep the costs of installation within the low cost range, the tube groins were placed directly on the lake bottom with no foundation mat, filter layer or toe protection. Undermining and settlement of the structures was, consequently, serious on sandy bottoms. Longard tubes are susceptible to tearing and loss of sand, resulting from impact of ice, debris and boats, vandalism, or improper sealing during construction. Structure life and efficiency are limited subsequent to such damage.

Barring major damage, the tubes functioned reasonably well as groins. Longard tubes do not have the longevity associated with more massive, durable materials, but their low cost can offset this primary disadvantage. In the Michigan project, the cost of Longard tubes was as low as \$40 per ft (\$131 per m) front of shoreline protected. The ease of construction, too, recommends the tubes as a competitive new concept in shore protection (Armstrong and Kureth, 1979; Brater, et. al., 1977).

Sand-filled Bags

A number of field installations of the Michigan Demostration Erosion Control Program made use of large nylon sand-filled bags as groins and revetments. Several of the structures were damaged by vandalism and impact by debris. An interim project evaluation concluded that the sand-filled bags were failing at such a rate that considerable cost would be required to restore and maintain their original condition (Brater, et. al. 1977). Yearly replacement of bags on the groins was projected as a necessary maintenance measure (Armstrong, 1976).

Sandbag groins, revetments and breakwaters have been constructed with varying degrees of success by private homeowners and communities. To generate rational design data, CERC initiated a project in 1968 to investigate the stability and effectiveness of sand-filled nylon bag breakwaters under the attack of shallow water waves. Results of full-scale laboratory tests, using standard size bags 5 ft (1.5 m) wide by 8 ft (2.4 m) long, are reported by Ray (1977). Several breakwater configurations similar to that shown in Figure 4.18 were tested in 12 ft (3.7 m) of water. Only breakwaters with crests above or slightly below the stillwater level effected wave attenuation greater than 30 percent. The data indicate that an effective sandbag breakwater, producing significant changes in wave height, will be susceptible to damaging amounts of bag movement and must be designed and constructed carefully to maintain a stable configuration. Some preliminary design guidelines

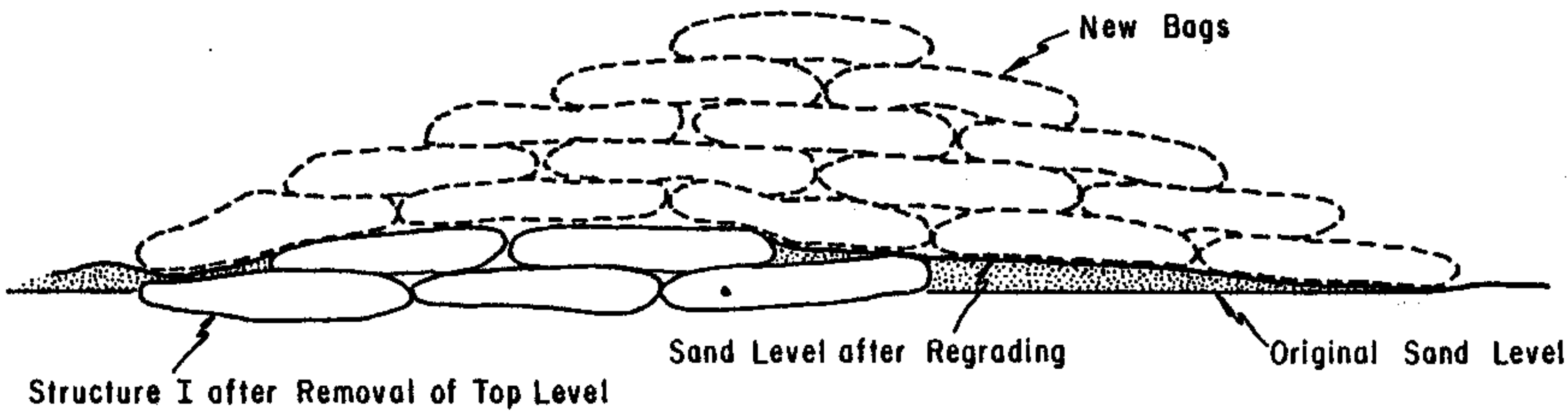


Figure 4.18 Sandbag Groin Experimental Cross-Section (Ray, 1977, p. 22)

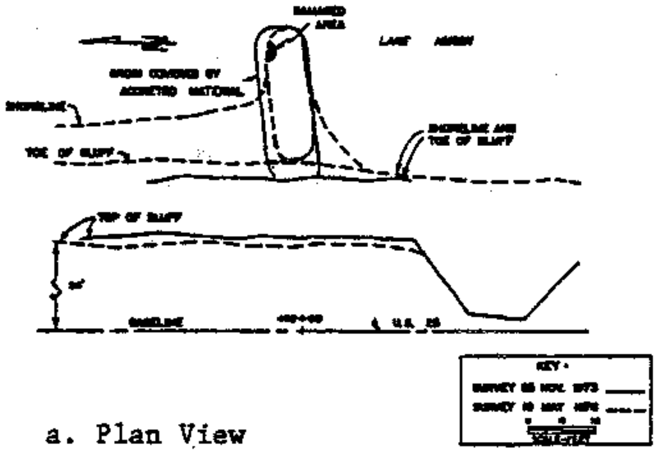
are given by Ray (1977) and additional information is expected in the Shoreline Erosion Control Demonstration Program report.

Testing problems associated with the use of sandbags included ultraviolet deterioration, closing filled bags and handling the bags, especially when frozen. A single uncoated nylon bag exposed to direct sunlight for 18 months tore open. Commercially marketed bags have since been improved with various plastic coatings to reduce exposure damage. Bags have been equipped with a self-sealing opening which allows them to be hydraulically filled while lying flat. Also, the bags are now being manufactured of heavier, more coarsely woven material with increased strength. Trapped air and water can more readily escape through the permeable envelope, enabling quicker consolidation and interlocking of the sandbags (Ray, 1977).

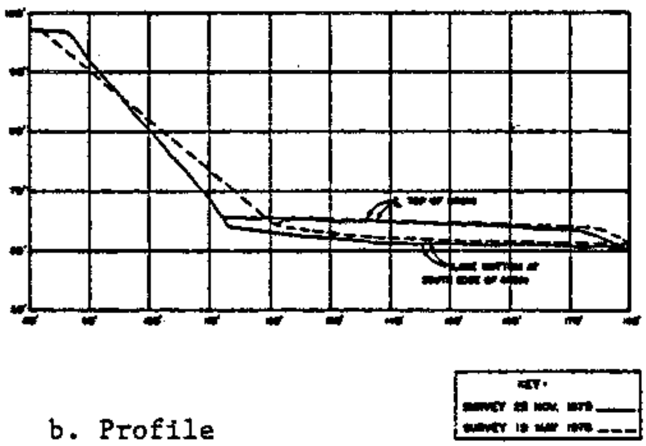
Rock Mastic

A rock asphalt-mastic groin was constructed in 1973 under the supervision of the University of Michigan's Coastal Zone Laboratory. Although existing literature recommended that mastic not be poured through more than 1 ft (0.3 m) of water, the installation of this groin demonstrated that mastic can be successfully poured through 7 ft (2.1 m) of water.

The rock mastic groin is 60 ft (18.3 m) long and has trapped large amounts of sand, providing a protective beach (Figure 4.19). The structure was installed at a cost of $45 per ft ($146 per m) of shoreline, and anticipated maintenance costs are quite low. The rock mastic lacks the aesthetic qualities of other materials, but the structure has proven stable and effective. The rock mastic groin has



a. Plan View



b. Profile

Figure 4.19 Rock Mastic Groin, Sanilac Township, Michigan (Brater, et. al., 1977, p. 38) performed satisfactorily and is a good example of successful, innovative low cost shore protection (Brater, et. al., 1977).

Precast Concrete Units

Permeable groins have been designed of precast concrete members and piles. Considerations in the use of waterfront concrete are presented by Hubbell and Kulhawy (1979a). A new concept in low cost breakwater design was tested in Pere Marquette Township on Lake Michigan. The breakwater consisted of precast, reinforced concrete panels bolted together to form zig-zag walls (Figure 4.20). Three walls were placed offshore, with 50 ft (15.2 m) spacings between structures. The breakwater system initially functioned well in building up a beach and preventing bluff recession (Figure 4.21). A major storm, with 6 to 10 ft (1.8 to 3.0 m) waves, then caused extensive damage to the breakwater and bluff. Presently, the structures are totally useless and bluff recession has continued unchecked. The experimental use of precast zig-zag walls was intended for onshore use only. Their performance in this offshore application was unsatisfactory (Brater, et. al., 1977).

The Pere Marquette breakwater was constructed without a foundation and toe protection so that it would fit the low cost classification. It is certain that inclusion of these basic features would have improved overall structural performance and averted such a failure. As demonstrated by this case, design modifications and omissions made for the sake of economy must be carefully weighed. Elimination of these aspects may save first-cost dollars, but will often result in structure undermining and settlement. A structure which eventually requires large maintenance expenditures or is rendered inoperable is no bargain. A

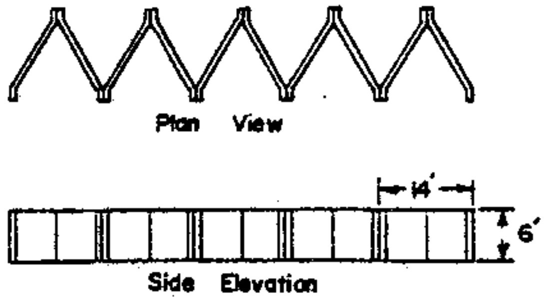


Figure 4.20 Precast Concrete Inshore Breakwater (Hanson, Perry and Wallace, 1978, p. 26)

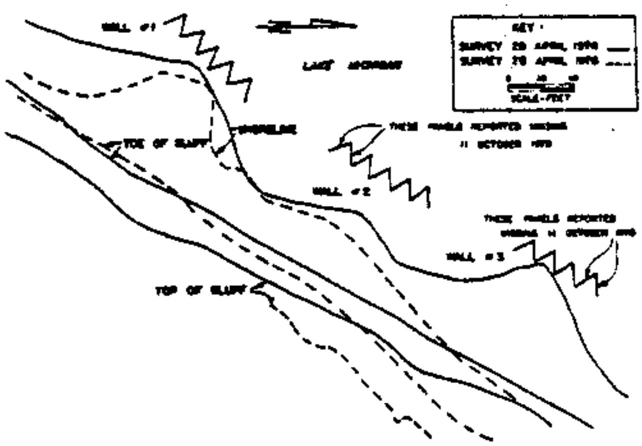


Figure 4.21 Concrete Zig-Zag Wall Breakwater, Pere Marquette Township, Michigan (Brater, et. al., 1977, p. 43)

little extra investment in properly engineered design at the outset could save greatly on overall project costs.

Other Materials

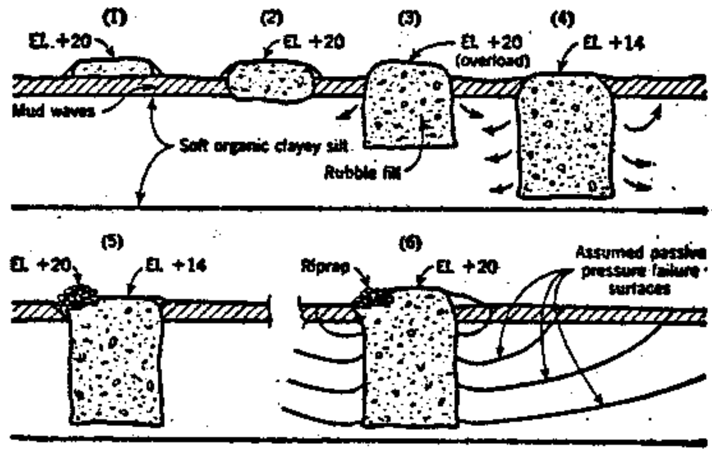
Any material that has an acceptable lifespan, is non-polluting and will remain stable under the imposing environmental forces has potential for shore protection construction. Low cost surplus ships, barges and drydocks are nontraditional building materials, yet can suitably perform as offshore portions of breakwaters, groins and jetties. They are simply towed into place and sunk. A major drawback to their use is the difficulty and cost of their removal when they deteriorate to the point of disuse.

Experiments with innovative no-cost materials proceed as well. One substance that is the subject of intensive research in the United States is stabilized blocks of waste material from coal fired power plants. In areas dependent on coal for electrical generation, the waste blocks might be used to build reefs and submerged breakwaters (Sanko and Smith, in preparation).

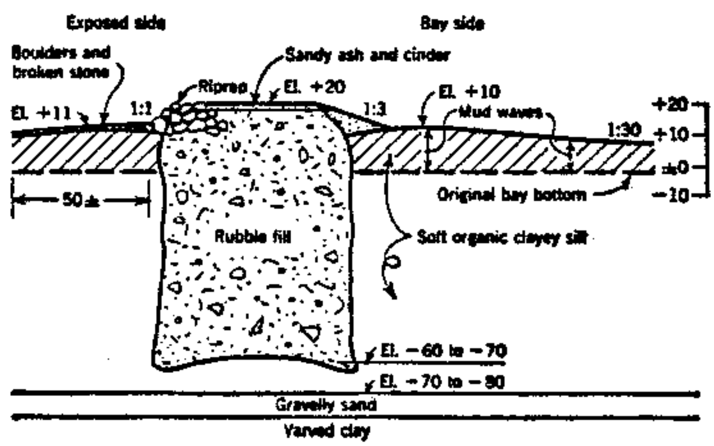
A rubble dike breakwater to protect small craft at the New York World's Fair Marina was built entirely of no-cost fill. Truckers paid a premium for the privilege of convenient disposal of heavy construction debris and rubble. The only method of achieving a stable embankment was to displace the 70 to 80 ft (21 to 24 m) of soft organic clayey silt deposits, replacing their volume with the fill. An overload to a height of 20 ft (6.1 m) above MLW was intentionally maintained throughout the fill process to assure displacement of the in-situ material. At the advancing tip of the breakwater, successive passive failures in the clayey silt formed mud waves around the mound as displacement progressed (Figure 4.22a). This heaving of bay bottom provided lateral support to the body of the fill and acted as a consolidation load to strengthen the remolded silt adjacent to the fill. The construction rate of sinking was approximately 1 ft per hour (0.3 m per hour). In the final configuration (Figure 4.22b) probings indicated that the mound sides were vertical and that rubble had penetrated as much as 70 ft (21 m) into the soft organic silt. The breakwater has a length of 3000 ft (914 m) and a crest width of 40 to 50 ft (12 to 15 m) at 18 to 20 ft (5.5 to 6.1 m) above MLW. The costs incurred in this unique project were for engineering design and supervision of construction only. Similar displacement embankments might be appropriate where the underlying deposits are too soft to support fill loads and the resulting displacements can be tolerated, and an ample supply of inexpensive fill is available (Torikoglu, 1966).

4.4 SUMMARY

The general structural variations of breakwaters, jetties and groins are similar. The exact purpose and scale of the project play a major role in selecting from among the available configurations; harbor breakwaters and jetties are typically massive structures of conventional design while smaller jetties and groins are suited to a wider range of materials and designs. The three structural groups addressed are mounds, walls, and low cost shore protection methods. Construction materials are discussed only briefly here; a more complete treatment of the subject is included in Hubbell and Kulhawy (1979a).



a. Sequence of fill construction



b. Final configuration

Figure 4.22 No-Cost Fill Breakwater, New York
(Torikoglu, 1966, p. 59)

Mounds are broad-based structures which derive their stability largely from their weight. They absorb and dissipate wave energy through runup on their rough, sloped faces. The most advantageous characteristic is their response to damage; they tend to settle and readjust progressively, usually without severe consequences. Rubble mounds, comprising layers of quarried rock, are the most common structural configuration of breakwaters, jetties and groins. They are effective structures, because of the large laboratory and field data bases associated with their design. The use of gabion mounds is less widespread at present, but seems to be a viable alternative. Gabions are particularly appropriate for groin construction, where the transmission of wave energy through the permeable structure is not critical.

Walls reflect wave energy. When attacked by waves higher than the design wave, they can fail suddenly; their design specifications must therefore be more demanding. Steel and timber sheet piles can be used in low, moderate or higher wave climates, in single wall, double wall or cellular configurations. Foundation considerations (Chapter 5) are quite important in assuring pile penetration to the design depth. Concrete caissons can serve as larger-scale breakwaters and jetties. At these and other wall structures, riprap must be placed along the base to protect against foundation scour.

Low cost shore protection is a new and exciting trend in small-scale protection alternatives. Low cost breakwaters and groins of innovative design and unusual materials are among the experimental structures being studied by the U.S. Corps of Engineers. Sand-filled tubes and bags, rock-mastic mounds, gabions, and floating breakwaters appear to be successful and competitive protection methods. State-of-the-art information on low cost shore protection developments can be obtained as described in Appendix A.

CHAPTER 5

STRUCTURAL DESIGN CONSIDERATIONS

The structural design of shore protection structures is initiated with an evaluation of the nature and intensity of environmental loads. Breakwaters and jetties are built primarily for the purpose of resisting these forces; groins also must remain stable under their attack. Waves impose the most critical loads on rubble mound structures. Other common and likely loading conditions addressed in this chapter are currents, soil stresses, impact pressures, ice, earthquakes and tsunamis.

The influence of soil and foundation conditions on the stability of rubble mounds must not be underestimated. A slight case of toe scour, innocuous at inception, can proceed to cause structural damage and, in the extreme, can result in radical breaching and failure. The effects of settlement and inadequate soil bearing capacity can be similarly severe. The potential for such difficulties should be identified in the initial phases of analysis so that remedial measures can be incorporated in the foundation planning. Foundation design deserves at least as much attention as the structural design of the overlying mound. This topic is introduced in the second section.

Mounds are flexible structures composed of discrete elements. Under attack by environmental forces, individual units move relative to each other and readjust to a stable configuration. Similarly, scour and foundation settlement may cause the structure to subside and deform, but the damaged mound will generally continue to function, to some degree, as intended. Rubble mound damage is progressive and therefore will not induce immediate and catastrophic failure.

The flexible behavior of mounds is a design advantage. When some measure of damage is allowable, design can be based on loads lower than the maximum which can occur. It is more economical to tolerate some damage, and to repair the mound periodically, than to preclude damage by designing for the maximum loading condition. For small-scale shore protection devices, foundation and structural design based on maximum environmental loads is generally considered overdesign and, therefore, not efficient engineering.

5.1 SUMMARY OF DESIGN LOADS

Coastal engineering design requires an analysis and understanding of the response of coastal structures to environmental loads. Loading conditions depend intrinsically on the purpose and orientation of the structure; for example, sheet pile harbor bulkheads are subject to forces different in nature and intensity from those which act on sheet pile groins. Environmental loading depends, by definition, on the site characteristics as well. Common coastal zone loadings are described by Hubbell and Kulhawy (1979b).

The behavior of rigid and flexible structures under the same loading condition is radically different; structural type, then, is the key to structural response. Construction materials and methods are interrelated contributing factors. Design methodologies outlined by Hubbell and Kulhawy (1979b) relate predominantly to vertical-faced rigid structures. This section addresses environmental loads as they affect mound type structures, as a prelude to the design procedures presented in Chapter 7.

Current rubble mound technology cannot quantify conclusively the complex forces required to displace individual armor units from the cover layers. At present, empirical design methods (See Chapter 7) include wave parameters as the only environmental load contributing to mound stability. Because wave loading controls mound design, it is particularly important to choose and characterize properly the design wave.

Certainly, other loads are acknowledged as affecting stability. The foremost among these, described in this section, include: 1) currents, 2) soil stresses, 3) impact pressures, 4) ice and 5) earthquakes and tsunamis. Although their influence has not yet been integrated into standard rubble mound design procedures, it should not be inferred that they are always of secondary importance. Ice and earthquake forces especially can be of primary importance, depending on the regional climatic and geologic conditions. When judged necessary, design modifications and reinforcements can be included to counter the actions of these forces.

Waves

The action of wind-generated water waves against coastal structures is the most constant and severe of environmental loads. The structural design of breakwaters, jetties and groins depends on the selection of the design wave height. Deepwater waves are evaluated, and propagated shoreward. Diffraction and refraction in shallow water affect the wave characteristics at the structure site. Hubbell and Kulhawy (1979b) review these topics and related analytical techniques in detail. Wave loads on vertical-faced structures are also presented in that work and will not be described here. This section will review general aspects of wave loading, emphasizing their relation to the design of rubble mound structures.

The water depth at the structure controls the type and height of waves which the structure will have to withstand. The depth is calculated from the hydrography and tidal range, and usually corrected for estimated storm surge and wave setup (see Hubbell and Kulhawy, 1979b). Structures may be subject to different forms of wave action as the water level varies at the site and along the structure length. Maximum wave forces on jetties and groins, for example, need not occur at the seaward end of the structures. The possibility of such variations should be considered in establishing water levels and design waves (CERC, 1977).

A coastal structure may experience forces from three types of waves: nonbreaking, breaking and broken. Where the wave height is not limited by shallow depths, a nonbreaking condition exists. The force due to nonbreaking waves is essentially hydrostatic. Waves breaking directly against the structure impose the most severe forces, an added hydrostatic force coupled with a short duration dynamic pressure that acts near the region where the crests hit the structure. Broken waves occur in somewhat shallower water and do not exert significant design forces.

In rubble mound design, the design wave height is a critical parameter. It is input directly into stability equations (Chapter 7) where it affects, to the third power, armor unit weight. Prediction of wave type and subsequent selection of the design wave height are presented below.

Breaking Waves. Waves may break by spilling, plunging, collapsing or surging (Figure 5.1), and each type imposes different pressures on a nearshore structure. Spilling and surging waves exert only an added hydrostatic pressure, while plunging waves can create a dynamic shock pressure. It is important to estimate the breaker type of the design wave, since it is more critical to design against the plunging wave than the spilling or surging one (Galvin, 1969). All of the limited design data available regarding the effect of breakers on rubble mound stability relate to the plunging wave condition. Breaker classification methods are outlined by Hubbell and Kulhawy (1979b).

A common approximation is that waves will break on a structure that has a water depth at the toe, d_s, of less than 1.3 times the design wave height. This d_s guideline is not always valid, however, and should not be used for design purposes (CERC, 1977). A wave which plunges on a coastal structure actually initiates breaking at some depth, d_b, seaward of the structure toe. This wave, which travels to the structure during the breaking process, will be larger than that predicted with consideration only of d_s (Weggel, 1972). Therefore, design wave heights must be evaluated with reference to d_b rather than d_s.

The horizontal travel of plunging waves during breaking was investigated by Galvin (1969). Parameters of breaker geometry and

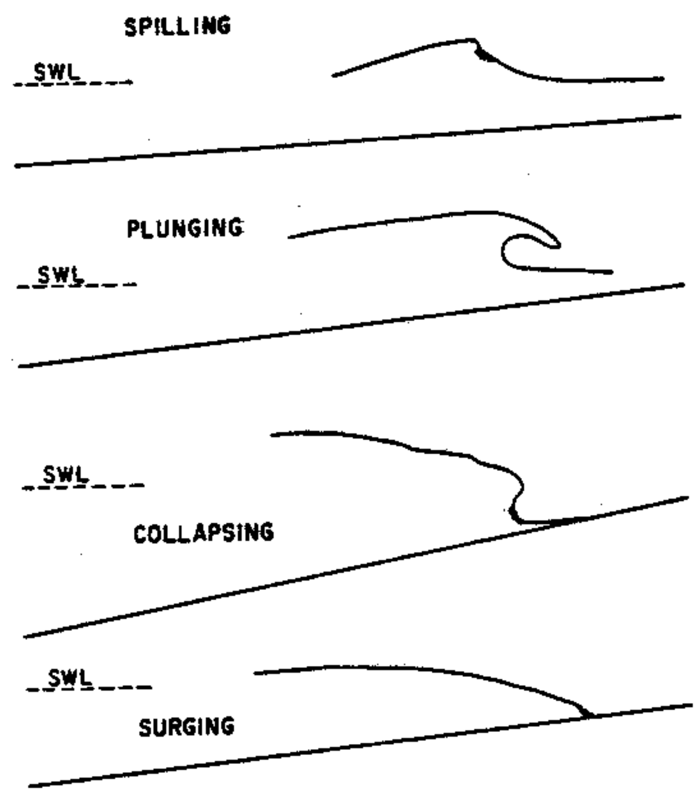


Figure 5.1 Breaker Types (Galvin, 1969, p. 178)

travel are defined in Figure 5.2. The distance a breaker travels before collapsing, X_p, is a function of the nearshore slope, m, and the breaker height, H_b:

$$X_p = (4.0-9.25m)\ H_b \tag{5.1}$$

The travel distance, X_p, delineates the zone of influence of a breaking wave for various still water levels (Galvin, 1969).

It is desirable to determine the maximum breaker height a coastal structure could reasonably experience. Figure 5.3 or 5.4 can be used to evaluate the design breaker height, H_b, depending on the known parameters. The nearshore slope, m, and d_s are obtained at the site; the wave period and deepwater wave height are predicted as described in Hubbell and Kulhawy (1979b). The use of these graphs is illustrated by Design Example 5.1.

For a particular still water level, the limiting depths for wave breaking are defined as:

$$d_b(\max) = \alpha H_b \tag{5.2}$$

$$d_b(\min) = \beta H_b \tag{5.3}$$

Figure 5.5 is used to evaluate and and these are, in turn, used to calculate the minimum and maximum breaking depths, as demonstrated in Design Example 5.1.

The evaluation of the breaker travel distance, X_p, and the limiting breaker depths, d_b(min) and d_b(max), defines a region that will be subject to breaking waves for a given still water level. In general, structures located in depths greater than d_b(max) will experience nonbreaking wave forces. Conversely, broken waves will impinge on structures built in depths shallower than d_b(min).

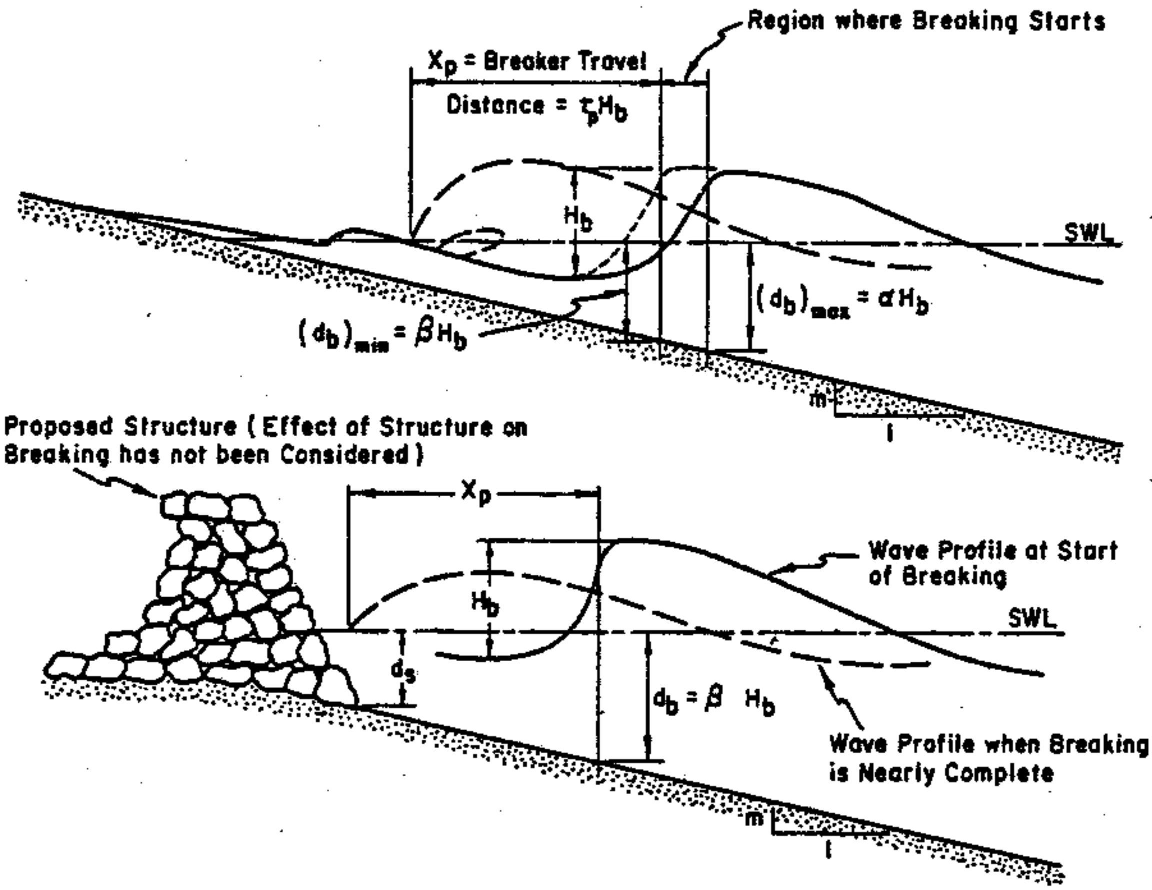


Figure 5.2 Definition of Breaker Geometry (CERC, 1977, p. 7-4)

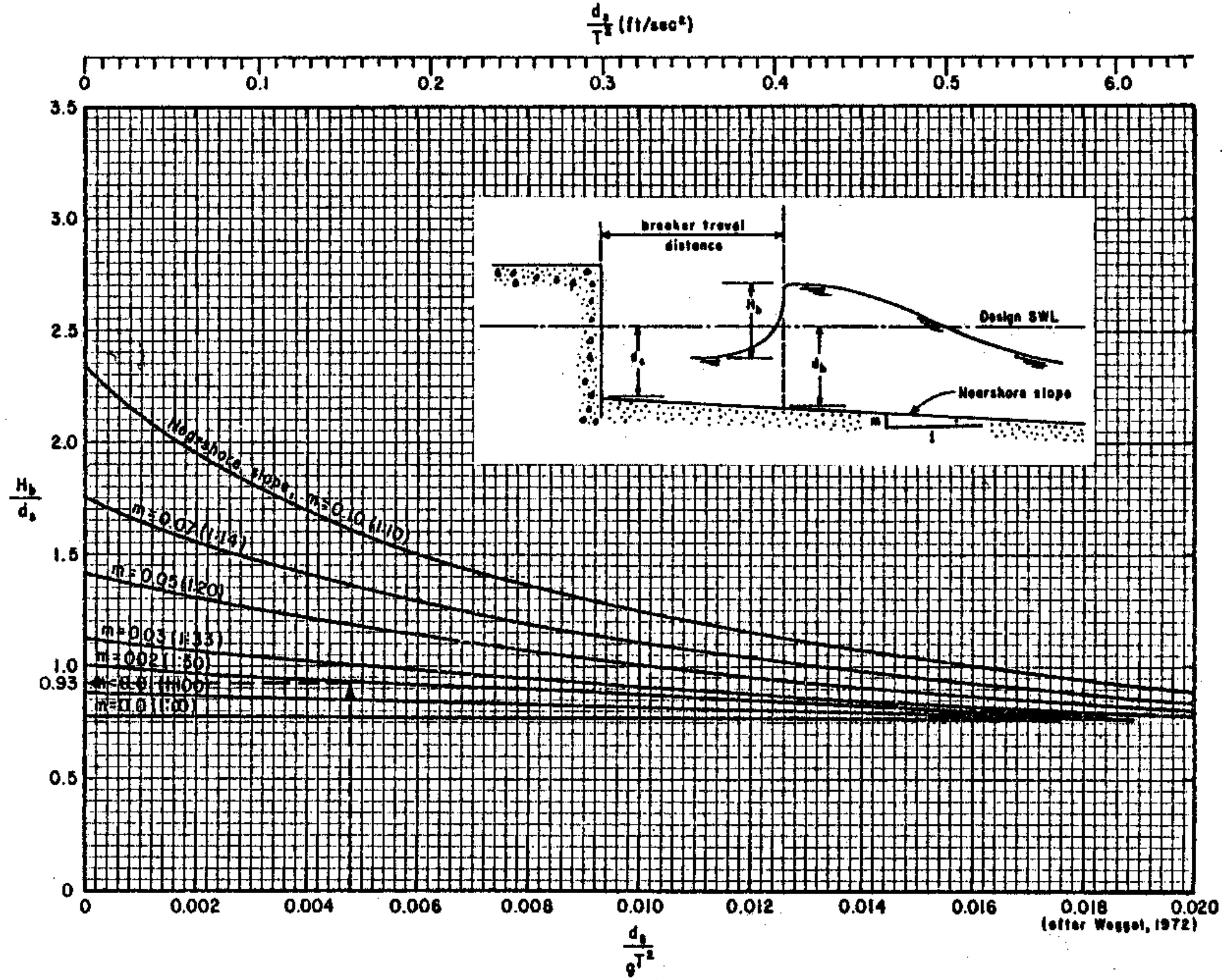


Figure 5.3 Dimensionless Design Breaker Height versus Relative Depth at Structure (CERC, 1977, p. 7-9 after Weggel, 1972, p. 427)

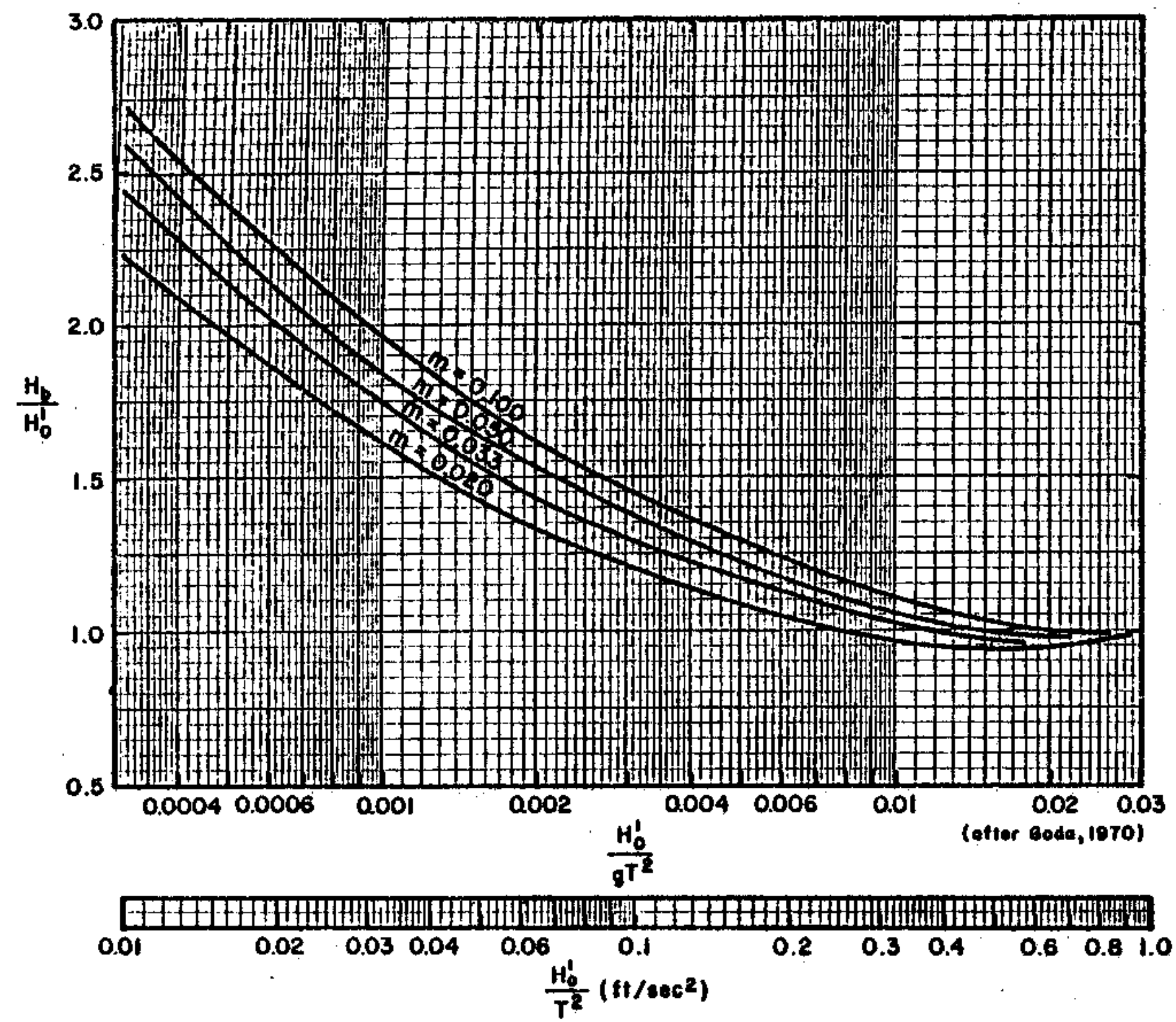


Figure 5.4 Breaker Height Index versus Deepwater Wave Steepness (CERC, 1977, p. 7-7)

DESIGN EXAMPLE 5.1

DETERMINATION OF BREAKER CHARACTERISTICS

GIVEN : DESIGN DEPTH AT STRUCTURE TOE , $d_s = 10.0$ FT
BEACH SLOPE , $m = 0.02$ $(1:50)$
DESIGN WAVE PERIOD , $T = 8$ SEC

REQD : a) MAXIMUM BREAKER HEIGHT, H_b
b) LIMITING BREAKER DEPTHS, d_b (MIN) AND d_b (MAX)

SOLUTION :

a) $$\frac{d_s}{gT^2} = \frac{10}{(32.2)(8^2)} = 0.00485$$

ENTER FIGURE 5.3 TO THE CURVE FOR $m = 0.02$,

READ $\dfrac{H_b}{d_s} = 0.93$

$$H_b = 0.93\, d_s = (0.93)(10) \quad \therefore \underline{H_b = 9.3 \text{ FT}}$$

BREAKERS LARGER THAN 9.3 FT WILL BREAK FARTHER OFFSHORE FROM THE STRUCTURE AND WILL DISSIPATE MUCH OF THEIR ENERGY BEFORE REACHING THE STRUCTURE. BREAKERS SMALLER THAN H_b MAY BREAK DIRECTLY ON THE STRUCTURE, BUT WILL NOT ESTABLISH A CRITICAL DESIGN CONDITION.

b) $$\frac{H_b}{gT^2} = \frac{9.3}{(32.2)(8^2)} = 0.0045$$

ENTER FIGURE 5.5 TO THE β CURVE FOR $m = 0.02$,

READ $\beta = 1.15$
$\alpha = 1.52$

THEN,

$$d_b\ (MIN) = \beta H_b = (1.15)(9.3) = \underline{\underline{10.7\ FT = d_b\ (MIN)}}$$

$$d_b\ (MAX) = \alpha H_b = (1.52)(9.3) = \underline{\underline{14.1\ FT = d_b\ (MAX)}}$$

THE CALCULATED DESIGN WAVE WILL INITIATE BREAKING IN DEPTHS BETWEEN 10.7 AND 14.1 FT.

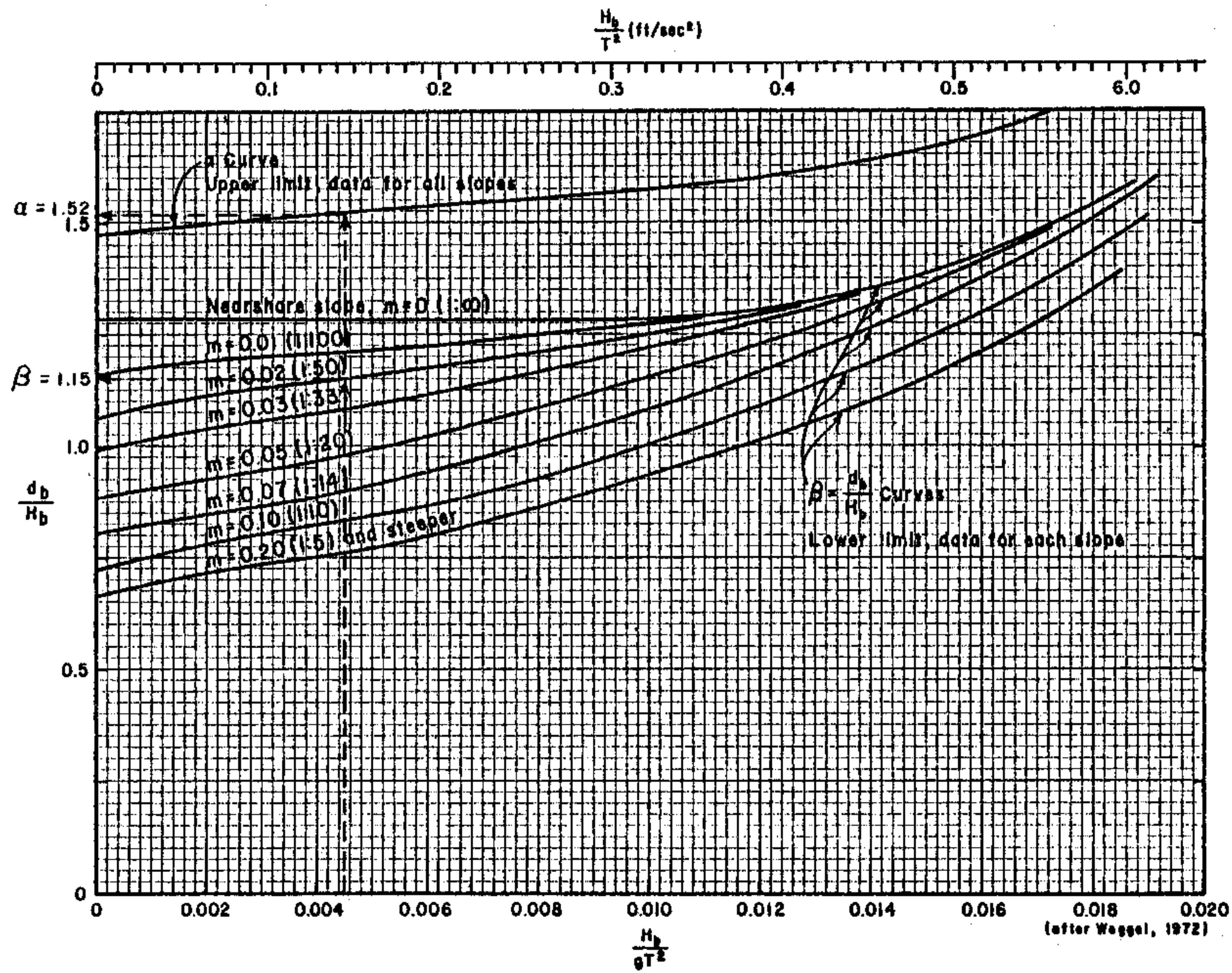


Figure 5.5 α and β versus $H_{b/gT}{}^2$ (CERC, 1977, p. 7-6)

The breaking process will be modified by proposed structures located in the nearshore zone. Where the effect of the structure is not significant, incident waves will generally break when the depth slightly exceeds d_b(min) (CERC, 1977). Modification of breaker location and height by the presence of rubble mound structures was studied by Jackson (1968b). As wave reflection effects of the structures become more significant, the depth of breaking increases and the zone of breaking translates seaward. Further research is necessary to fully explain the influence of structures.

The foregoing analysis results in a design breaker height from known deepwater wave characteristics. The problem might be approached from the opposite angle, i.e., the maximum breaker height is the known parameter. The deepwater wave height that results in a known breaker height can be established using Figure 5.6 and refraction data for the site (See Hubbell and Kulhawy, 1979b, on wave refraction). Design Example 5.2 applies this method.

Nonbreaking Waves. Nonbreaking waves occur against a structure when the toe water depth, d_s, is greater than about 1.5 H_i, the incident wave height. This wave form is essentially a wave of oscillation, which breaks when the forward velocity of the crest particles exceeds the velocity of propagation of the wave itself. Nonbreaking wave forces are the longest duration wave load, although the peak nonbreaking force is less than that of the breaking wave.

The bottom slope influences the occurrence of nonbreaking waves. As the slope steepens, the limiting depth for breaking decreases and structures can be designed for the nonbreaking condition in shallower waters. The upper limiting envelope of Figure 5.5, the α curve, yields

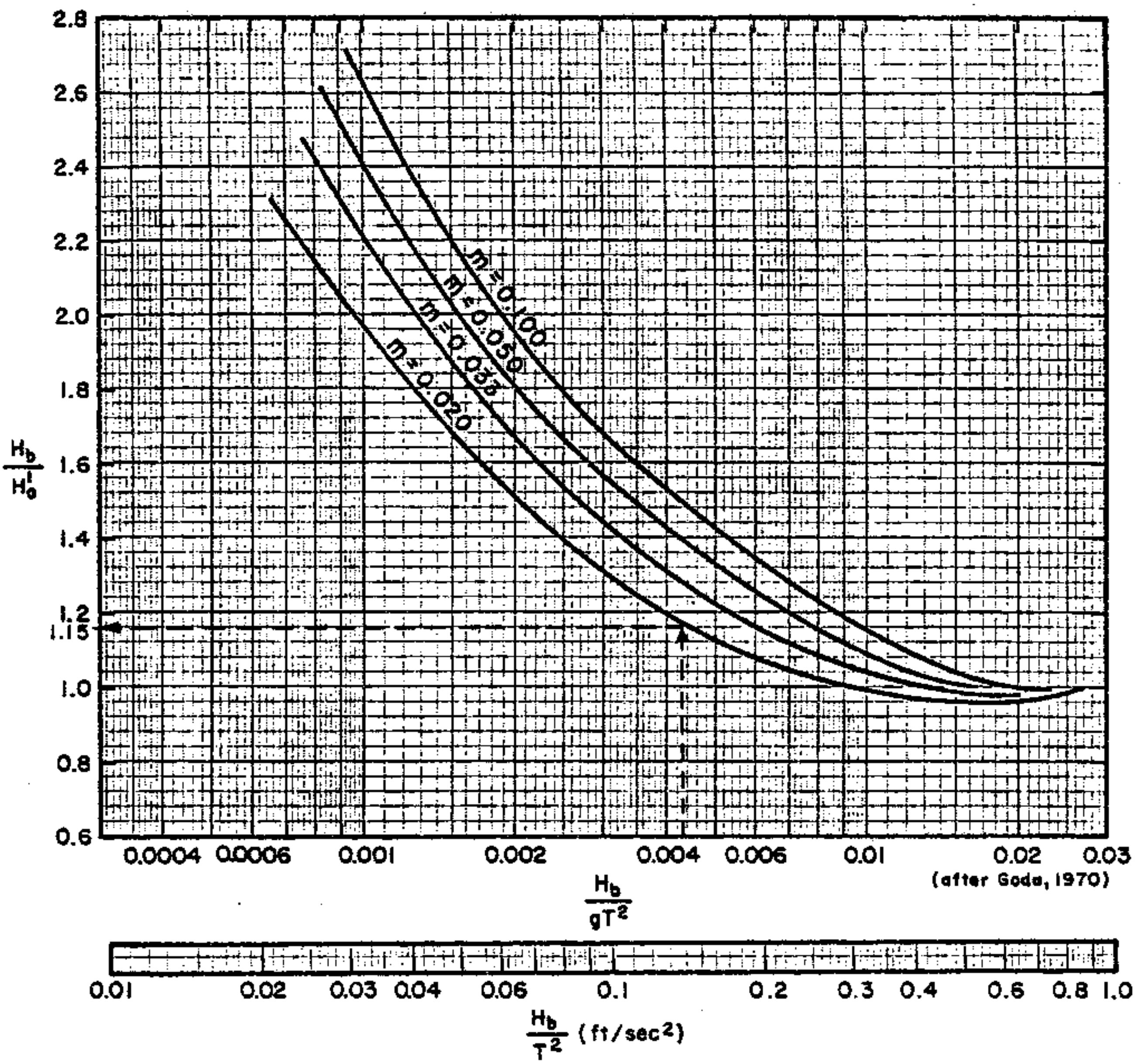


Figure 5.6 Breaker Height Index versus H_b/gT^2 (CERC, 1977, p. 7-11)

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DESIGN EXAMPLE 5.2

DETERMINATION OF DEEPWATER WAVE HEIGHT

GIVEN : $H_b = 9.3$ FT
$T = 8$ SEC
$m = 0.02$ (FROM DESIGN EXAMPLE 5.1)
REFRACTION COEFFICIENT, $K_R = 0.90$ FOR A SPECIFIED DEEPWATER DIRECTION OF WAVE APPROACH (SEE HUBBELL AND KULHAWY, 1979b)

REQD : DEEPWATER WAVE HEIGHT, H_0, OF THE WAVES WHICH RESULT IN THE GIVEN BREAKER HEIGHT, H_b

SOLUTION :

$$\frac{H_b}{gT^2} = \frac{9.3}{(32.2)(8^2)} = 0.0045$$

ENTER FIGURE 5.6 TO THE CURVE FOR $m = 0.02$,

READ $\frac{H_b}{H_0'} = 1.15$

THEN $H_0' = \frac{H_b}{1.15} = \frac{9.3}{1.15} = 8.1$ FT

H_0' IS THE UNREFRACTED DEEPWATER WAVE HEIGHT.

$$H_0 = \frac{H_0'}{K_R} = \frac{8.1}{0.9} \quad \therefore \quad \underline{H_0 = 9.0 \text{ FT}}$$

H_0 IS THE ACTUAL DEEPWATER WAVE HEIGHT. THUS, A 9.0 FT DEEPWATER WAVE WITH $T = 8$ SEC, ADVANCING FROM THE ANALYZED DIRECTION OVER $m = 0.02$, WILL RESULT IN THE MAXIMUM BREAKER HEIGHT ON THE STRUCTURE.

a conservative estimate of the boundary between nonbreaking and breaking water depths.

Broken Waves. Broken waves occur in relatively shallow waters. They exert low pressures, having lost energy in wave breaking and through bottom friction. Broken waves do not pose significant environmental design loads in rubble mound design.

Selection of the Design Wave. The design wave height is the height of the wave that is potentially most damaging to an economically feasible coastal structure. This is different from, and less than, the maximum wave height. The maximum force wave is generally assumed to be the largest wave breaking directly on the structure or, in the case of nonbreaking waves, the largest wave to reach the structure (Galvin, 1969). The design wave is selected with consideration of the structure use, the frequency of occurrence of the maximum wave, permissible damage to the structure, and economic factors.

For nonbreaking waves, the design height is chosen from a statistical frequency distribution of wave heights from empirical hindcasts (See Hubbell and Kulhawy, 1979b). The distribution is often based on the significant wave heights, H_s, the average of the highest third of the wave heights occurring in a given record. However, depending on the type of structure and the allowable margin of safety, the design may be based on higher heights, as H_{10}, the average of the highest ten percent of the heights. Table 5.1 gives the ratios of commonly used wave height parameters to significant height.

In recommendations of the Corps of Engineers (CERC, 1977), selection of the nonbreaking design wave height depends on whether the structure is rigid, semirigid or flexible, and its corresponding

Table 5.1 Various Design Wave Heights, Related to Significant Height (Quinn, 1972, p. 41)

Significant height (H_s)	1.0
Average height	0.6
Average height of highest 10% (H_{10})	1.27
Height not exceeded more than 20% of the time	0.9
Height not exceeded more than 10% of the time	1.1
Height not exceeded more than 5% of the time	1.2
Height not exceeded more than 3% of the time	1.3
Height not exceeded more than 1% of the time	1.6
Average height of highest 1% (H_1)	1.7
Maximum height	1.87

response to wave attack. Because rigid structures cannot deform to absorb wave energy, a high wave within the wave train could cause sudden and complete structural failure. These structures are designed by the most stringent design wave specifications. Semirigid types, such as cellular sheet pile configurations, can absorb wave pounding. Damage to flexible rubble mound structures is progressive, and short durations of extreme wave action seldom create serious destruction. They can, accordingly, be designed for statistically lesser wave heights. The CERC guidelines are as follows:

1. Use H_1 for rigid structures.
2. Use H_{10} to H_1 for semirigid structures.
3. Use H_s to H_{10} for flexible structures. The higher (H_{10}) values may be used if the storm frequency is such that extensive annual damage would require costly continual maintenance.

In calculating the design wave height for breaking conditions, the significant and related wave heights are not directly considered, because larger waves break seaward of the structure and smaller waves are ignored. For sites on the open coasts in shallow depths ($d \leq 10$ ft or 3 m) or where it is necessary to design against the absolute maximum possible wave height, the calculated maximum breaker height is adopted as the design wave height. For other sites, if the maximum breaker height does not occur with a great frequency, a lesser wave height from the frequency distribution is selected for design purposes (Galvin, 1969).

Design depths and wave conditions can usually be evaluated concurrently. The parameters must be coupled in structural design as they are likely to occur simultaneously at the site. For example, high water levels generated by hurricane storm surge and the associated extreme wave action occur together, and usually provide maximum design criteria. The frequency of occurrence and duration of combinations of water level and wave action are important considerations in the design of proposed shore protection schemes (CERC, 1977).

Currents

In the structural analysis of rubble mounds, the forces of surface wave related currents do not add significant structural design loads. Because they are responsible for littoral drift, the influence of currents is, instead, on the functional design of shore protection structures, e.g. length, spacing, orientation, etc. (Chapter 3). The role of currents in setting up nearshore circulation cells and longshore littoral transport is examined in Chapter 2. Scouring forces resulting from currents are also quite important. Their effects are highlighted below and discussed more fully in Section 5.2.

Rip currents are strong, narrow currents that flow seaward from the surf zone. Once a rip current is established, the increased water velocity can remove material from its path in scoured channels. If rips occur along the sides of a shore-connected structure, as a groin or jetty, severe undermining can result in flanking and toe failure. For this reason, the spacing of rip currents, usually one to eight times the width of the surf zone (Table 2.1), should be considered in planning nearshore structures (Inman, Tait and Nordstrom, 1971).

Soil Stresses

Shore-connected breakwaters, groins and jetties are, in a sense, retaining walls which retain the littoral drift they accrete. The stability of wall type structures (Chapter 4) depends in part on the soil stresses on the updrift side which tend to overturn the wall, and those on the downdrift side which provide a restraining moment. These forces are not explicitly considered in the design of mound structures.

Soil stress is a maximum when the wall structure is filled on one side and empty on the other. Similarly, maximum soil forces occur at the shoreward end, where more littoral material is retained (Figure 5.7). Toward the seaward end, progressively less material is accreted, and soil stresses are decreased. Finally, at the seaward end, the accretion wedge ideally tapers to meet the downdrift profile.

Because soil forces vary along the length of a shore protection structure, analysis as a retaining wall is complex. The problem is complicated further by wave loading which also varies with location along the wall. Evaluation of these coupled loadings on wall structures is beyond the scope of this chapter. Saczynski and Kulhawy (in preparation) and basic texts on soil mechanics and foundation engineering cover the general theories for soil stresses. Geotechnical considerations related to rubble mound foundation design are dealt with in Section 5.2.

Shock Pressures

Waves breaking against coastal structures induce impact pressures of very high intensity and short duration, called shock pressures. These are followed by longer duration pressures of lower intensity (Figure 5.8). Shock pressures can also be caused by the slamming of ships and other solid objects.

Studies by Kamel (1968) resulted in the following conclusions regarding shock pressures. Because the pressures have short duration,

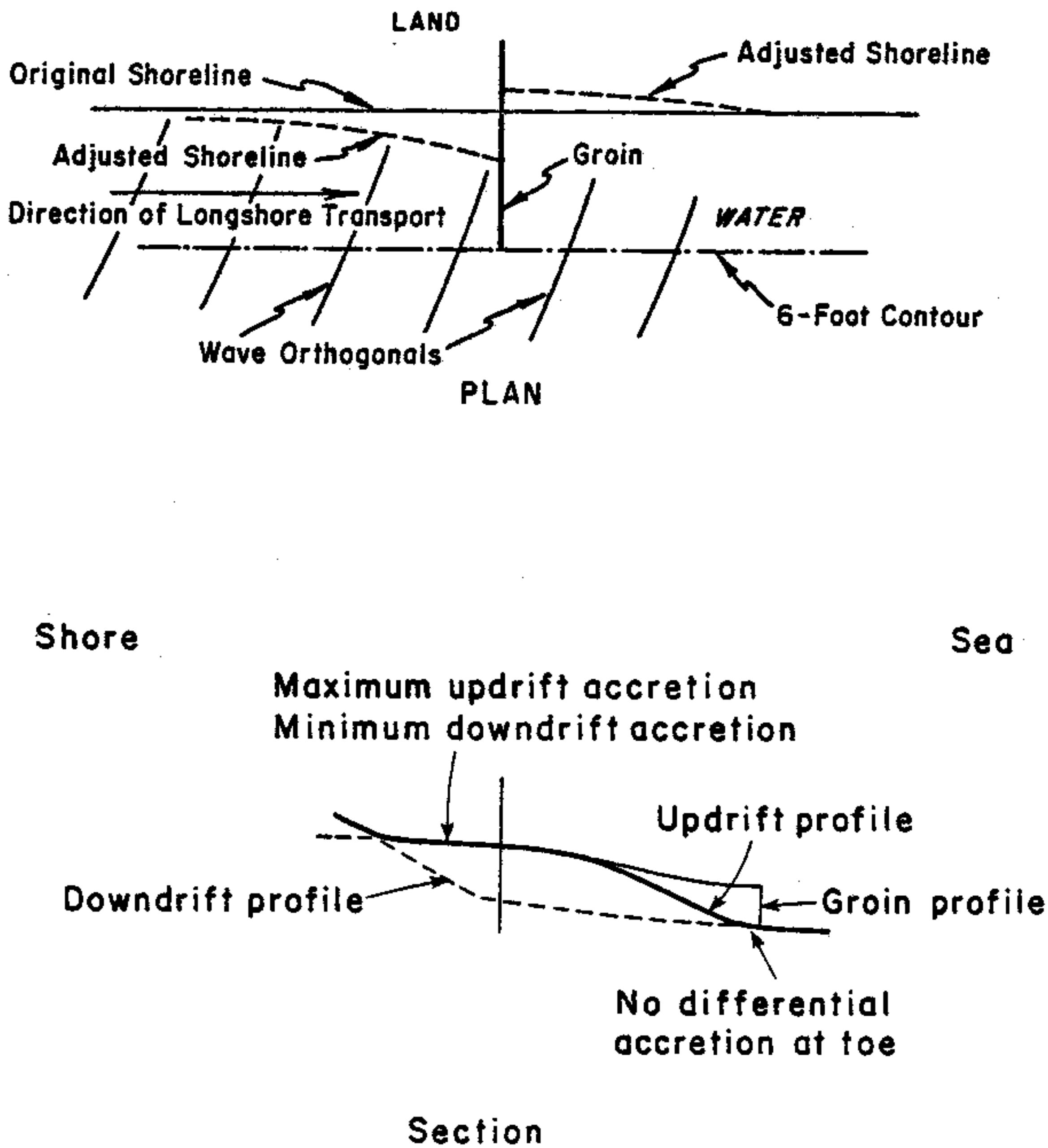


Figure 5.7 Soil Forces on a Groin Profile (after CERC, 1977, p. 5-35)

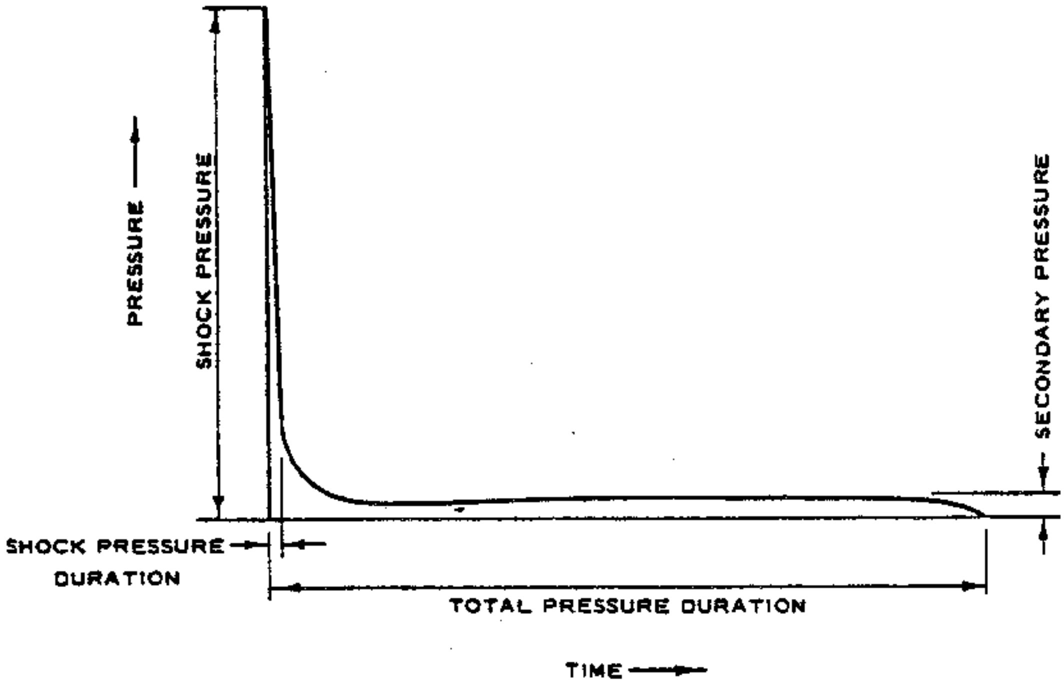


Figure 5.8 Typical Pressure - Time Curve for Waves Breaking on a Structure (Kamel, 1968, p. 19)

on the order of a millisecond, and occur only at some spots on the surface of the structure, they should not be used in checking the structural stability. This is especially true for rubble mounds, because the flexible construction tends to absorb shocks. The short duration loads are more important in the design of rigid structures; the pressures may cause cracks in the casings of rock-filled steel caissons, for example. Also, they affect the stability of structures that have natural frequencies within the range of duration of shock pressures.

Ice

In cold regions marine structures can be affected significantly by ice problems. Because rubble mounds depend for their stability on the cover layer armor units, ice forces which tend to dislocate or damage these units are most destructive.

Wave and wind driven ice fragments impacting a rubble mound have the capacity to dislodge armor units from the face. The mound is a flexible structure and can rearrange to survive the loss of some units; in the extreme, however, underlayers and core material could be exposed and eroded. This damage would almost certainly not cause structural failure of the mound, but would necessitate prompt repair. Well-keyed rubble elements endure ice impact best. It has been observed that damage is less extensive during severe winters, when piled ice acts as a buffer to impact (Carey, Ashton and Frankenstein, 1973).

The expansion of water to ice can produce excessive pressure in the voids of a rubble mound. This internal static ice stress might act to jack cover layer units from their places on the mound face. According to CERC (1977) the probable maximum pressure that can be generated by water freezing in an enclosed area is 30,000 psi (207 MN/m^2).

Degradation of the cover layer materials themselves can occur. Moving ice floes may significantly abrade rock and concrete armor units at the water level. Frost action and freeze-thaw cycling can totally destroy the fabric of susceptible rocks. An evaluation of the freeze-thaw durability of prospective armor unit materials is imperative in prone regions. Appropriate procedures are described in Chapter 6.

Ice forces do not generally impose greater stresses than the wave forces which rubble mounds are designed to resist. Because the maximum wave forces and ice thrust do not occur in combination, no special design allowance is commonly made for periodic ice conditions. Should it be necessary to quantify ice forces, Hubbell and Kulhawy (1979b) provide the required procedures.

Earthquakes and Tsunamis

There is little documented literature concerning rubble mound design for earthquakes. Some elucidation is provided by Wang, et. al. (1978) who studied the reaction of rubble mound breakwaters to earthquake ground motion. Model tests were performed in an attempt to reproduce the behavior of the structures under earthquake loading, to identify failure modes and to examine the stability of armor units. The following experimental conclusions apply to rubble mound breakwaters on a rigid foundation; possible foundation failure was not studied.

The fundamental damage mode exhibited was crest settlement and some slight slope deformation. Slope steepness was an influential factor; crest settlements and horizontal toe displacements were greater on the steeper (leeward) sides of the model breakwaters, as demonstrated in Figure 5.9. Under severe or repeated shocks, the dolosse mat settled as a whole down the face, causing thinning or rifting at the crest. In general, however, the two layers of armor units remained intact.

The percentage change in crest elevation, the primary damage indicator, is plotted as a function of horizontal earthquake acceleration in Figure 5.10. The intersection of envelope A-A, the zero damage line, and the acceleration scale occurs at about 0.4g. This indicates that earthquakes of less than 0.5g have no significant effect on rubble mound breakwaters. Clough and Pirtz (1958) discovered the same lower limit for earthquake damage to rock-fill dams. The high degree of resistance is attributed to the structural flexibility inherent in the rubble mound configuration. The interlocking of armor units is also perceived as important to earthquake resistance.

Presettled breakwaters were less susceptible to moderate shock damage than new ones (See envelope B-B in Figure 5.10). In the moderate case, settlement is due largely to internal densification. For larger ground accelerations, the advantage of presettlement decreased. For strong shocks, crest settlement is caused by a change in structural shape and modification of side slopes.

For rubble mound breakwaters which protect critical facilities, as offshore deep water ports, refineries and power plants, seismic effects cannot be ignored. The possible crest settlement resulting from the design earthquake can be approximated, and subsequently allowed for, from Figure 5.10, using envelope A-A for new breakwaters with little shakedown and line B-B for older breakwaters with more than four percent presettlement.

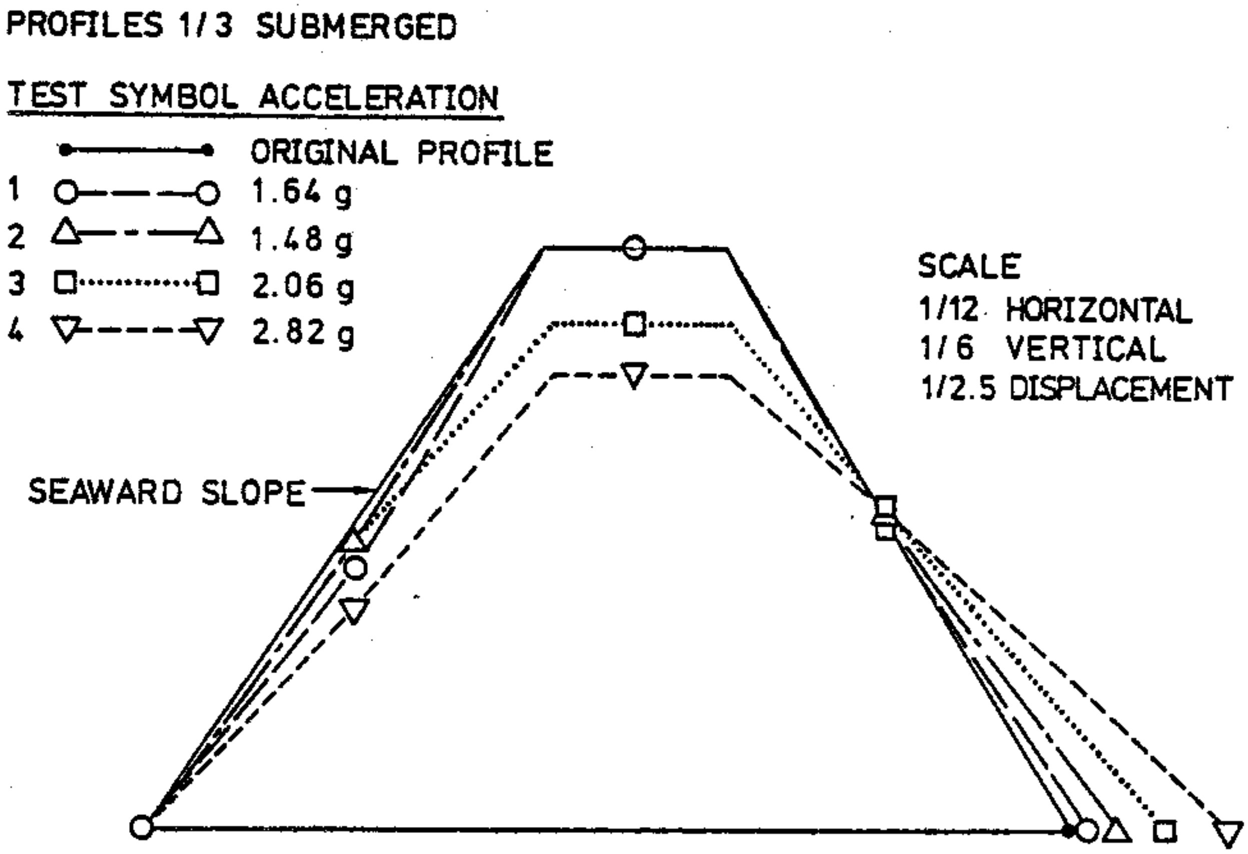
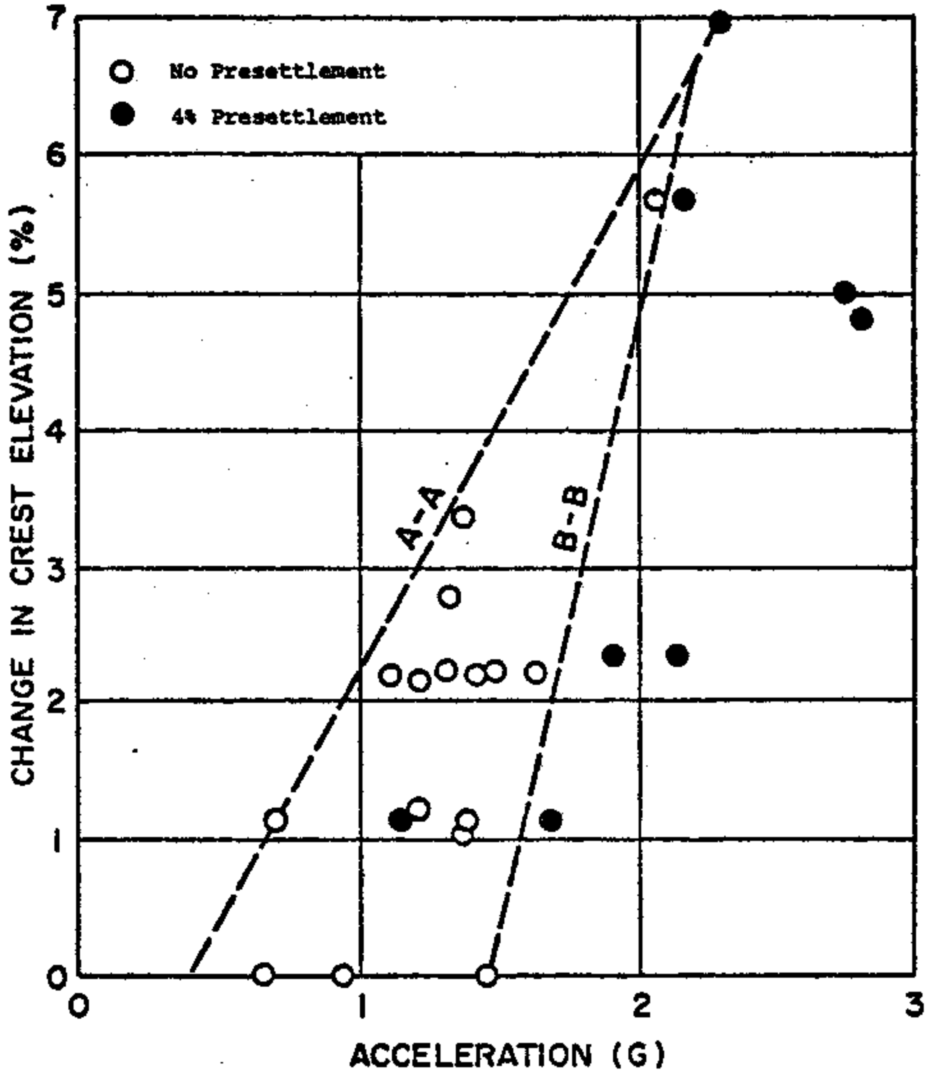


Figure 5.9 Breakwater Profile Changes due to Earthquakes
(Wang, et. al., 1978, p. 2252)



A-A DESIGN CURVE: No presettlement
B-B DESIGN CURVE: 4% presettlement

Figure 5.10 Crest Elevation Change versus Earthquake Acceleration; the Effect of Presettlement on Breakwater Damage (Wang, et. al., 1978, p. 2255)

For small-scale structures, costly fortification against earthquakes is seldom economically justified. Damaged breakwaters might still offer some protection and rubble mound repair may prove less expensive than initially overdesigning the crest elevation. The controlling factors in incorporation of earthquake design, then, are the financial and environmental consequences of failure of the protective elements (Wang, et. al., 1978).

Tsunamis. Tsunamis are long period waves produced by undersea earthquakes and related landslides, bottom slumping and volcanic eruptions. They are generated by a rapid large-scale disturbance of a mass of ocean water that results in displacement of the ocean surface and the creation of waves. The direct attack and associated onshore runup of high waves has caused several major catastrophies in coastal areas prone to tsunamis. Most tsunamis are generated at the active earthquake boundary along the rim of the Pacific Ocean (Aleutian Islands, Japan, New Zealand, and the west coast of South America) and weaker tsunamis have been recorded in other parts of the world (Sorenson, 1978).

Most onshore structural damage is caused by waves, flooding, high flow velocities in the runup surge, and impact of objects carried by the surge (Sorenson, 1978). Massive tsunami barriers can be constructed, although such measures are extremely expensive. It is unrealistic, however, to believe that even rigorously designed protective measures can totally counter the effects of tsunami attack. The establishment of a warning system, escape routes and evacuation training are well-advised tsunami countermeasures (Horikawa, 1978).

5.2 GEOTECHNICAL CONSIDERATIONS

Foundation conditions have a profound influence on selection of the structural type to be used. Rubble mounds are an attractive alternative in this respect as they are adaptable to a wide range of soil conditions and water depths. Too often, however, rubble mound design proceeds with little attention to geotechnical aspects, when the foundations for marine structures warrant careful study.

Nearshore soil deposits usually comprise sand and clay, in alternating layers of variable thickness, with varying silt contents. Geotechnical problems vary accordingly; failure of the mound-foundation complex could be initiated by excessive settlement, insufficient soil bearing capacity, critical toe scour, or a combination of these. Possible instability from these sources should be evaluated. If judged necessary, the foundation design can incorporate measures to reduce the potential for damage and failure. The scope of geotechnical investigation and analysis is dictated primarily by the scale and purpose of the structure. Specification of acceptable factors of safety is related similarly to the nature of the project, and is the responsibility of the design engineers and the authorizing and regulatory agencies.

It is common practice, on projects of all scales, to place one or more bedding layers over the foundation soils to act as a base for the overlying rubble mound. Properly designed and installed bedding mats can provide scour erosion control and counter settlement and bearing capacity failure. The design of these important foundation elements is presented as the last part of this section.

Settlement

Rubble mounds can settle into their sandy foundations because of removal of the bottom supporting material, as by scour. When the mound is founded on soft compressible clay strata, consolidation of this material can cause large-scale settlement of the structure, as shown in Figure 5.11a. These two settlement mechanisms are addressed below.

Settlement of toe stones into scour holes and undermined trenches is the net result of wave and current turbulence on the sediment bed. This interaction and the structural problems it can precipitate are included in the discussion on scour. The best measure against this mode of settlement is the use of a properly designed and installed foundation filter layer. Filter design guidelines are presented in the final portion of this section. In the absence of a filter layer, rubble stones will settle to a depth at which the sand is not disturbed by bottom current effects. Large quantities of rubble may be required to allow for the settlement loss. This can, however, provide an effective and stable base for construction (CERC, 1977).

Many marine clays and silts are highly compressible as well as weak in shear resistance. These foundation materials may consolidate and induce large and detrimental deformations in the overlying structure. It may be possible to consolidate the soft material by dumping rock until a stabilized base has been built. This fabricated foundation should be permitted to settle under its own weight for a period of time prior to rubble mound construction. In lieu of this, excavation and replacement of the clay with material more competent to receive the structural loads will prevent large settlements (Quinn, 1972).

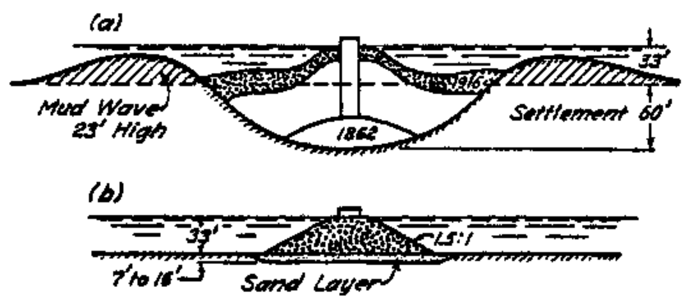


Figure 5.11 Rock-fill Breakwater, Spezia, Italy, Constructed a) By dumping rock directly onto clay, b) By dumping rock onto sand layer in a shallow dredged cut (Terzaghi and Peck, 1967, p. 465 after Barbaris, 1935)

A case history reported by Terzaghi and Peck (1967) demonstrates the effectiveness of settlement control by a shallow dredged cut filled with sand. A harbor breakwater installed in Spezia, Italy, 1862, comprised a rock-fill foundation. The site water depth of 33 ft (10 m) was underlain by soft clay. Construction commenced with dumping large rocks into the water, as was the standard practice of the time. This procedure destroyed the structure of the uppermost layer of clay and induced very large local stress concentrations in the material. The settlement of the fill was correspondingly large. The addition of more material to maintain a constant crest elevation simply accelerated the rapid settlement. During a period of 50 years the material added was equivalent to a layer 60 ft (18 m) thick (Figure 5.11a). When a new breakwater section was added in 1912, measures were taken to prevent excessive settlement. The mud was removed to a depth of between 7 and 16 ft (2.1 and 4.9 m) and replaced with sand. The fill rocks were now supported by the sand and no local stress concentrations were developed in the clay (Figure 5.11b). After nine years, the settlement reached only 2.7 ft (0.8 m).

The rubble mound structure is highly flexible and can rearrange and internally adjust to some foundation settlement. This is a useful advantage. It may be practical simply to allow the mound to deform in response to settlement rather than take precautionary measures. Similarly, the mound can be overbuilt initially to allow for anticipated foundation settlement. In the final configuration the structure will have deformed to the design crest elevation.

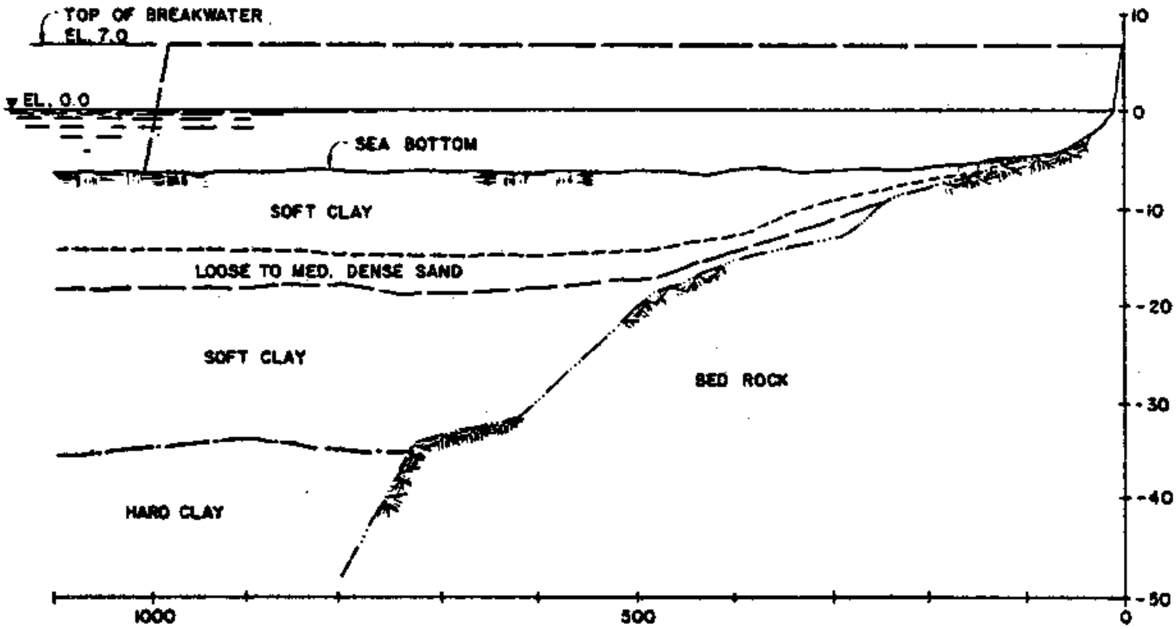
The foundation design for an LNG port breakwater, Malaysia, incorporated the principles of material replacement and overbuilding.

The subsurface profile (Figure 5.12a) is characterized by interbedded very soft to stiff marine clays and silts, underlain by the sandstone bedrock. It was judged that these soils were too compressible to support the breakwater directly. The recommended foundation scheme involves excavation of the unsuitable soil to bedrock or the top of the overconsolidated clays, and replacement with fine sands dredged from the inner harbor (Figure 5.12b). The sands will be quick dumped from bottom dump barges and vibrated into place to a uniform density. The mound head final elevation is +8.0 m (+26.2 ft); it will be built to + 9.8m (+32.1 ft) to allow for settlement. This design is quite conservative, owing to the critical nature of the LNG berth (Cameron and Lin, 1980).

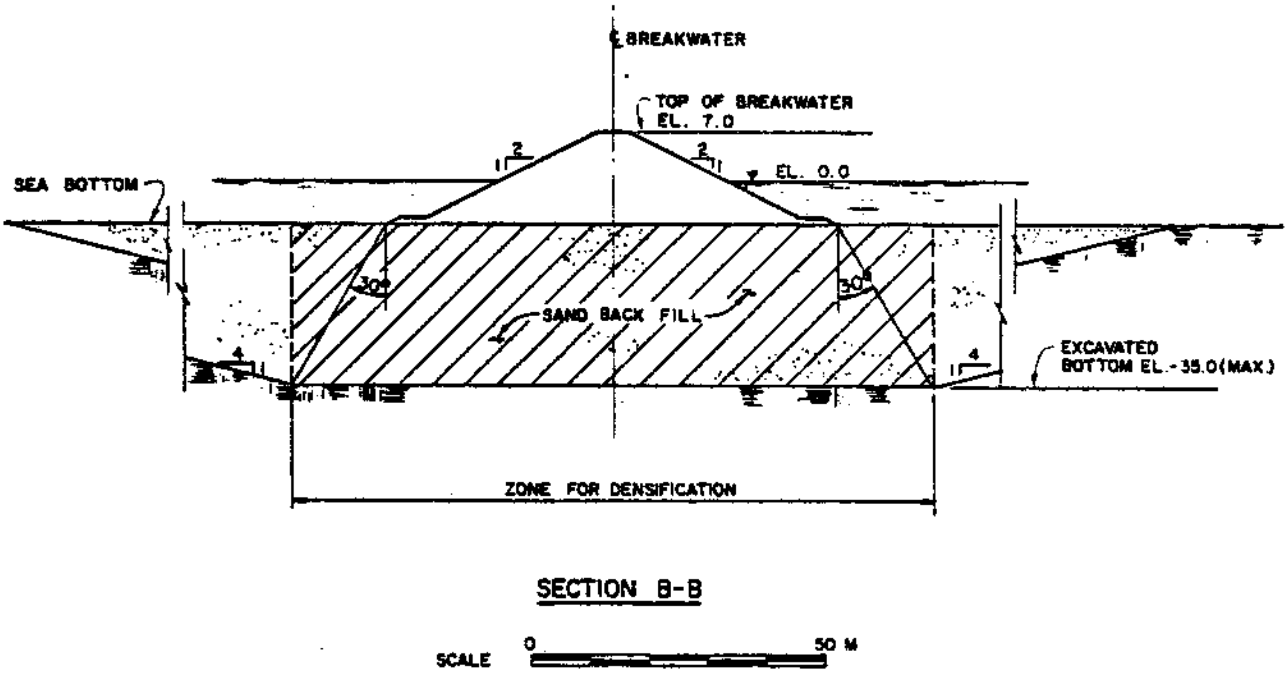
Bearing Capacity

When the load placed exceeds the bearing strength of the soil, the material can fail by shearing along a curved plane, cutting the bottom at some distance beyond the toe of the superimposed load. A layer of material stronger than the base soil, laid beyond the toe and the anticipated plane of failure, will reduce the potential for bearing capacity failure. The weight and increased shearing resistance of a properly designed foundation mat are sufficient to resist upheaval of the soil beyond the toe. Quinn (1972) suggests that the thickness and shear strength of the base should be specified to provide a factor of safety of at least 1.5 against bearing capacity failure at the toe, and that the layer extend out such that the critical plane of failure will have to pass through its base.

The evaluation of rubble mound slope stability requires a detailed geotechnical analysis. Usually, for slope stability studies, various



a. Subsurface profile



b. Proposed breakwater foundation scheme

Figure 5.12 LNG Breakwater, Bintulu, Malaysia (Cameron and Lin, 1980, pp. 463-464) circular and wedge failure surfaces are analyzed until a critical one is found. It is beyond the scope of this study to present analysis methods; slope stability theory and procedures are well-documented in soil mechanics texts and literature.

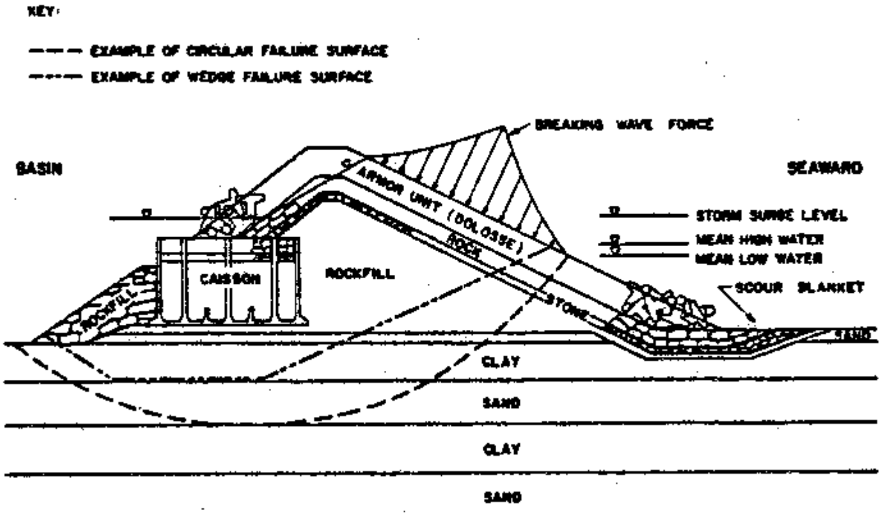
According to Fischer and Lu (1975), breakwater slope stability should be evaluated for two critical conditions:

1. Maximum breaking wave acting on the breakwater embankment in combination with the maximum storm surge.
2. Rapid drawdown - the water on the leeward side is at its maximum level, followed immediately by the retreat of the wave to its lowest level on the seaward side.

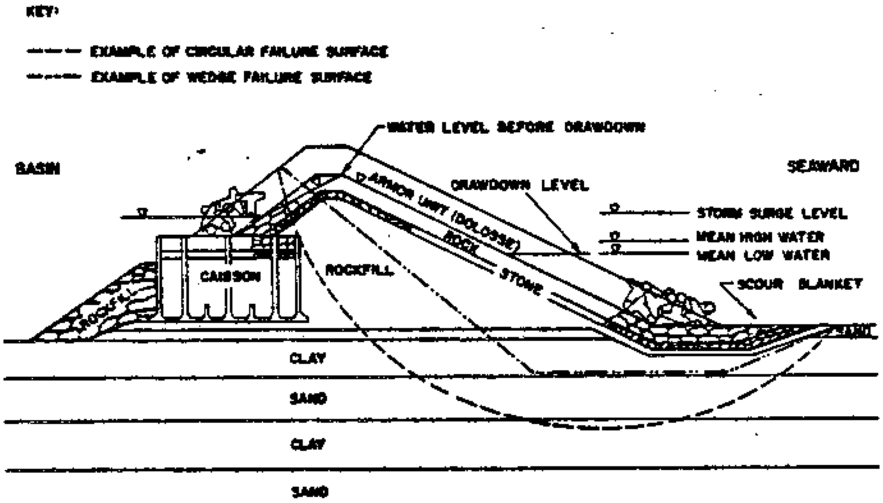
The first case has not been addressed in breakwater design. As shown in Figure 5.13a, the breaking wave force will increase the tendency of failure toward the breakwater inner slope while enhancing the stability of the seaward face. A rapid drawdown condition exists when no significant drainage occurs during drawdown. Because rubble mounds are pervious and do not retain water, the possibility of true rapid drawdown is remote. However, Fischer and Lu (1975) recommend this case be considered in the conservative design of major breakwaters under storm wave action. The stability of the seaward slope will be more critical than that of the inner slope under rapid drawdown, as shown in Figure 5.13b.

<u>Scour</u>

Rubble mounds may be subject to severe toe erosion by undermining and scour. Scour is a process of removal of materials on the sediment bed. Waves and turbulence place the natural bottom materials in suspension where they can then be transported away by longshore and



a. Breaking wave condition



b. Rapid drawdown condition

Figure 5.13 Slope Stability Analyses for Rubble Mound Breakwaters (Fischer and Lu, 1975, p. 590) other currents. The removed material is often not replaced by an influx of sediments and a scour hole or trough develops along the toe of the mound structure.

Scour-induced damage or failure of the rubble mound toe can seriously threaten overall structural stability and functional adequacy of the works. If the toe stones are dislodged, the armor units above them can slide down the face slope. Crevices opened in the armor layer render the underlayers and core vulnerable to wave attack. In this unshielded condition, a rubble mound could be totally and catastrophically breached by severe storm waves. Failure could, instead, progress over a long period of time. Armor stone has been observed to bridge over cavities up to 20 ft (6 m) in diameter. In any case, once a crevice is initiated in the armor layer, the stability afforded by keying and interlocking of units is lost, and the overall mound stability can be reduced by as much as 50 percent (Sullivan, 1979).

Foundation scour has significantly affected the economics and service life of many existing coastal structures. Scour problems encountered during construction can lead to substantial cost overruns, as indicated in Table 5.2. For example, the Cape Hatteras groin field, in North Carolina, has been plagued with problems since its construction in 1970, as described by Machemehl (1979). The concrete and steel sheet pile groins were undermined in deep scour pockets. The loss of bottom support allowed their deflection and toppling under wave action. Major repairs are required to restore the groins to a functional level. Another case is documented by Sullivan (1979). Apparent displacement or settlement of the small foundation stone was partly responsible for the eventual major breaching of the east Kahului breakwater, Hawaii, in

Project	Estimated Cost due to Scour	Estimated Percent Increase due to Scour	Price Level	Estimated Cost due to Scour in 1979 Dollars*
Murrells Inlet South Jetty	$ 300,000**	**	1979	$ 300,000
Baptiste Collette Jetties	449,249†	41.0†	1979	449,249
Tiger Pass Jetties	295,838†	28.0†	1979	295,838
Ventura Marina Breakwater	62,159	4.0	1972	102,252
Dana Point Harbor Breakwaters	515,818	6.5	1970	909,387
Ponce de Leon Jetties	1,676,140	73.0	1971	2,837,705
Buffalo Disposal Dike No. 4	699,776	5.0	1977	815,939
Colorado River Jetties, Texas	1,477,433††	25.1††	1977	1,722,687
Destin East Pass Jetties	92,623	13.0	1970	163,294
Tillamook South Jetty	1,844,090‡	21.1‡	1972	3,033,528
Total				$10,629,879

* Based on Consumer Price Index referenced to 1967; The World Almanac and Book of Facts 1979, 1970--0.860; 1971--0.824; 1972--0.799; 1973--0.752; 1974--0.677; 1975--0.620; 1976--0.587; 1977--0.551; 1978--0.517; and assumed inflation rate of 10 percent for 1979.

** Construction still in progress; ultimate effect unknown at this time.

† Includes both scour and foundation settlement effects.

†† Construction expected to begin in late 1979; scour effects estimated.

‡ Phase I and Phase II.

Table 5.2 Estimated Construction Scour Effects (Hale, 1980, Rpt. 1, p. 146)

1958. Subsequent rehabilitation undertaken by the Corps of Engineers has improved the harbor breakwaters. The most recent round of repairs, in 1977, included placement of a band of dolosse for toe protection.

Each climatic region of the United States has different wave and soil conditions. Correspondingly, each generates characteristic foundation scour and erosion problems in coastal construction. The extremes in wave climates in the United States are represented by the high wave energy of the North Pacific coast and the low energy environment of the Great Lakes. The relatively mild wave climate of the Gulf of Mexico is somewhat offset, for foundation problems, by the adverse foundation characteristics of the Mississippi River deltaic formations on the Louisiana coast. Regional problems and construction techniques to overcome scour are reported by Hale (1980, Report 1). This is an excellent guide to site specific scour effects.

<u>Mechanism of Scour</u>. Scour occurs around any object that obstructs the normal underwater flow patterns. When marine foundations alter the dynamic equilibrium, local velocities increase, and additional turbulence and vortices are generated. The flow locally obtains an increased capacity for sediment transport and erodes unprotected foundation materials.

The erosive action of oscillating waves and water currents combine, to differing degrees, to produce scour around nearshore structures. On a straight coastline protected by groins or breakwaters, scour phenomena are attributed predominantly to wave action. Sediments are tossed into suspension by wave attack and transported from the region by wave-induced currents. Where strong currents (other than wave-induced) exist, these may be the dominant scouring mechanism. For example, strong tidal or riverflow currents around a jetty initiate scour themselves, in addition to transporting material removed by wave effects (Hale, 1980).

The magnitude of scour that will occur depends on the type of structure, the characteristics of wave attack, and foundation soil parameters. A rule of thumb estimate of the expected scour, over a long period of time, near reflecting structures is that the maximum depth of scour below the natural bed is about equal to the height of the maximum unbroken wave that can be supported by the original water depth at the structure toe (CERC, 1977). Scour-related laboratory investigations have largely dealt with scour around piles and pipelines, and in front of vertical-faced seawalls. These have identified important variables, but their conclusions are predominantly qualitative in scope.

Sawaragi (1966) studied the phenomenon of toe scour responsible for remarkable subsidence of concrete armor blocks on coastal structures in Japan. He related the void ratios of permeable slopes to the reflection coefficient, the ratio of the reflected to incident wave height, and these parameters to the scouring depth. The depth of scour increased in proportion to the reflection coefficients, for coefficients exceeding 0.25. Thus, energy absorbing sloped rubble structures develop less toe scour than vertical smooth walls.

Herbich and Ko (1968) extended this work and developed a mathematical model to describe scour in front of seawalls. Important variables in the model are water velocity and sand particle diameter. The influence of the reflection coefficient was difficult to isolate, because the reflection itself depends on many variables, such as wave characteristics, seawall slope and the kinematic wave type. For example, there is a great difference between the reflection coefficients of a nonbreaking and breaking wave on a seawall for the same wave characteristics. All experiments indicated an asymptotic limit to scour depth. Erosion advances rapidly during the first few hours and slows progressively until the state of ultimate erosion is attained.

Scour in front of vertical and inclined seawalls was investigated experimentally by Sato, Tanaka and Irie (1968). The most significant parameters in their study were wave reflection effects and the water depth, as well as the characteristics of the waves. In the field, foundation scour was intensive where a breakwater transversed a longshore bar, and at the corner and tip of the structure, owing to the sharp energy gradient of currents and waves in these areas. They found that the maximum scour depth would probably not exceed the deepwater wave height that produced the scour. The role of currents in contributing to field scour was acknowledged.

Wave reflection from walls is generally accepted as a major mechanism of bed scour. Silvester (1977) attributed foundation and downcoast scour to short-crested reflected waves. Submerged structures are as susceptible to the problems of reflection and vortex scour as those extending above the water surface. Although rubble mounds provide less reflected energy than vertical or smooth sloping walls, long period waves of small height can be reflected with little dissipation. The continual action of persistent swell can cause detrimental scour.

Hotta and Marui (1976) performed experiments to clarify the characteristics of scour around detached breakwaters. Local scour and larger-scale changes in bathymetry occurred concurrently in a complex interaction. The maximum scouring depths varied from 0.6 to 1.0 times the initial wave height and were largest at $X/X_b = 0.38$, where X is the distance from the shoreline to the breakwaters and X_b is the distance from shore to the breaking wave point (See Section 5.1). The breakwater position which results in the least scour is at $X/X_b \approx 1.0$; that is, when the breakwaters are situated at or just inside the breaking point. This study concurred with others in concluding that the scour depth will not be greater than the order of the deepwater wave height.

The above investigations identify key parameters in mound foundation scour and confirm the significance of this phenomenon in the design of coastal structures. However, the extent of scour to be expected cannot be projected conclusively. Development of accurate predictive techniques will lead to more effective measures to minimize scour (Hale, 1980).

Protective Measures. There are four general methods to combat scour-induced erosion of rubble mound foundations. These are:

1. Installation of a foundation blanket
2. Placement of excess stone on the toe
3. Overbuilding the rubble structure
4. Excavation of bottom sediment to a predetermined depth

Use of a foundation bedding layer to prevent the formation of a scour hole is common practice. The mats extend some distance beyond the structure to shift wave and current action away from the toe. The mat thickness serves also to distribute structural loads over a wider base, thus reducing settlement and bearing capacity problems. Mats are designed as filters to avoid removal of foundation materials through the blanket and the loss of blanket material into the voids of the cover stone. The design of graded stone and synthetic fabric filter layers is described in the next section. In recent years, gabions have been used to serve this purpose as well (Hale, 1980).

The second two methods of scour control are passive; that is, they prepare for the occurrence and effects of scour rather than prevent them. An extra berm of stone can be placed on the lower slope and toe to drop down and fill any scour trench that may develop. Similarly, the entire mound can be initially overbuilt such that it will eventually settle to the design crest elevation.

Hale (1980) considers foundation excavation as a means of preventing scour on high wave energy coasts. The ultimate scour depth is estimated, and the foundation is excavated to that level. Expected scour depths in sand ranged from 2 to 6 ft (0.6 to 1.8 m). It is desirable to excavate to bedrock or other scour-resistant material when they are located at shallow depth. For example, on the Hawaiian coast it is frequently possible to dig down to a firm coral foundation.

During construction, turbulence at the working end tends to scour a hole in unconsolidated bottom material. If construction proceeds from the outer end toward the shore, similar scour will occur at the unfinished inner end, particularly as it approaches the shoreline. In high wave energy areas, it may be extremely difficult to place the necessary protective bedding layer within the surf zone. Some suggestions for alleviating the problem are (ASCE, 1969):

1. Develop special construction techniques

2. Increase the layer thickness

3. Where possible, use a coarser graded bedding layer

4. Restrict construction in the surf zone to periods when the smallest surf occurs

Techniques for the control of scour during construction are detailed in a three volume Corps of Engineers report by Hale (1980). It is recommended that foundation bedding materials be placed ahead of the core construction at least 50 ft (15 m) to prevent scouring and undermining of the working section. At the end of the construction day, a 30 to 50 ft (9 to 15 m) section of bedding should be laid to minimize overnight scouring. Accelerated core placement has been used successfully in crossing scour holes subject to continuing scour. No more core stone should be placed than can be armored during the construction season.

It is important that scouring magnitudes be predicted and incorporated into the cost estimate and construction planning. Some geographic regions have developed unique procedures for estimating anticipated scour, based on experience. This enables planning for additional project quantities to be required as a result of scour. Also, the careful selection of the construction season, and series of days when tide predictions indicate most favorable working conditions, contribute significantly to the successful completion of rubble mound construction.

Foundation Blanket Design

Bedding or filter blankets are essential in the design and construction of rubble mounds. They serve two vital purposes:

1. Distribute the load over a wider base to reduce soil contact stresses and prevent settlement and bearing capacity failure

2. Provide erosion control by preventing scour of the toe and foundation materials

The lateral extent and thickness of the bedding layer must be sufficient to provide an adequate bearing surface for the overlying material. To perform the second purpose, the layer must also be designed as a filter system and extend some distance beyond the structure toe. Design for these two criteria are discussed in the following two sections.

Filter Design. Partial or complete failure of rubble mounds can be traced in some cases to improper filter design, or a lack of any filter design at all. In the absence of a filter layer, foundation soils can be removed by local scour due to intense wave and current effects. The negative results of this internal erosion include settlement of the toe and main structure and the ensuing washout of underlayer and core materials, as detailed in the prior discussion on scour.

The installation of a filter blanket to protect against undermining is recommended by CERC (1977), except in the following situations:

1. Where the water depth is greater than three times the maximum wave height

2. Where the expected current velocities are too weak to move the foundation sediments

3. Where the foundation is a hard, durable material, as bedrock

Sandy bottoms are most prone to lose material through scour. Cohesive foundations are less inclined to internal erosion, and a filter blanket may not be required. A bedding layer or apron of quarry spalls, gravel or other crushed rock should be provided, however, to remove turbulence scour from the structure toe (CERC, 1977).

A properly designed filter system must satisfy two seemingly contradictory criteria. First, it must be much more permeable than the base or underlayers. It must permit effective drainage of the underlying material so that excess pore water pressures will not be generated. Second, the filter must be graded finely enough to avoid base particle migration into its voids. Washout of the foundation materials or clogging of the filter pores will defeat the purpose of the filter. The filter gradation depends on the characteristics of the foundation material and on the void diameter of the overlying rubble stones. Methods for the design of graded stone filters and synthetic fabric filters are discussed below. Their relative merits and disadvantages are summarized in Table 5.3.

Graded stone filters should be well-graded from their minimum to maximum particle sizes. This allows keying action as the smaller stones fill the voids between the larger stones. Also, any excess or lack of intermediate sizes would increase the tendency toward segregation. Optimally, the gradation curve of the stone filter should approximately parallel that of the base material (Hale, 1980). The design procedure involves the following steps.

1. Mechanical analysis of the base (foundation) material
2. Estimation of the void diameter of the overlying rubble stress
3. Filter design in accordance with the criteria developed by Terzaghi and extended by the Corps of Engineers. These are (Hale, 1980):

$$\frac{d_{15f}}{d_{85b}} < 5 \tag{5.4}$$

$$4 < \frac{d_{15f}}{d_{15b}} < 20 \tag{5.5}$$

Table 5.3 Graded and Plastic Filter Systems-Advantages and Disadvantages (after Lee, 1972, p. 1924)

GRADED FILTER SYSTEM

Advantages	Disadvantages
1. Most likely available	1. Difficult to construct to specs under water
2. Widely accepted in practice	2. Difficult to determine the armor stone void diameter, a parameter needed for filter design
3. Less effect of long-term operation on permeability and filtration	3. Stone filter has no independent strength, i.e., depends on soil for its stability
4. Not affected by bio-deterioration	

PLASTIC FILTER SYSTEM

Advantages	Disadvantages
1. Filtering ability can remain the same during installation	1. Materials may not be readily available
2. Independent tensile strength	2. Initial cost may be higher than graded filter
3. Easier to construct to specs-eliminates screening process required for graded stone filter	3. More difficult to maintain permeability and filtration in the long-term
4. More consistency in as-placed condition	4. Effectiveness may be reduced due to biodeterioration
5. Applicable regardless of geographic location, e.g., availability of graded materials not important	

$$\frac{d_{50f}}{d_{50b}} < 25 \tag{5.6}$$

$$\frac{d_{85f}}{d_{\text{stone voids}}} > 2 \tag{5.7}$$

where d_{15}, d_{50} and d_{85} are the particle sizes from a particle size distribution plot at 15, 50 and 85 percent, respectively, finer by weight. "f" refers to filter and "b" to the base soil sizes. For example, 85 percent by weight of the particles in the foundation soil are smaller than d_{85b}.

A sample design of a graded stone filter layer for a rubble mound is given in Design Example 5.3.

For rubble mound structures with large voids, it is necessary to design a multilayer graded stone filter. The size distribution of each layer is governed by the gradations of the layers adjacent to it, in accordance with the stated design criteria. The process is repeated until the filter material size is sufficiently large to resist invasion into the rubble stone voids (Lee, 1972). The bedding materials specified for many rubble mound jetties and breakwaters are quarry run spoils, from 1 to 50 pounds (4.4 to 222 N) and varying in gradation to 12 inches (0.3 m). This efficient use of up to 80 percent of the quarry spoils generally results in a lower unit cost (Hale, 1980).

The civil engineering use of synthetic fabrics, formed of manmade fibers, has expanded rapidly in the past decade. Fabrics have been quite effective in providing drainage and scour control on shore protection structures. There are currently more than 25 different fabric types commercially available, of various permeabilities and tensile strengths. According to Keown and Dardeau (1980) three key factors must be carefully evaluated in the selection and placement of a filter fabric for a specific project application. These are:

DESIGN EXAMPLE 5.3

DESIGN OF GRADED STONE FILTER LAYER

GIVEN : FOUNDATION SOIL GRADATION CURVE (CURVE I, P. 3)

$d_{15b} = 0.15$ mm
$d_{50b} = 0.31$ mm
$d_{85b} = 0.74$ mm

VOID DIAMETER OF OVERLYING RUBBLE ESTIMATED ≈ 200 mm

REQD : ACCEPTABLE GRADATION LIMITS FOR GRADED STONE FILTER MATERIAL

SOLUTION :

FROM EQN. 5.4, $d_{15f} < 5 d_{85b} = 5(0.74)$

$\therefore d_{15}$ MAX = 3.7 mm (POINT A, P. 3)

FROM EQN. 5.5, $d_{15f} > 4 d_{15b} = 4(0.15)$

$\therefore d_{15_f}$ MIN = 0.6 mm (POINT B)

AND $d_{15f} < 20 d_{15b} = 20(0.15)$

$\therefore d_{15f}$ MAX = 3.0 mm (POINT C)

FROM EQN 5.6, $d_{50f} < 25 d_{50b} = 25(0.31)$

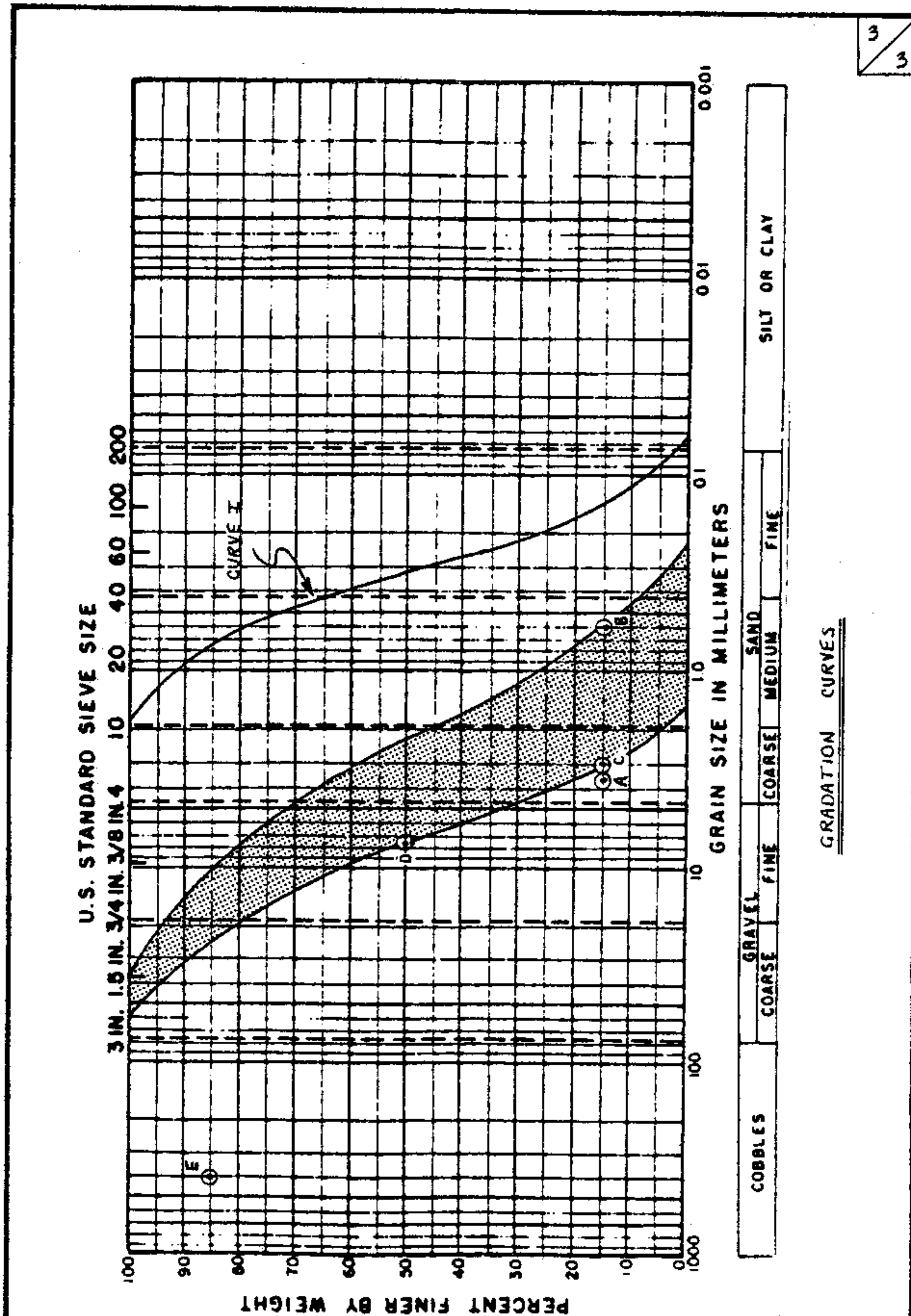
$\therefore d_{50f}$ MAX = 7.75 mm (POINT D)

FROM EQN 5.7, $d_{85f} > 2 d_{STONE} = 2(200)$

$\therefore d_{85f}$ MIN = 400 mm (POINT E)

THE ACCEPTABLE GRADATION RANGE IS SHOWN AS A SHADED REGION ON P.3. THE RANGE WAS DRAWN TO APPROXIMATELY PARALLEL THE FOUNDATION SOIL GRAIN SIZE CURVE.

IF THE SELECTED FILTER MATERIAL DOES NOT INCLUDE POINT E ($d_{85f} > 400$ mm), THE FILTER MATERIAL COULD WASH THROUGH VOIDS IN THE OVERLYING RUBBLE. THE SPECIFICATIONS FOR A SECOND FILTER LAYER SHOULD BE DETERMINED, USING THE CHARACTERISTICS OF THE FIRST FILTER LAYER AS THE "BASE" MATERIAL.



GRADATION CURVES

1. Filtration - the fabric must be an adequate filter, allowing water flow while preventing infiltration of bed particles.

2. Chemical and physical properties - the fabric composition must resist deterioration from climatic conditions and chemicals in the environment, and must be strong enough to prohibit tearing and puncturing during placement and in use.

3. Acceptance of mill certificates and compliance testing - the fabric must meet government standards in these areas.

The filtration of a fabric is characterized by the equivalent opening size (EOS) of the fabric and the gradient ratio (GR) of the fabric-soil matrix. The EOS must be known for the various fabrics available. The following guidelines (Keown and Dardeau, 1980) should be used to select an effective filter fabric:

1. For fabric to be placed adjacent to granular materials containing 50 percent or less fines (particles passing the no. 200 sieve, 0.074 mm), the following criterion must be satisfied:

$$\frac{d_{85} \text{ foundation material (mm)}}{\text{EOS of filter cloth (mm)}} \leq 1 \tag{5.8}$$

2. For fabric to be placed adjacent to other soils, the EOS should be no larger than the openings in the no. 70 sieve (0.211 mm). Filter fabric should not be placed on soils comprising 85 percent or more fines.

3. No fabric should be specified with an EOS smaller than the openings of a no. 100 sieve (0.149 mm).

It is preferable to specify the largest EOS allowed by the criteria. Design Example 5.4 illustrates the use of these guidelines.

The gradient ratio, GR, of a fabric-soil system is the ratio of the hydraulic gradient over the 1 inch (25 mm) of soil adjacent to the fabric, i_f, to the gradient over the 2 inches (51 mm) of soil between 1 and 3 inches (25 and 76 mm) from the fabric, i_g:

1/2

DESIGN EXAMPLE 5.4

DESIGN OF FABRIC FILTER LAYERS

GIVEN: FOUNDATION SOIL GRADATION CURVES I, II AND III (P.2)

SOIL I: $d_{85b} = 0.50$ mm
$d_{50b} = 0.18$ mm

SOIL II: $d_{85b} = 0.50$ mm
$d_{50b} = 0.058$ mm

SOIL III: $d_{85b} = 0.074$ mm
$d_{50b} = 0.017$ mm

REQD: ACCEPTABLE FILTER FABRIC EQUIVALENT OPENING SIZE (EOS) FOR EACH FOUNDATION SOIL

SOLUTION:

SOIL I: % FINES < 50
∴ FROM EQN 5.8, EOS MAX = d_{85b} = 0.50 mm (#35)

EOS MIN = 0.149 mm (#100)

USE LARGEST EOS POSSIBLE BETWEEN 35 AND 100

SOIL II: % FINES > 50
∴ EOS MAX = 0.211 mm (#70)

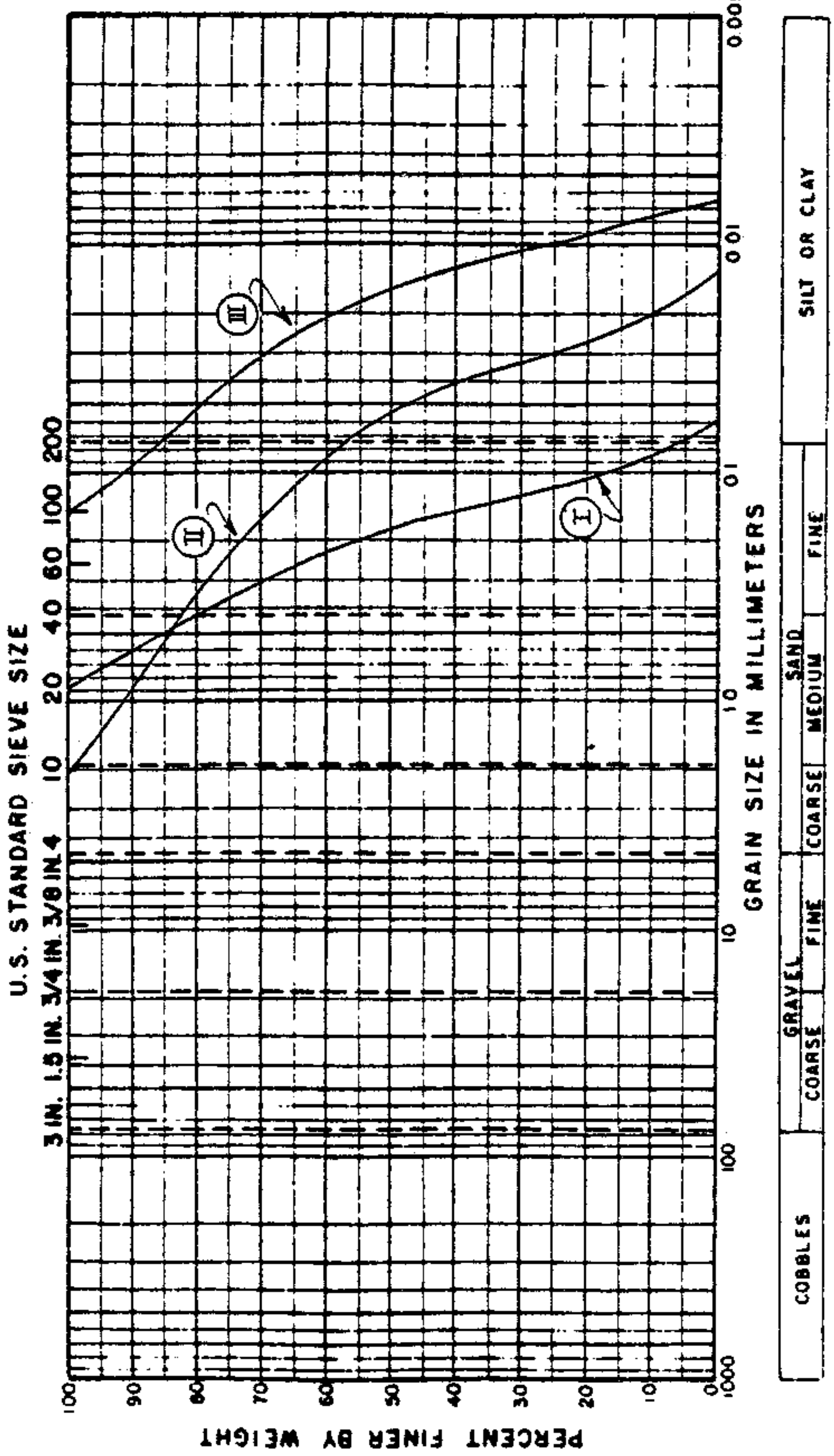
EOS MIN = 0.149 mm (#100)

USE LARGEST EOS POSSIBLE BETWEEN 70 AND 100

SOIL III: % FINES = 85

FILTER FABRIC SHOULD NOT BE USED TO AVOID CLOGGING

2/2



GRADATION CURVES

$$GR = \frac{i_f}{i_g} \tag{5.9}$$

If fine soil particles clog the fabric, the GR will increase. Similarly, if fine particles move through the filter fabric (piping), the GR will decrease. As a general rule, the GR should not be greater than three (Keown and Dardeau, 1980).

The chemical and physical properties of construction fabrics must meet the current Corps of Engineers specifications. Fabric properties, physical, mechanical, hydraulic and environmental, are presented by Koerner and Welsh (1980). Because manufacturers often change specifications of their fabrics while retaining the same style classifications, the listed properties should be verified by contacting the manufacturer directly. The addresses are provided in Appendix B. Also included is a tabulation of material costs for selected fabrics.

The underwater placement of graded stone filter layers presents a difficult problem. Variations in placement and in materials can result in a nonuniform filter bed with local weak spots. If properly placed, the use of synthetic fabrics eliminates much uncertainty regarding the as-placed condition of the filter (Hale, 1980). Several placement methods are discussed in Dunham and Barrett (1974), Keown and Dardeau (1980) and Koerner and Welsh (1980).

Load Distribution. When large quarry stones are placed directly on the bottom, wave and current turbulence will scour sand from beneath the stones and they will sink. Larger-scale foundation settlements can occur when the heavy load of a rubble mound bears directly on soft compressible soils. Further, when the imposed load exceeds the bearing capacity of the soil, a slip-circle failure may occur through the foundation strata. These settlement and bearing capacity problems are described independently in preceding sections. It is commonly accepted that most rubble mound structures require foundation mats to prevent potential failures of this nature. Mats distribute the foundation loads over a wider area and thus reduce the contact pressures felt by the soil. Bedding mats designed as filters also serve to control scour erosion.

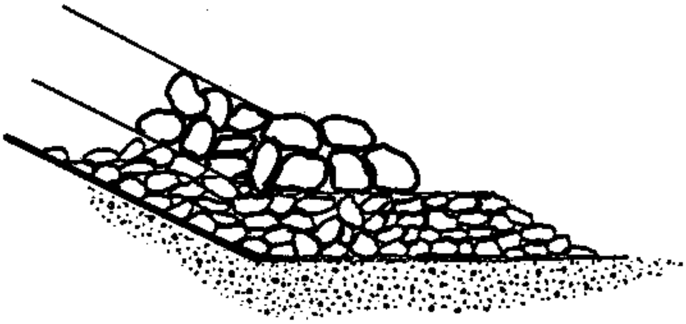
Bedding layers prevent the subsidence of heavy units simply by raising the large stones off of the sediment bed. In some improperly designed or placed graded stone layers, small filter material may migrate up through the armor and allow the units to rest directly on the bottom. This problem will not occur with filter cloths. The fabrics are continuous elements with independent tensile strength and therefore cannot move up through the mound unit by unit as gravel can (Dunham and Barrett, 1974).

Blanket thickness depends on the water depth and the sizes of the overlying quarrystone, but should not be less than 1 ft (0.3 m) to ensure complete coverage of bottom irregularities (CERC, 1977). Dunham and Finn (1974) recommend a thickness of about 1.5 times the average diameter of the bedding stone. Blanket thicknesses on Corps of Engineers projects vary with location, but are on the order of 2 to 3 ft (0.6 to 0.9 m). In the Great Lakes region, a layer of sand has been placed initially on soft muds, and then covered with quarry run stone. Off the coast of Louisiana, where shell is abundant, a layer of this material is often used (Hale, 1980).

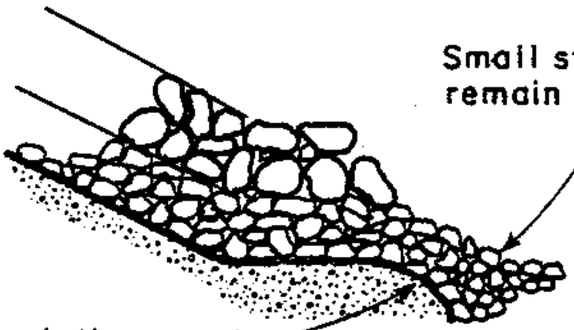
A thin layer of synthetic fabric may be an adequate filter, but will not distribute the load of the overlying mound. Also, heavy and angular stones dropped directly onto the fabric, even from heights less than 1 ft (0.3 m), can puncture and rip the material. For these reasons, it is generally necessary to increase the thickness of filter fabric bedding layers by covering them with a cushioning layer of gravel, quarry spalls or other granular material. Care should be taken to ensure that this intermediate layer does not form a low permeability barrier between the mound and fabric (Keown and Dardeau, 1980). If the armor stones weigh on the order of 10 tons (89 kN) or more, two supporting layers may be required to achieve satisfactory load distribution over the base soil (Dunham and Barrett, 1974).

The base material should be extended beyond the toe, for scour protection, and beyond the potential plane of bearing capacity failure. The standard procedure is to place graded stone layers to at least 5 ft (1.5 m) beyond the toe of the cover stone (CERC, 1977). In practice, the mat extends from 5 to 25 ft (1.5 to 7.6 m) beyond the toe. In the heavy wave climate of the Oregon coast, for example, the foundation bedding is 5 ft (1.5 m) thick and continues beyond the structure toe for 25 ft (7.6 m) (Hale, 1980).

When filter fabric is used in the foundation mat, the seaward end of the fabric should be overlapped by a few feet of stone (Figures 5.14a and 5.15). If scour occurs at the toe and the rocks beyond the fabric are undermined, they will drop into the scour hole but remain to protect the toe (Figure 5.14b). However, if the fabric extends beyond the stone, the material will flap in the wave action and accelerate the



a. Initial placement



b. After many wave cycles

Figure 5.14 Filter Fabric at Rubble Mound Toe
(Dunham and Barrett, 1974, p. 17)

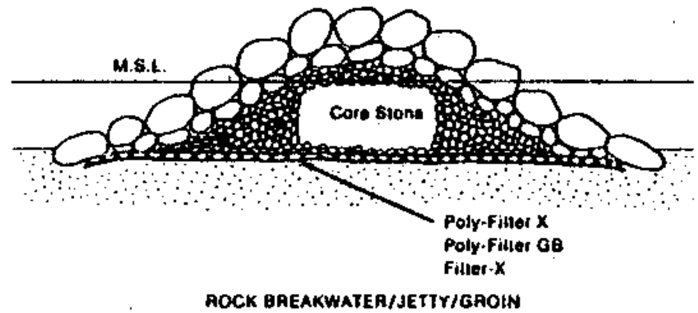


Figure 5.15 Filter Fabric as a Rubble Mound Bedding Layer (Carthage Mills, undated, p. 2)

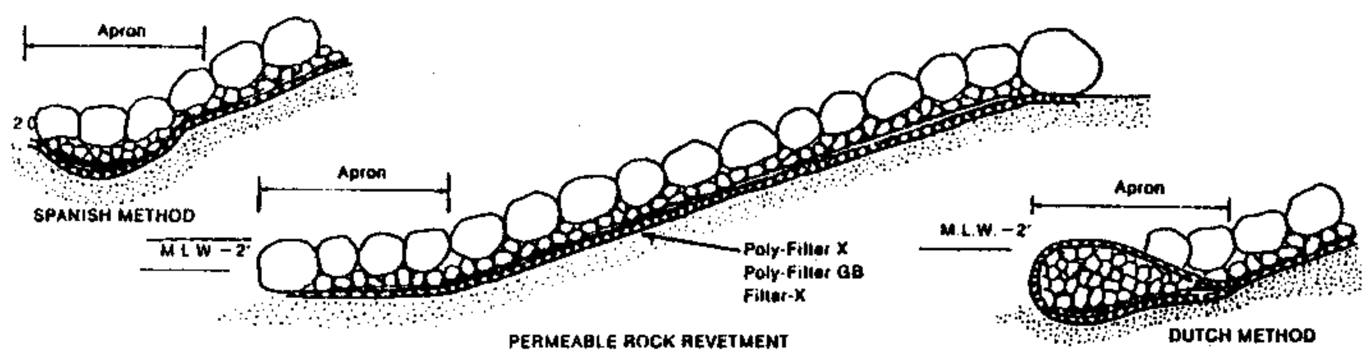


Figure 5.16 Alternative Toe Treatments with Filter Fabrics (Carthage Mills, undated, p. 2)

formation of a scour pit at the toe (Dunham and Barrett, 1974). Alternative toe treatments are shown in Figure 5.16.

5.3 SUMMARY

Shore protection structures are exposed to a number of forces, which interact to produce a complex and often unquantifiable net result. Environmental loads must be defined as accurately as possible for their input to structural design. Waves impose the most significant loads on rubble mounds, and wave parameters therefore have a prominent role in current design formulas (Chapter 7). Proper characterization of the design wave is extremely important. The influences of other environmental forces discussed in this chapter should be evaluated on a site-specific basis. When judged appropriate, their magnitudes can be estimated and incorporated into the design.

The importance of foundation conditions in rubble mound design must not be underplayed. Excessive settlement and insufficient soil bearing capacity can result in complete structural failure. Similarly, critical toe scour can threaten stability during construction and throughout the service life of the mound. Adequate geotechnical investigations and analyses form a sound basis for rational foundation design.

The environmental and geotechnical conditions are unique for each site. Their characteristics can change with modifications in the proposed functional characteristics, as structure orientation and configuration, and the structural design, i.e., whether rigid or flexible, mound or wall. For this reason, the design considerations must be reanalyzed for each option in the design phase.

CHAPTER 6

COVER LAYER MATERIALS

The selection of rubble mound cover layer materials demands particular attention. The integrity of a mound structure depends fully on the stability characteristics of the armor layer units. Hydraulic stability is assured through proper specification of their weight, shape, interlocking and other aspects; Section 7.1, on cover layer stability, deals with these considerations. Structural stability is a matter of the durability of the material in the coastal environment, the focus of this chapter.

Armor units can be large quarrystones or precast concrete shapes. Historically and most commonly, rock has been used in cover layer design. When durable rock of the appropriate sizes is available this is often the most economical material choice. The literature addresses the hydraulic stability of rock elements in great detail, but there is a significant lack of detail regarding structural factors and performance. The material properties of rock which affect its durability in coastal engineering applications are investigated in Section 6.1.

In larger installations and more severe environments, progressively higher armor unit weights are necessary for cover layer hydraulic stability. Specially formed concrete armor units have been used increasingly for these projects. Small concrete shapes can provide the same protection as larger rock units, because the manufactured elements have superior hydraulic stability characteristics (See Section 7.1). In some cases, rock of the necessary size is simply unavailable; 20 tons is commonly thought to be the largest rock that can be economically produced and handled (Fookes and Poole, 1981). The smaller concrete shapes might be an economical and convenient material alternative. The performance of concrete in the coastal zone has been discussed by Hubbell and Kulhawy (1979a). Section 6.2 reviews features of material durability and production unique to concrete armor units.

Material availability and related considerations are quite important and may in fact impose limitations on the structural design. Similarly construction aspects (Section 7.3) might limit or prevent the use of certain materials. The identification and evaluation of possible construction materials is often initiated before the design phase but, as demonstrated, is integral to effective structural design.

Considerations pertinent to the choice of cover layer material are summarized in Table 6.1. In the final analysis, economic constraints usually prevail. A recommended basis for comparison of quarrystone and concrete armor unit costs is the average annual cost of the rubble mound, for equal protection, computed over the design life of the structure (Hudson, 1974).

6.1 ROCK FOR ARMOR UNITS

Rock is the primary material used for the construction of rubble mounds. A major anachronism in rubble mound design is the lack of adequate guidelines for the evaluation of rock durability and acceptance. Too often, rock is deemed suitable based only on a cursory examination of an exposed rock face and the proximity of the source. Such preliminary considerations are far from a complete assessment of

Table 6.1 Considerations in the Selection of Rubble Mound Armor Units (after CERC, 1977 and Hubbell and Kulhawy, 1979a)

MATERIAL FACTORS	
a. General	Performance record Permissible damage/longevity Volume of core materials needed (may vary with type of unit selected)
b. Quarrystone	Availability Size and number needed Transportation to site Stockpiling Other costs
c. Concrete Shapes	Availability of forms Size and number needed Quality of concrete Choice of shape Need for reinforcing Transportation Stockpiling Royalty costs Other costs
CONSTRUCTION FACTORS	Method of placement Equipment needed for installation Installation cost Contractor experience with material Contractor skill
OTHER FACTORS	Environmental considerations Ease of repair Esthetics of the final product Intangibles, as local custom, peer pressure and preconceived notions

the material acceptability. Specifications for armor unit rock typically include such phrases as "dense, hard and sound." Although these are desirable rock characteristics, they are strictly qualitative terms, subject to individual interpretation. It is the intent of this section to present a more effective guide to the appraisal of rock quality for rubble mound armor units.

The suitablity of a particular rock source can be judged according to two broad criteria:

1. The rock must be durable in the marine environment.
2. The source must be able to provide enough rock of the proper size for the project.

It is necessary to understand first the weathering forces to which cover layer rock will be exposed. The review in the first part of this section can be supplemented by Hubbell and Kulhawy (1979b). Site and laboratory investigative programs are then presented, with an emphasis on identifying features and properties which will influence rock durability. The second criterion listed is highly dependent on quarrying procedures; these must be carefully planned with consideration to the geological setting and characteristics of the rock. Finally, the effect of pertinent rock properties on quarry design are examined.

Rock Weathering Processes

Weathering is defined as "the process of alteration of materials occurring under the direct influence of the hydrosphere and atmosphere." Weathering of intact rock masses occurs over geological, or very long, time spans. Alternatively, short term weathering effects which take place during the service life of engineered structures are of interest to this and other engineering studies (Fookes, Dearman and Franklin, 1971). The modes of short term weathering of rock can be considered under three headings: 1) chemical, 2) mechanical, and 3) biological weathering. Rubble mound armor units are exposed to all three types of attack, as described in the following paragraphs.

Chemical weathering, or decomposition, involves chemical alteration of the rock and implies transformation of the constituent minerals, usually to some form of clay (Weinert, 1974). Minerals most vulnerable to weathering are those rich in magnesium, calcium and iron. Quartz generally remains unaffected. Common rock minerals are classed, in Table 6.2, by their resistance to weathering. Rocks which comprise a large percentage of low resistance minerals are most likely to decompose (Lama and Vutukuri, 1978).

Solution by seawater is the prevalent means of chemical weathering in the marine environment. Solution is the disassociation of a mineral in a solvent, as water. The mineral substance tends to be attacked by the solvent until saturation is reached. For example, cold seawater and strongly diluted harbor waters may become locally undersaturated with calcium carbonate ($CaCO_3$). Strong solution of calcareous rock can occur in these waters. Similarly, soft-water lakes, with a $CaCO_3$ concentration of less than 40 ppm and a pH of 6.8 to 7.4, are corrosive to carbonate rock and concrete. Hard-water lakes, usually located in areas of glacial drift or carbonate rocks, are generally lime-saturated and not aggressive (Winkler, 1973).

Mechanical weathering results in a physical breakdown of the rock. The net effects of all mechanical weathering modes are particle size

1. Low resistance:	
olivine	staurolite
pyroxene	iron-rich garnet
hornblende	melilite
calcic plagioclase	epidote
biotite mica	feldspathoids
2. Intermediate resistance:	
intermediate plagioclase	sillimanite
sodic plagioclase	andalusite
orthoclase feldspar	kyanite
microcline feldspar	calcium aluminium garnet
muscovite mica	iron-poor garnet
	clinozoisite
3. High resistance:	
quartz	hematite
magnetite	tourmaline

Table 6.2 Resistance to Chemical Weathering of Common Minerals of Igneous and Metamorphic Rocks (Lama and Vutukuri, 1978, p. 270)

reduction and increased surface area (Fookes, Dearman and Franklin, 1971). The various phenonema of mechanical weathering are summarized by Lutton, Houston and Warriner (1981) as follows:

1. Cracking - development within individual rock fragments of one or a few throughgoing cracks, usually propagated parallel or perpendicular to planar geological structures.

2. Spalling - relatively thin shells break away from the fragment surface.

3. Delaminating or splitting - separations occur preferentially along geologic features as bedding, shaly layers, partings, etc.

4. Disaggregating - continuing erosion of increments of rock, usually associated with granular rocks where individual grains are held together by a weak cementing material.

5. Disintegrating - the most severe and rapid mechanical deterioration, leaves few or no traces of the original fragments.

In weathering modes 1 through 4, some semblance of the original rock remains intact; for this reason, rocks which deteriorate physically are preferable to those which decompose, if such a distinction must be made.

Mechanical weathering is caused by a number of forces in the coastal zone. Foremost among these are wave impact, thermally induced volume change, frost action, and wetting and drying cycles.

Biological weathering combines the actions of chemical and mechanical weathering. Microbiological attack on stone is largely chemical in nature. The destructive forces of higher plant and animal life may be biochemical or mechanical in nature (Winkler, 1973).

Rock boring organisms act in the intertidal zone. Mechanical borers, such as Angel Wings, dig at a rate of 12 mm or 0.47 inches per year, with a total depth of 150 mm or 5.9 inches. Some genera of sea urchins bore 10 mm or 0.39 inches per year in limestone, and more in softer rock. Chemical borers are restricted to carbonate rocks. Biochemical erosion in the subtidal zone of Bermuda reaches 14 mm or 0.55 inches per year for Cliona, a sponge, and 13 mm or 0.51 inches per year for the boring clam Lithophaga (Winkler, 1973). Although such biological weathering may progress at a fast rate, the resulting erosion does not generally threaten the structural integrity of armor unit rock.

Climatic Considerations. For all modes of weathering, climate plays a vital role in influencing the engineering performance of rocks. A climatic index of weathering proposed by Weinert (1974) focuses on the availability of moisture as a most important parameter. The N index indicates, in general, that as the annual precipitation increases, decomposition becomes the predominant form of crystalline rock weathering. Peltier (1950) also correlated type and intensity of weathering with climatic conditions, using temperature as the significant parameter. As high temperatures expedite most chemical reactions, chemical weathering is enhanced in warmer climates. Frost and ice action in cold climates result in increased rock disintegration or mechanical weathering. Peltier's diagram, Figure 6.1, suggests the relationship of various temperature-precipitation conditions to weathering modes (Fookes, Dearman and Franklin, 1971).

These climatic considerations, while not definitive or conclusive, may provide preliminary guidance concerning the material durability. This point was demonstrated in investigations of riprap disintegration, reported by Esmiol (1968). In most cases, rock weathering modes at the respective quarries were identical to the weathering which caused failure of the riprap. Simple attention to climatic considerations

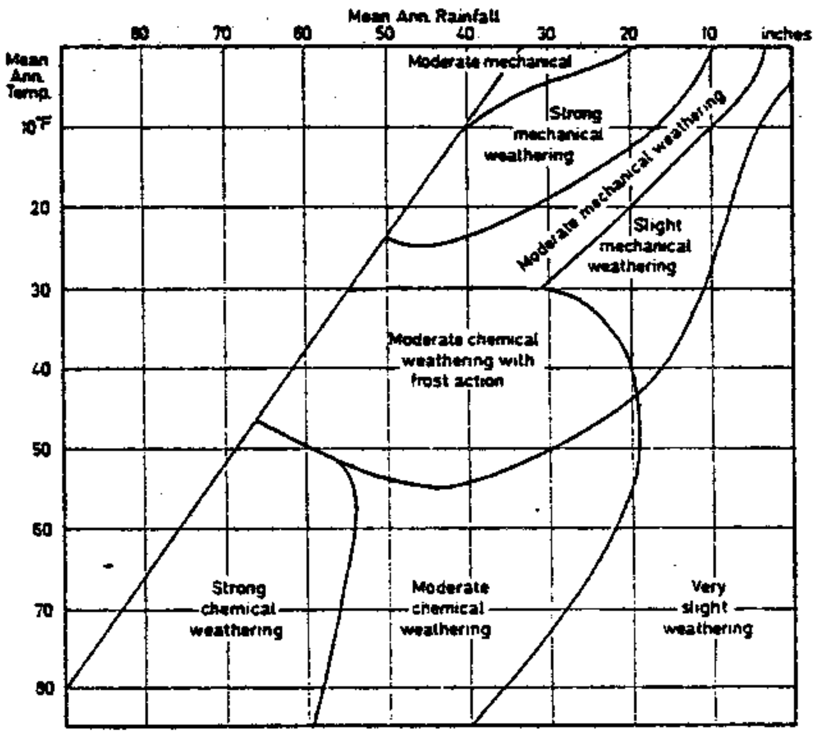


Figure 6.1 Suggested Relative Importance of Various Weathering Modes (Fookes, Dearman and Franklin, 1971, p. 163 after Peltier, 1950, p. 219)

might have signaled the unacceptability of the materials for the planned projects.

Weathering Zones. The four main weathering zones of the coastal environment, with reference to mound structures, are shown in Figure 6.2. The processes which occur at each level are described by Fookes and Poole (1981).

Zone I is a splash zone above water level. Surfaces may be coated by salt spray, abraded by windblown particles, and wet by intermittent rains. Plant growth may be active. The influence of climatic factors is particularly important in this zone. In hot climates, decomposition by chemical weathering may be prevalent; in cold regions, physical disintegration by freeze-thaw cycling may be predominant.

Zone II, above the high water level, is the region of wave runup. Correspondingly, most weathering action occurs from intermittent wetting and drying.

Zone III is the intertidal zone. Deterioration is likely to be most severe in this area. The cycles of wetting and drying are dominant and destructive weathering forces. The zone below the lowest water level is subject to wave action but acted on by very limited subaerial weathering or wetting and drying. Biological attack by boring organisms may be significant locally.

Zone IV is permanently submerged. No subaerial weathering can occur. Climatic factors, such as warm sea water, are still of importance, as are fluctuating water currents. This is the least aggressive weathering zone.

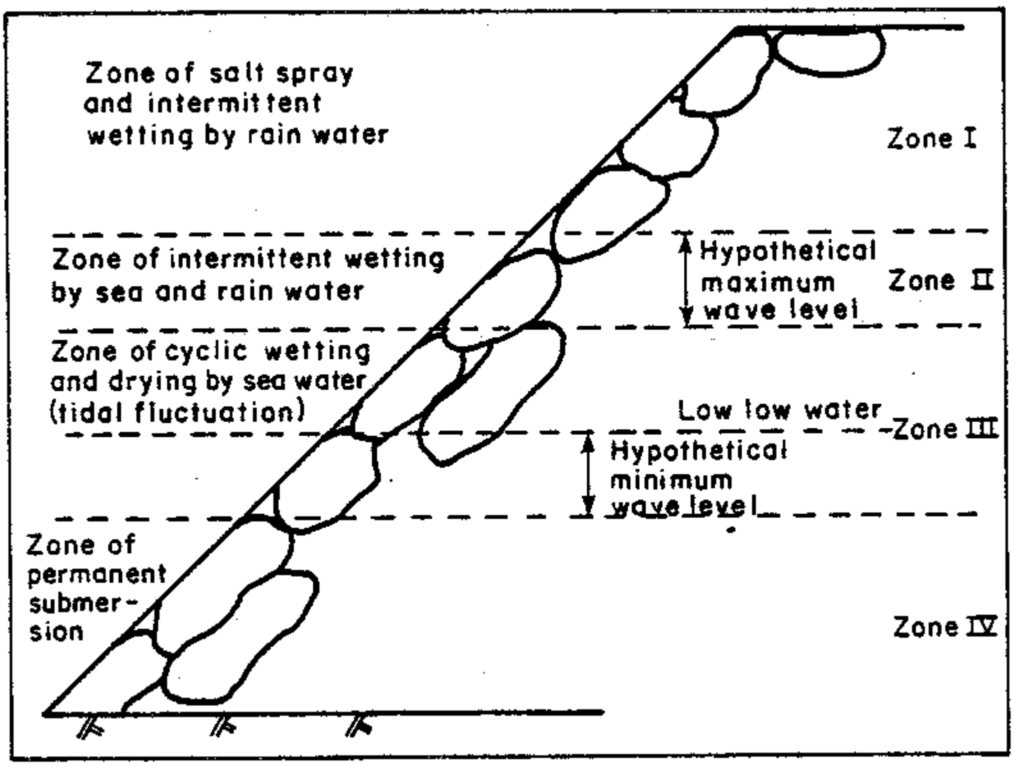


Figure 6.2 Four Main Weathering Zones of the Coastal Environment (Fookes and Poole, 1981, p. 98)

Field Investigation and Assessment

Site investigation of potential sources of cover layer rock is an integral part of the rock evaluation. Without field data the results of even extensive laboratory testing are meaningless. Data required from the site reconnaissance phase are summarized below and in Table 6.3. The following review is intended to acquaint the unfamiliar reader with the important aspects of site investigation and is therefore far from an exhaustive survey on the subject. The actual field study must be performed by persons who are qualified, by education and/or experience, to do so.

Preliminary examination of sources is initiated by a desk study. Available maps, aerial photographs, reports and local knowledge are compiled and studied to isolate likely sources for concentration of field efforts (Fookes and Poole, 1981). Survey of opened sites presents many advantages. Subsurface investigations have been performed and geologic features are often readily accessible for examination. Details of the quarrying operations may be secured. Case histories of rock from the quarry are an inexpensive source of valuable information. It may be discovered that stone which did not meet laboratory test specifications has actually performed well as placed. In the final evaluation in such cases it is reasonable to weigh the service record more heavily than the laboratory results (Treasher, 1964).

The follow-up site reconnaissance visit involves appraisal of promising sites noted in the desk study. Geological maps of surface outcrops should be compiled, with emphasis on establishing the proportions of suitable and usable materials (for the project) available. Information gathered at an opened quarry may, in specific

Table 6.3 Check List for Quarry and/or Outcrop Site Exploration (compiled from Treasher, 1964 and Brown, 1981)

---

1. Quarry or outcrop information: Local name, or designated name or number: Condition - operating, idle, undeveloped.

2. Location: Region, county, state, topographic quadrangle: Mileage log from easily located point.

3. Topography: Of region and site: Gradients of slopes, valley width, vegetation, precipitation, length of working season. Dimensions of outcrop, depth of overburden.

4. Petrography: Rock classification(s): Color, hardness, grain size, bonding of grains, clay, weaknesses, microfracturing, grain boundaries.

5. Structure: Bedding, joints, fractures, faults: Orientation, spacing, weathering alteration, length, aperture, persistence, filling.

6. Quality: Estimate for each rock type: Specific gravity, hardness, toughness, brittleness, porosity, weaknesses.

7. Quantity: Estimate: Block size, percentage of overburden waste, cubic yards produceable.

8. Exploration: Recommend subsurface exploration: Trenches, drilling, quarry shots - location, depth, cost estimate.

9. Operation: Recommend: Drill and blast method, processing of broken rock, loading, hauling.

10. Spoil Area: Location: Distance, gradient to site, how transported, cubic yard capacity, cost per ton of disposal.

11. Haulage: Mileage, gradients (adverse or favorable), roads from site to project: Ownership, weight regulations, traffic restrictions.

12. Utilities: Water, electric power, telephone.

13. Service Record: When, where, placement, size and gradations, duration service, lab data, maintenance and cost data.

Table 6.3 (Continued)

14. Laboratory Testing: Recommend petrographic analysis, specific gravity, absorption, etc. (See next section)

15. Ownership: Land, mineral rights, royalty - or rental, access.

16. Recommendations: Excellent, good, fair, poor, or unusable in accordance with predetermined rating scheme. Sampling and exploration needed to evaluate fully.

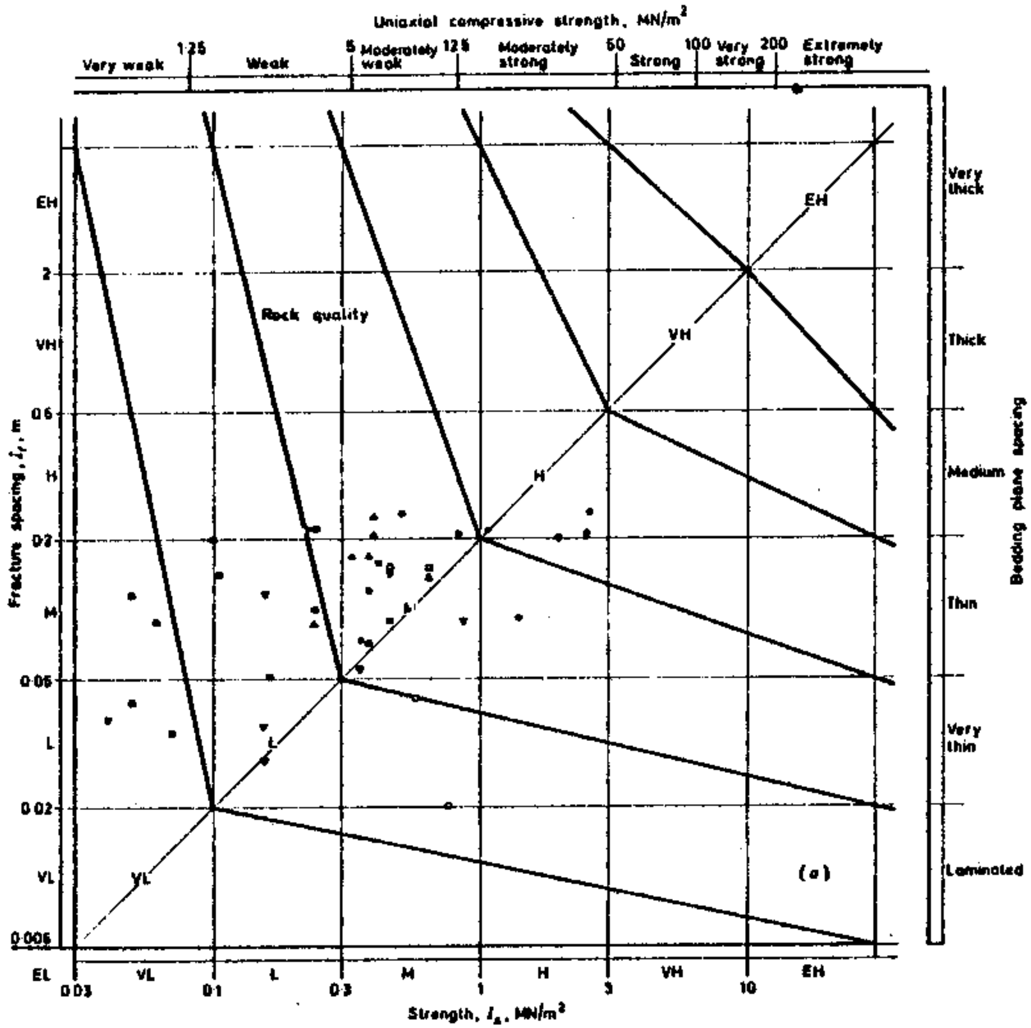
17. References: Sketches of quarry or outcrop; Location and other maps; Tabulate references.

18. After Exploration Add: Topographic and geologic maps; Maps showing exploration sites - superimpose geotechnical data; Logs and evaluation of exploration data; Results of laboratory investigation; Final evaluation of site.

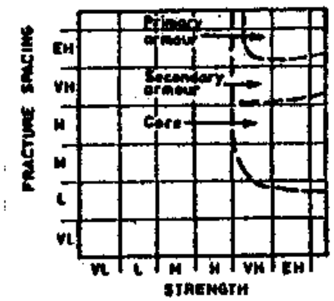
cases and with caution, be extrapolated to aid in the study of similar rock at a distant location. Geotechnical data should be superimposed on geologic maps to maximize their usefulness. Rock mass characterization includes details on the elements highlighted below.

— Lithology. An initial investigation of the lithology can often eliminate much additional work later. A trained geologist can easily determine that certain rock units will not be suitable for use as armor stone, particularly those that are thin bedded, severely fractured, or quite shaly (Lienhart and Stransky, 1981). A detailed lithologic log of the proposed source should be compiled. Each material should be categorized in accordance with geologic and engineering classification systems. A useful general rock quality designation scheme is illustrated in Figure 6.3a. The subdivisions on the top and right of this diagram are based on those suggested by the Geological Society of London (Franklin, Broch and Walton, 1971). A similar relation specifically for breakwater stone is shown in Figure 6.3b.

Structure. Information on bedding planes, joints, faults and other linearities should be reported. Details include spacing, attitude, aperture, fillings and persistance of discontinuities. The rock size obtainable at a given location, a vital parameter in cover layer design, can be estimated by the fracture spacing index, I_f. The index, the mean diameter of a typical block, is evaluated by averaging the dimensions of several representative rock specimens. Table 6.4 gives corresponding qualitative descriptions. Block shape can be assessed by observing the orientations and spacings of each fracture set or potential planes of weakness (Franklin, 1974). Reference should be made to the suggested methods for the quantitative description of discontinuities in rock



a. General purpose rock quality classification diagram (Franklin, Broch and Walton, 1971, p. A7)



b. Rock classification for suitability as rubble mound material (Fookes and Poole, 1981, p. 127)

Figure 6.3 Graphical Rock Quality Designation Schemes

Table 6.4 Descriptive Terminology for Fracture Spacing Index, I_f (Fookes, Dearman and Franklin, 1971, p. 149)

Spacing	Fracture Spacing Index, I_f	
	Meters	Feet
Extremely High	>2	>6.56
Very High	0.6-2	1.97-6.56
High	0.2-0.6	0.66-1.97
Medium	0.06-0.2	0.2-0.66
Low	0.02-0.06	0.06-0.2
Very Low	0.006-0.02	0.02-0.06
Extremely Low	<0.006	<0.02

masses set forth by the International Society for Rock Mechanics, in Brown (1981).

Additional Characteristics. Information on the weathering profile should be reported. The depth of overburden will affect the cost of quarrying and the weathering depth affects the yield of large rocks. The level of groundwater, permeabilities of rocks and natural drainage courses should be noted. Suitable access from the quarry to the construction site must be located. Samples should be taken during mapping for preliminary field or laboratory testing, discussed subsequently (Fookes and Poole, 1981).

Based on this information the most likely sources are selected for detailed subsurface exploration. Trenches can be excavated inexpensively to determine the nature and extent of overburden. Borings yield more definitive information regarding rock quality. Drill holes may be initiated in trench bottoms, to avoid drilling through overburden. The boring pattern should roughly delineate the site. Spacing and inclination should be planned such that the intact rock and irregularities are explored in full (Treasher, 1964). Rock core quality is judged by a core recovery ratio, the rock quality designation (RQD), on cores of 2.125 inches (54 mm) or more in diameter. Recovered core includes only those fragments of 4 inches (102 mm) or more in length. The percentage ratio of total recovered length to total drilled length for a core run is the RQD. Table 6.5 gives qualitative rock quality descriptions for ranges of RQD.

A quarry test shot forms an additional opening to be explored in detail by the geologist. The pilot also demonstrates the style of rock breakage and enables a practical evaluation of the relative quantity and

Table 6.5 Relation of RQD and In-Situ Rock Quality (after Deere, 1963)

RQD (%)	Rock Quality
90-100	Excellent
75-90	Good
50-75	Fair
25-50	Poor
0-25	Very Poor

quality of blocks produced (Treasher, 1964). It is often difficult to predict these accurately without blasting trials.

Laboratory Testing

A well-planned laboratory testing program is required to evaluate fully the durability of armor stone. The rock properties and parameters judged pertinent to this task vary with the performing laboratory and the climatic region. For example, in areas with mild temperatures, freeze-thaw durability is not an appropriate guideline to durability. The characteristics most commonly determined are:

1. Petrography
2. Specific Gravity
3. Absorption
4. Freeze-Thaw Durability
5. Soundness
6. Abrasion
7. Ultrasonic Rock Cavitation

Other tests are proposed occasionally and some deserve careful evaluation for applicability. This continuing review process assures that the state of the technology will remain current (Lutton, Houston and Warriner, 1981).

The rock tests are discussed below with emphasis on the information they reveal, and the use of these data in evaluating rock durability. Details of the actual testing procedures are not presented here. The test designation numbers noted throughout correspond to the specifications of the American Society for Testing and Materials (ASTM,

1980) and to the Rock Testing Handbook (RTH), the standards of the Corps of Engineers (USCOE, 1980). These detailed specifications should be consulted for descriptions of the laboratory procedures cited. Further, guidelines have been developed by the International Society for Rock Mechanics, ISRM, in Brown (1981), and should be followed where appropriate.

Sampling. Laboratory test results are only as representative as the sample submitted. Stone selected for testing should be nearly the same size as that to be used in the project or as large as practical. Rock samples which exhibit the typical variations in the lithologic unit, both vertically and laterally, are the most valuable; units which vary considerably from the norm for the production face should not be selected. Potential weaknesses must be analyzed and evaluated for their effect on material longevity. It is preferable that samples be obtained from freshly produced stone (Lienhart and Stransky, 1981).

Petrography (ASTM C295-79, RTH 102-80). Petrographic examination affords a valuable qualitative appraisal of rock quality. Petrographic analysis identifies rock origin, mineralogy and details of the fabric. The presence of swelling or soluble minerals may be noted. Critical weaknesses in the rock mass, as micro-fissures, clay seams, alteration zones and unsound areas, are detected. Details of texture and porosity may be sufficient to estimate the probable response of rock to weathering conditions. The data revealed also allow a more informed evaluation of other test results.

Specific Gravity (ASTM C128-79, RTH 107-80). The importance of rock specific gravity is indicated by its prominence in the rubble mound stability formula, Equation 7.2, and Design Example 7.3. To maximize resistance to rock displacement by wave action, it is advantageous to choose a high specific gravity rock. However, the opposite requirement, a low specific gravity, is desirable in consideration of transportation and material handling costs.

Table 6.6 lists typical values of specific gravity for various rock types. These values are from a study in which the upstream slope protection systems of earth dams were evaluated. The data tabulated are for materials which performed excellently as riprap and, so, are representative of "good quality" materials (Esmiol, 1968).

Absorption (ASTM C128-79, RTH 107-80). The absorption index measures the amount of moisture absorbed by the stone. It also gives a rough indication of porosity and the related degree of weathering or alteration (Brown, 1981). The absorption value can be successfully used in conjunction with petrographic examination to determine pore size and pore system extent and the related freeze-thaw susceptibility.

Absorption values of lower than 2.5 percent generally signify adequate rock quality (Treasher, 1964). Absorption values for good quality materials are listed in Table 6.6.

Freeze-Thaw Durability (ASTM C666-77). This accelerated weathering test is designed to simulate the exposure to wintertime conditions by subjecting the rock to cycles of freezing and thawing. Rocks soaked continuously and then exposed to freezing temperatures are generally the most vulnerable to frost damage. On rubble mounds, this situation exists in cover stones just above the water level (Winkler, 1973).

The test procedure is most effective in exposing the poor freeze-thaw durability of minutely pored rocks which are not free

Table 6.6 Laboratory Test Data for Riprap on Dams Rated "Excellent" (compiled from Esmiol, 1968)

Rock Group	Rock Type	Specific Gravity	Absorption, %	Abrasion Loss, % (500 revolutions)	Soundness Loss, % (sodium sulfate-5 cycles)
Igneous	Rhyolite	2.67	0.3	20.6	1.4
	Granite	2.55-2.65	0.46-1.3	23.0-41.7	1.2-27.2
	Syenite	2.65-2.74		17.6-18.2	0.6-1.9
	Andesite	2.75-2.96	0.0-0.2	11.8-15.8	0.4-2.5
	Diorite	2.67-2.69	0.5	23.0	
	Basalt	2.62-2.83	0.7-2.8	15.3-34.3	0.9-6.8
Sedimentary	Sandstone	2.50-2.76	1.1-2.1	38.9-45.0	0.6-20.0
	Siltstone	2.63			0.5
	Limestone	2.64-2.70	0.1-1.0	26.0-34.0	1.6-2.5
Metamorphic	Quartzite	2.65	0.0		
	Phyllite and Schist	2.74	0.3	21.8	1.8

draining, as cherts and some limestones. The least affected are larger pore permeable rocks. The mode of physical breakdown manifested in the test should also be noted, as this indicates failure phenomena which may occur in the field. For example, rocks which displayed slaking and spalling in laboratory tests have disintegrated by ~~a~~ like mechanism in practice (USCOE, 1962).

Some controversy exists regarding this evaluation of frost sensitivity. The action by which the rock specimens fail is not completely understood. Hudec (1978) and Winkler (1973) may be consulted for discussions on this topic. A practical disadvantage to freeze-thaw simulation is that it can take as long as five weeks to perform 250 cycles.

Soundness (ASTM C88-76). The soundness test follows a procedure similar to the freeze-thaw test, but sodium sulfate or magnesium sulfate solutions are used as the soaking agents. There is some doubt that this test reveals any deterioration properties of the rock not indicated by the freeze-thaw or related wetting-drying tests. Further, the test may cause failure in rocks which, in nature, would be little affected by freezing and thawing. The soundness test is no longer listed in the Corps of Engineers guide specifications, but is still conducted routinely as a durability test in some labs (Lutton, Houston and Warriner, 1981).

The performance of aggregates for use in highway construction is often assessed by the soundness loss. For example, the maximum five cycle soundness loss for concrete pavement aggregates in Illinois is 15 percent (Harvey, et. al., 1978). Typical values for acceptable riprap are much lower, as demonstrated in Table 6.6.

Abrasion (ASTM C131-76, RTH 115-80). The Los Angeles abrasion test is useful in evaluating the resistance of rocks to wear between rock pieces and to impact forces produced by an abrasive charge (Brown, 1981). Lienhart and Stransky (1981) recommend the test be used only in cases where the petrographic exam indicates potential problems with "soft" rocks. For example, a friable specimen may be relatively free of planes of weakness and may be durable in accelerated weathering tests, but may be unable to withstand the impact of wave action.

Evaluation of abrasion may be aided by simple field methods as hammering the rock or crushing large pieces. Table 6.6 lists abrasion values representative of acceptable materials. Weight losses in excess of 40 percent are generally considered to be unsatisfactory. In some instances, however, materials which exhibited losses of up to 75 percent have performed well as rubble mound stone (Treasher, 1964). The abrasion test, therefore, should not be accepted as the sole indicator of rock durability.

Ultrasonic Cavitation. The application of ultrasonic energy to cause disaggregation of weak rock may prove promising for testing riprap. A quasi-qualitative scale, Table 6.7, is used to rate the durability based on microscopic inspection of the tested specimen (DePuy, 1965). Advantages claimed are low cost and the short time required for testing. It remains to evaluate this technique by actual project testing (Lutton, Houston and Warriner, 1981).

Interpretation and Evaluation. Performance specifications may be used to translate quantitative lab data into a relative rating of rock durability. For example, the standards used by the U.S. Bureau of Reclamation in testing riprap are shown in Table 6.8. The applicability

Table 6.7 Scale for Visual Estimation of Cavitation Damage (DePuy, 1965, p. 32)

Numerical Durability Rating	Amount of Cavitation Damage
0	Sample completely broken up before 1 minute
1	Sample broken up by end of 1 minute test
2	Severe deterioration
3	Severe general erosion
4	Severe pitting
5	Moderately severe pitting
6	Moderate pitting
7	Minor pitting or some intergranular erosion
8	Some minor intergranular erosion
9	Very slight pitting
10	No visual damage

Table 6.8 U.S. Bureau of Reclamation Proposed Rock Test Criteria (after DePuy, 1965 p. 34)

Test	Quality		
	Poor	Fair	Good
Specific Gravity	2.5	2.5-2.65	2.65
Absorption, %	1.0	0.5-1.0	0.5
Freeze-Thaw Loss, %	5	0.5	0-0.5
Soundness Loss, % (sodium sulfate)	10	5-10	5
Los Angeles Abrasion Loss, % (100 Revolutions)	10	5-10	5
Ultrasonic Cavitation Rating	0-5	5-7	7-10

of laboratory values is limited by the degree to which the tested specimens represent the material.

No one parameter alone is sufficient to qualify rock durability. Several indices must be considered to give a clearer characterization of the rock. It is advantageous to initiate an extensive testing program to maximize the data collected. However, it is often not economically feasible to perform an exhaustive investigation of durability. There is a point beyond which the additional information obtained from further testing does not justify its cost (Franklin, Broch and Walton, 1971). The type and amount of testing to be conducted are determined on a project-specific basis.

Laboratory data and field information must be used together to evaluate a particular rock source. The ultimate value of lab and field data depends on the ability of the evaluators to understand, communicate and utilize the results fully.

Quarrying Procedures and Practices

Quarries are surface excavations in which the rock mined is the end product. The size of rock pieces, or fragmentation, required of the quarrying operation depends, to a large degree, on the intended use of the fragments. In quarrying for rubble mound rock, the sizes required are delineated in the design stage.

Optimum quarry design assures proper breakage and minimizes overshot stone. To attain this goal, the blast designer manipulates many variables, as the type of explosive, the timing of initiation, and the drill pattern, usually according to experimentation and experience. However, the design elements that cannot be altered for a given quarry are the rock properties. The succeeding discussion focuses on the influence of various rock features on the preparation of quarrying operations.

Even in the initial stages of a blasting operation, with the selection of drilling equipment, rock properties are an influential factor in the design of the procedure. A conservative rule of thumb for choosing a drilling method, Table 6.9, qualitatively relates the system utilized to rock strength. The penetration rate of drilling has been used as an indication of the hardness of rock and, therefore, the ease with which it will be fractured (DuPont, 1977). Another gauge to rock hardness is its rating on the Moh's hardness scale. Harder rock is expected to be more difficult to break and should require a drillhole pattern that is relatively closely spaced (Parker, 1971).

The laboratory measured density of intact rock may also indicate the relative difficulty with which the rock will be fragmented. Denser materials may be expected to fracture more readily when explosives with high detonation pressures are used. Less dense, more porous rocks absorb portions of the explosive energy and complicate the control of fragment size and gradation.

The velocity of propagation of body waves induced by a detonation is a dominant characteristic of the rock mass. Velocities measured on rock cores in the laboratory are generally higher than those measured in the field. It is preferable, then, to record this information in situ, where the effects of structural discontinuities in the rock are manifest (USCOE, 1972). As demonstrated in Table 6.10, denser rocks, as granite, tend to exhibit higher longitudinal wave velocities than more porous formations.

System	Resistance of Rock to Penetration			
	Soft	Medium	Hard	Very Hard
Rotary-drag bit	X	X		
Rotary-roller bit	X	X	X	
Rotary-diamond bit	X	X	X	X
Percussive	X	X	X	X
Rotary-percussive	X	X	X	

Table 6.9 Recommended Drilling Systems for Rocks of Different Strengths (USCOE, 1972, p. 4-4)

Properties of Some Explosives

Type of Explosive	Specific Gravity	Detonation Velocity fps	Characteristic Impedance lb/sec/in.^{3}
Nitroglycerin	1.6	26,250	47
Dynamite: 50% Nitroglycerin, 41% Ammonium nitrate, 5% Cellulose	1.5	22,650	38
80% Ammonium nitrate, 10% Nitroglycerin, 10% Cellulose	0.98	13,100	14
ANFO: 93% Ammonium nitrate, 7% Fuel oil	1.0	13,900	15

Properties of Some Rocks

Rock Type	Longitudinal Wave Velocity fps	Characteristic Impedance lb/sec/in.^{3}
Granite	18,200	54
Marlstone	11,500	27
Sandstone	10,600	26
Chalk	9,100	22
Shale	6,400	15

Table 6.10 Some Significant Properties of Explosives and Rocks in Blasting Work (USCOE, 1972, p. 6-3)

Rock breakage by blasting is directly related to the amount and efficiency of energy transfer from the explosive to the rock. This coupling action depends on the relative impedences of the explosive and rock. The impedence of an explosive is calculated as the product of its mass density and detonation velocity. The density of the rock and the velocity of propagation of body waves, discussed previously, are related through the characteristic impedence of the rock. This term has been defined as the product of the mass density of the rock and the velocity of the longitudinal or P-waves through the mass. Explosives with impedences nearly matching the characteristic impedence of the rock transfer energy most efficiently to the encompassing material. The properties of various explosives and rock materials are given in Table 6.10. Based on these values, combinations which suggest coupling possibilities, or the efficient transfer of explosive energy, are ammonium nitrate and shale, and nitroglycerin and granite.

Fragmentation is intrinsically related to the compressive and tensile strength properties of the rock. The efficiency of an explosive in a given circumstance is often related to strength characteristics in the form of a "powder factor", expressed in pounds of explosive per yard or ton of rock broken. Using an empirical relation, as in Figure 6.4, the blasthole pattern may be devised (USCOE, 1972).

Various formulas have been proposed to aid in the initial design of blasthole array. Many such expressions recognize the influence of rock characteristics and incorporate either an experimental "rock factor" or actual strength parameters. Although the equations do not yield exact values, such values are not essential to practical blasting conditions (Gregory, 1979).

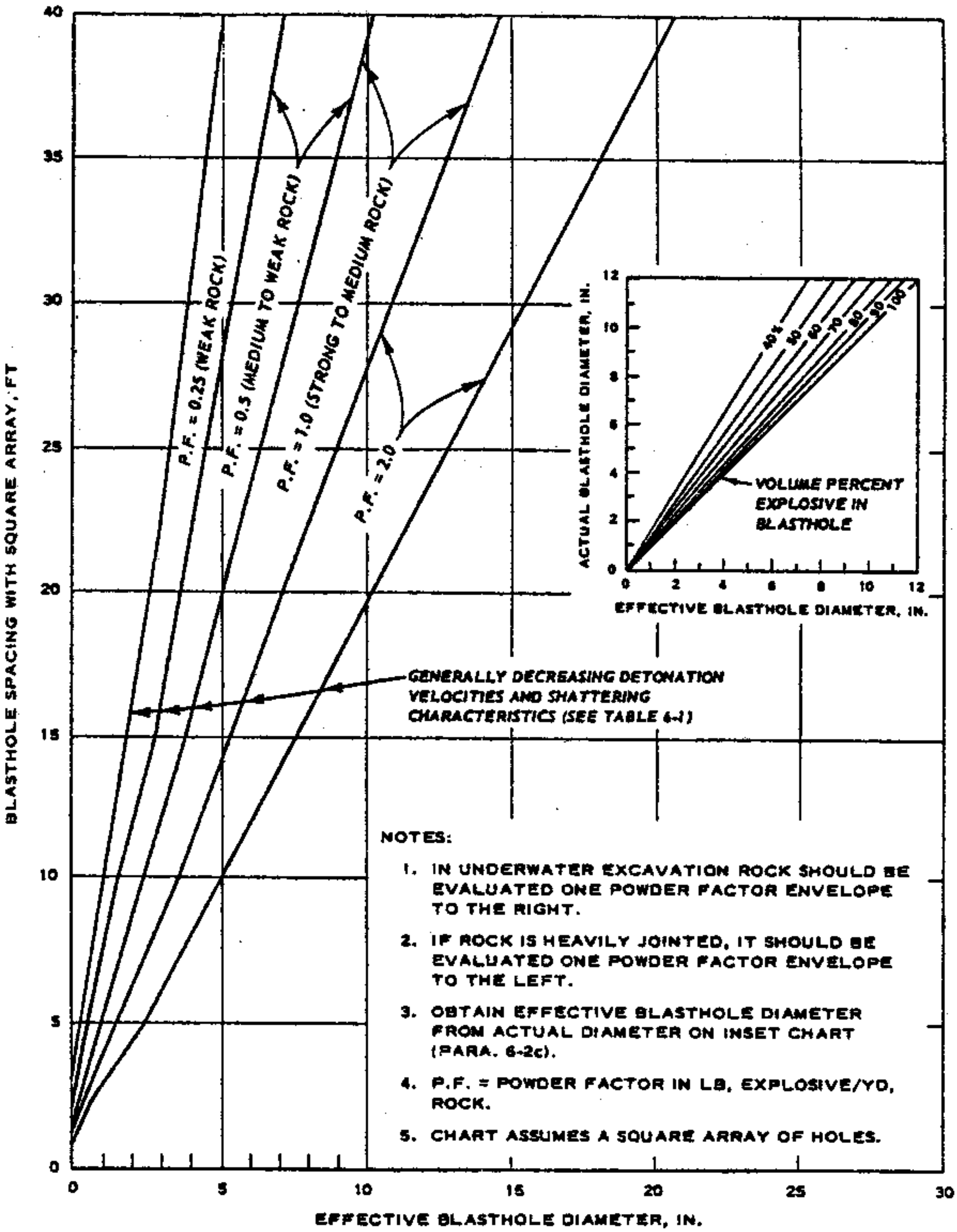


Figure 6.4 Blasthole Spacing and Diameter and Powder Factor for Multiple Row Blast Pattern in Rocks of Different Strengths (USCOE, 1972, p. 6-5)

In general, the most economical blast design is one which conforms to the inherent structure of the rock mass. Natural joints represent planes of least resistance to rupture by blasting. Less explosive energy is required to fracture a rock mass along the joints. This is further demonstrated by noting that the P-wave velocity and, thus, the characteristic impedence of jointed material are relatively lower than the values for unfractured rock. Therefore, the jointed material can be coupled with an explosive having a lower impedence.

Bedding planes in sedimentary rock and directions of foliation in metamorphic rock are similar planes of weakness. In flat-lying sedimentary formations, horizontally stratified with horizontal joints and one or two sets of vertical joints, the quarry face can be developed parallel to major vertical joints. With this configuration, it may be expected that blast fracturing will naturally occur in the direction of the free face. Also, less explosive force is necessary to maintain a horizontal bench (Langefors and Kihlstrom, 1978). Even under these seemingly ideal conditions, care must be exercised in planning the blast design. Over-charging can lead to gas migration along the natural fractures and initiate overbreakage into the quarry (USCOE, 1972).

Blasting may cause slope failure along a set of joints which is steeply inclined into the quarry, as illustrated in Figure 6.5. When the planes of weakness are pitched in this manner, it is preferable to develop the free surface no less than 45 degrees to the direction of weakness. Where the face must, for practical reasons, be developed parallel to the planes of weakness, reduced burdens and spacings as well as angle drilling are feasible measures to be enacted (DuPont, 1977).

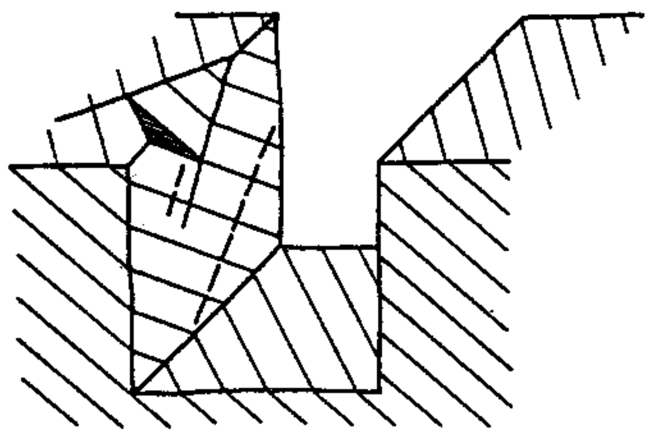


Figure 6.5 Adverse Dip of Joints into Excavation (USCOE, 1972, p. 6-10)

Rock fabric may be considered, for the purpose of blast design, as the mineralogical or granular texture that can impart anisotropy to the rock mass (USCOE, 1972). Inherent cleavage planes, construed as planes of weakness, can be used advantageously in breaking the rock. Consistent with foregoing considerations, optimum blast design enables the fracture of rock along such natural planes of weakness.

The production schedule and "curing" time must be carefully planned to minimize placement of stone with a high potential for breakage. Lienhart and Stransky (1981) point out wintertime production problems. "Popping" of quarried rock can result from loss of confining pressure on frozen pore water. Freeze-thaw fracturing may occur because of lack of curing time. In addition, stone is particularly brittle at low temperatures, and more susceptible to blast-induced damage. The authors suggest that all stone be stockpiled for six months prior to its placement to assure that it is free draining. They recommend that blasting for large size stone be performed only during non-freezing weather.

The ultimate choice of blasting procedure is generally biased by judgment, based on experience. The rock mass composition is unique for each site, however, and experience alone cannot provide the input necessary to formulate blast design. Awareness of the influences of rock properties on the efficiency of breakage can aid in optimizing the quarry design. This overriding effect of rock characteristics accentuates the need for well-planned and thoroughly reported exploration and testing programs. Other interrelated concerns in the economics of quarrying and material utilization are shown in Figure 6.6.

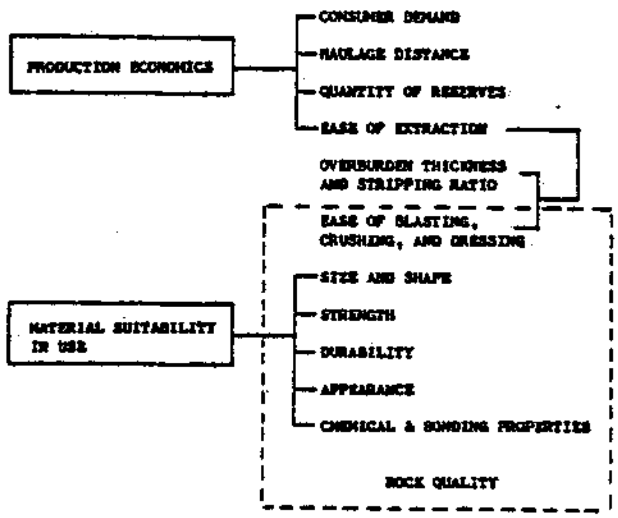


Figure 6.6 Factors Affecting Economics of Quarrying (Franklin, 1974, p. 2.2)

6.2 CONCRETE ARMOR UNITS

Over the past 25 to 30 years there has been a trend toward construction of larger rubble mound structures, exposed to deeper and rougher seas. At these sites of more severe wave attack, natural cover layer stone ~~of~~ the required size may be unavailable or uneconomical to procure. A variety of concrete shapes have been developed for use as armor units in such instances. Concrete units present two distinct advantages:

1. As they are specially cast, units of the precise weight specified by cover layer design can be manufactured.

2. They generally have higher stability coefficient (K_D) values than quarrystone (See Table 7.1), enabling a reduction in required weight or the steepening of structure side slopes (See Equation 7.2).

This section forms an introduction to the concrete armor units available and the associated technology. The durability and production of concrete units are reviewed briefly. Many of the weathering characteristics of concrete are similar to those for rocks, presented in Section 6.1. A more detailed study of concrete in the marine environment may be consulted in Hubbell and Kulhawy (1979a).

Concrete Armor Shapes

The first non-block concrete armor unit, the tetrapod, was devisd in Grenoble, France in 1950. With the development of Hudson's stability formula for armor units (Equation 7.2) in 1953, the advantages of using specially shaped concrete elements became apparent and were generally acknowledged. Over the past three decades, a substantial number of concrete armor unit shapes have been proposed throughout the world.

Most of these shapes are noted in Table 6.11, with the corresponding dates and locations of development.

The U.S. Army Corps of Engineers have used quadripods, tetrapods, tribars and dolosse in breakwater and jetty construction. They recommend the latter three shapes for use in rubble mound design (CERC, 1977). These more common concrete units are shown in Figure 6.7. Many other designs are illustrated in Hudson (1974).

The selection of one armor unit shape from among those available depends in large part on the stability characteristics of the unit. Dolosse exhibit the highest stability coefficient (See Table 7.1) and are considered by many as the superior precast cover layer unit (Hudson, 1974). It is preferable that the hydraulic stability of the selected unit be substantiated through laboratory testing. Some of the shapes listed in Table 6.11 have little laboratory or prototype data to recommend their use. Economic factors also play a major role in the choice of a specific armor unit shape. Attention should be given to the availability of forms and necessary royalty costs. For private use of those units covered by a United States patent agreement, the license holder must be paid a royalty per cubic yard of concrete used (Hudson, 1974). Other pertinent considerations are reviewed in Table 6.1.

Quality of Concrete

Environmental effects that cause deterioration of concrete in the coastal zone are detailed by Hubbell and Kulhawy (1979a). Key among these are attack by destructive chemicals, abrasion from ice, debris, wind and waves, disintegration due to the freezing of pore water, and sea water corrosion of reinforcing steel. In the case of armor units,

Name of Unit	Development of Unit	
	Country	Year
Akmon	Netherlands	1962
Bipod	Netherlands	1962
Cob	England	1969
*Cube	———	†
*Cube (modified)	United States	1959
*Dolos	South Africa	1963
Dom	Mexico	1970
Gassho Block	Japan	1967
Grabbelar	South Africa	1957
Hexaleg Block	Japan	—
*Hexapod	United States	1959
Hollow Square	Japan	1960
Hollow Tetrahedron	Japan	1959
N-Shaped Block	Japan	1960
*Pelican Stool	United States	1960
*Quadripod	United States	1959
*Rectangular Block	———	†
Stabilopod	Romania	1965
Stabit	England	1961
*Sta-Bar	United States	1966
*Sta-Pod	United States	1966
Stalk Cube	Netherlands	1965
Svee Block	Norway	1961
*Tetrahedron (solid)	———	‡
*Tetrahedron (perforated)	United States	1959
Tetrapod	France	1950
Toskane	South Africa	1966
Tribar	United States	1958
Trigon	United States	1962
Tri-Long	United States	1968
Tripod	Netherlands	1962

* The units have been tested, some extensively, at the Waterways Experiment Station (WES).

† 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.

‡ Solid tetrahedrons are known to have been used in hydraulic works for many years. This unit was tested at WES in 1959.

Table 6.11 Concrete Armor Units
(CERC, 1977, p. 7-194)

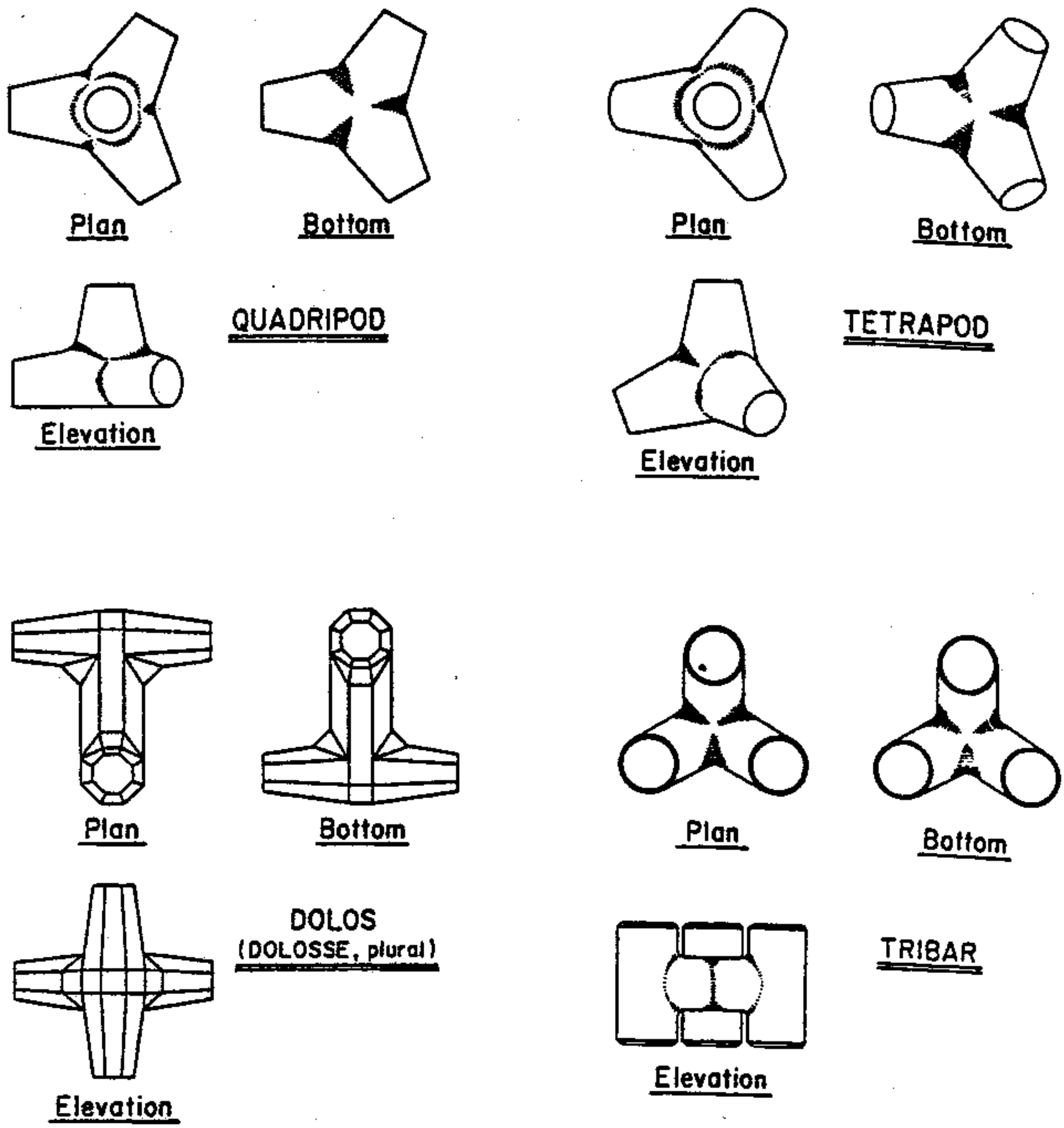


Figure 6.7 Concrete Armor Units (CERC, 1977, p. 7-195)

excessive stresses may also be induced during their transportation and placement onto the rubble mound, by various impact forces, or simply by the movement of units in response to storm wave attack. The concrete must be designed to withstand, as much as is practicable, these degrading forces.

Dense, watertight concrete is most resistant to environmental attack. Mix design considerations include the selection of aggregates, the water-cement ratio, and admixtures. Use of air-entraining cement agents is desirable (Hubbell and Kulhawy, 1979a). Careful workmanship during the mixing and casting processes will result in a higher quality product.

Curing duration is an important factor. Premature stripping of forms can cause cracks in the concrete units. Commonly, a greater number of bottom forms are provided, so that the top form can be removed and reused while the concrete undergoes additional curing in the bottom form. The units are moved to storage only after their strength is judged adequate to prevent cracking during handling and stockpiling operations (Hudson, 1974). In addition, the concrete strength at 28 days should be specified. Table 6.12 lists typical values.

Structural Design

From the manufacture and storage phases through transportation, placement and in-service performance, the forces to which concrete armor units are subjected are numerous and complex. This matrix of stresses which the units must withstand is, at present, poorly understood. Modeling and measuring the forces on individual units, though not impossible, is extremely difficult (Davidson and Markle, 1976). Thus,

Table 6.12 Typical Specified Strengths of Concrete Armor Units

Concrete Strength at 28 Days		Comments
psi	kN/m^2	
3500	24130	for Stabits - Singh, 1968
5000	34475	General - Hudson, 1974
5800	39990	for 42 ton dolosse, Humboldt Bay Jetties - Magoon and Shimizu, 1971
6100	42060	for 42 ton dolosse, Humboldt Bay Jetties - Magoon and Shimizu, 1971

there are insufficient data to enable the design of concrete armor units based on rational structural analyses (Hudson, 1974).

As the overall rubble mound stability depends on the weight, shape and interlocking characteristics of the cover layer elements, it is essential that structural integrity of the armor units be maintained. If the random breakage of units exceeds 15 percent of the number of dolos armor units in the top layer, for example, the rubble mound stability will be diminished (Davidson and Markle, 1976). Most breakage occurs during the manufacture, storage and placement of the shapes. Although there is no infallible method to end such damage, some modification in production of the units may be indicated. For example, Stabits are successfully moved by a simple double sling arrangement. Tensile stresses do not form since all structural members are kept in compression (Singh, 1968). To reduce breakage in dolosse, small curved fillets at the intersections of flukes and shanks are proposed by Lillevang and Nickola (1976). This modification would reduce excessive stress concentrations and minimize concrete imperfections.

The need for reinforcement in concrete armor units is the subject of ongoing debate. Lillevang and Nickola (1976) concluded that steel bar reinforcement would have to be placed quite close to the surface of a dolos to prevent fracturing. Corrosion of the steel by sea water could then occur. Also, use of the bars could induce shrinkage cracks during the hydration of cement in the concrete. With a similar opinion, Davidson and Markle (1976) indicated that the value of reinforcing steel in reducing dolos breakage is questionable. Based on experience at several coastal structures using large concrete armor units, Magoon and Shimizu (1971) recommended incorporation of reinforcing steel in 20 ton dolosse used on the Humboldt Jetties, California. The authors acknowledged that additional research is needed on this subject; little factual information is available to quantify the decision to reinforce.

6.3 SUMMARY

Quarrystone or concrete elements can serve as rubble mound cover layer units. Rock is the most common mound construction material and, when available, is usually the most economical choice. Many larger breakwater and jetty projects have been armored with specially formed concrete shapes. These have hydraulic stability properties superior to those of quarrystone and can therefore be relatively smaller, for equal wave protection, than rock.

Chemical, mechanical and biological weathering forces combine to cause high rates of deterioration in susceptible cover layer materials. Weathering mechanisms in the intertidal zone are especially degrading. The structural stability of the selected armor material must be maintained for the service life of the structure. The durability of rock in the coastal environment has been examined in depth in this chapter; durability aspects of concrete are more thoroughly covered in Hubbell and Kulhawy (1979a).

Rock durability is evaluated by field and laboratory studies. The scope of investigations depends on the scale of the project, construction site conditions, and properties of the material and source. These assessments must be performed and interpreted by qualified personnel. All available data are considered simultaneously in the final analysis to provide the most informed judgment regarding durability. Rock properties are also important in their effect on material availability. Quarry design for optimum rock breakage must be planned with consideration to fundamental rock characteristics.

The durability of concrete armor units is assured through proper mix design and careful production procedures. Unit breakage during manufacture, storage and placement on the mound must be minimized to attain design stability at a competitive cost.

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.

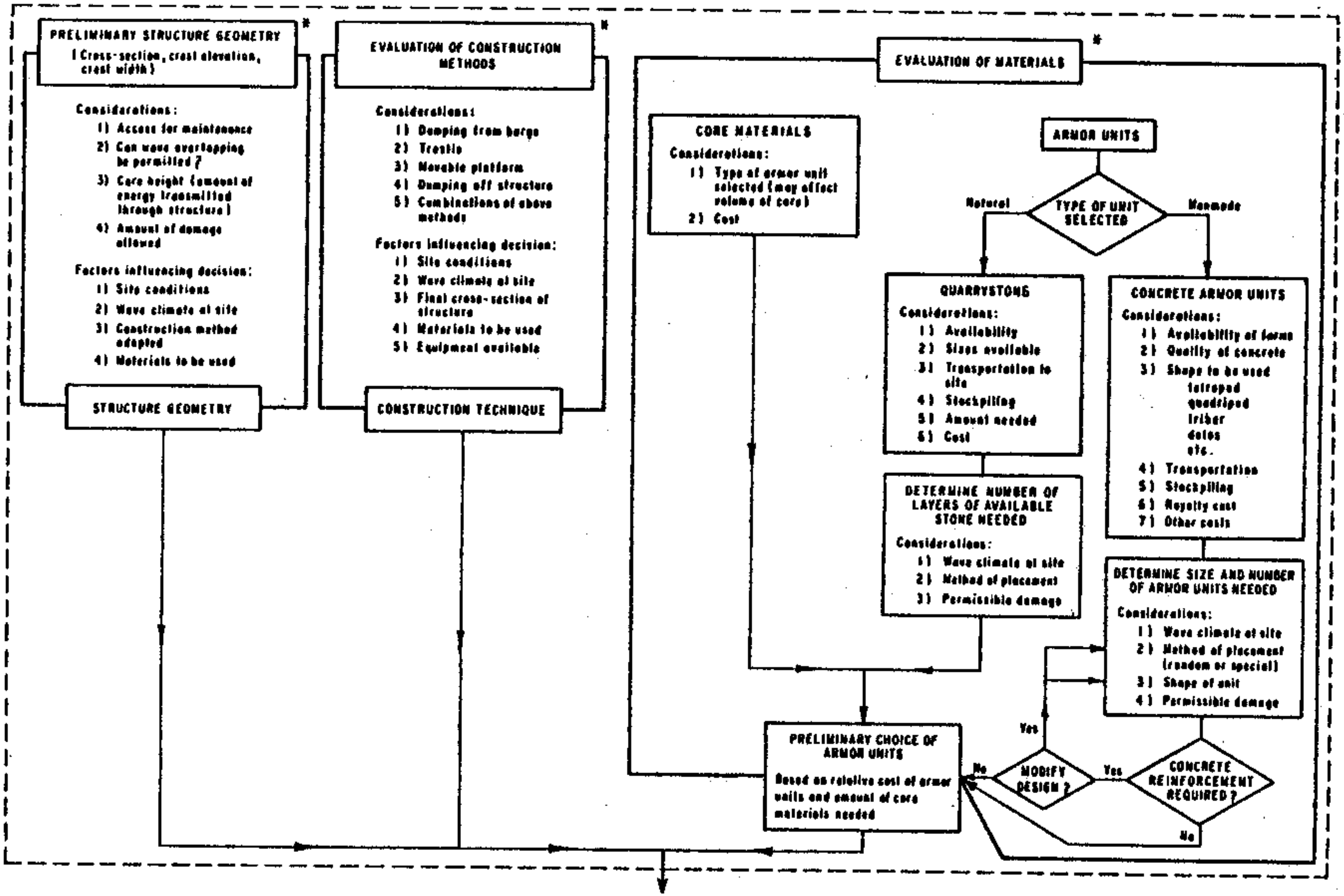


Figure 7.1b

Figure 7.1a Flow Diagram for the Design of Rubble Mound Structures (CERC, 1977, p. 7-205)

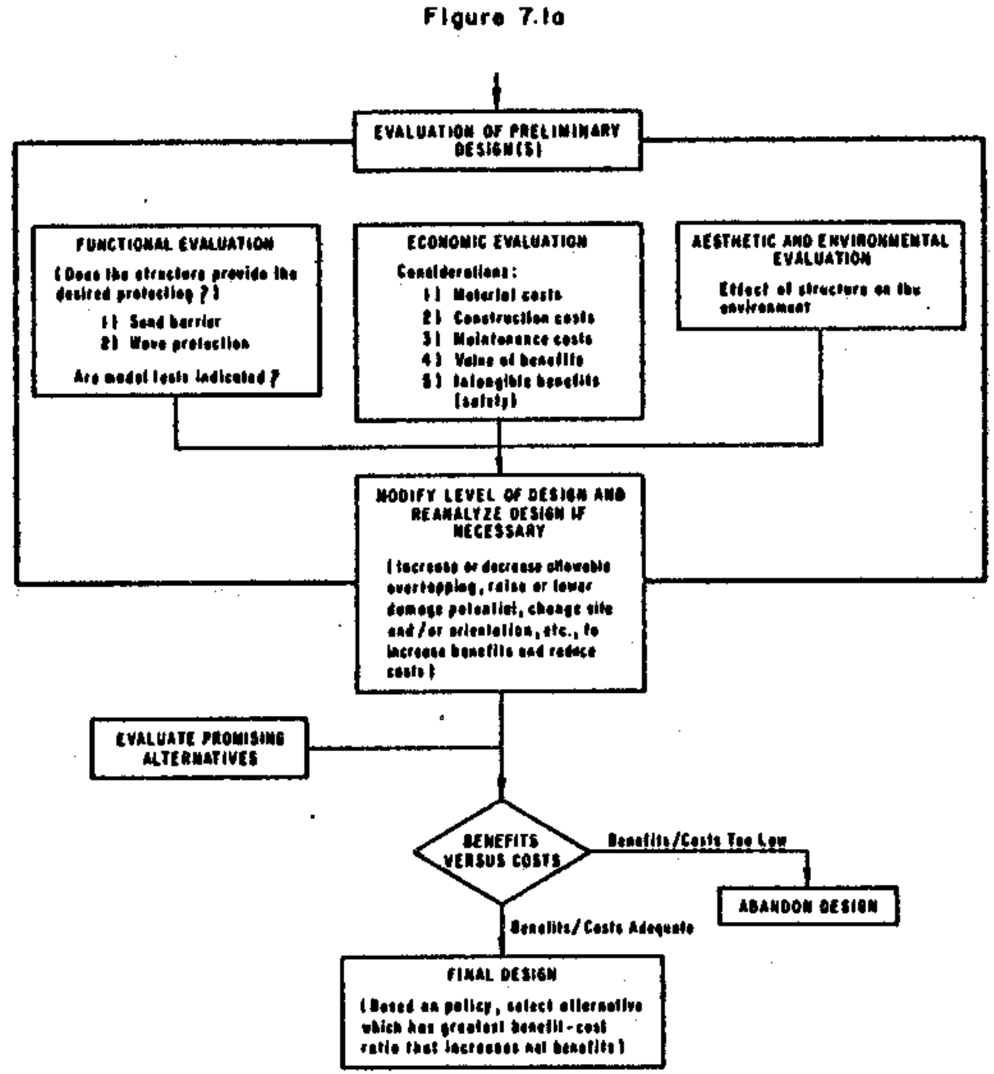


Figure 7.1b Flow Diagram for the Design of Rubble Mound Structures (CERC, 1977, p. 7-206)

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:

$$W = \frac{H^3 \gamma_r}{K_D (S_r - a)^b \ f(\theta)} \tag{7.1}$$

in which: W = weight of individual armor units; H = design wave height; γ_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. γ_w = unit weight of water, fresh water = 62.4 pcf, sea water = 64.0 pcf; θ = angle of seaward slope measured from horizontal; K_D = stability coefficient. The form of the function $f(\theta)$ depends on the force diagram assumed in the derivation. 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:

$$W = \frac{H^3 \gamma_r}{K_D (S_r - 1)^3 \cot \theta} \tag{7.2}$$

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 quarrystones, the sizes can range within 0.75 to 1.25W, with 75 percent of the individual stones weighing more than W.

2. The cover layer slope angle, θ, 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

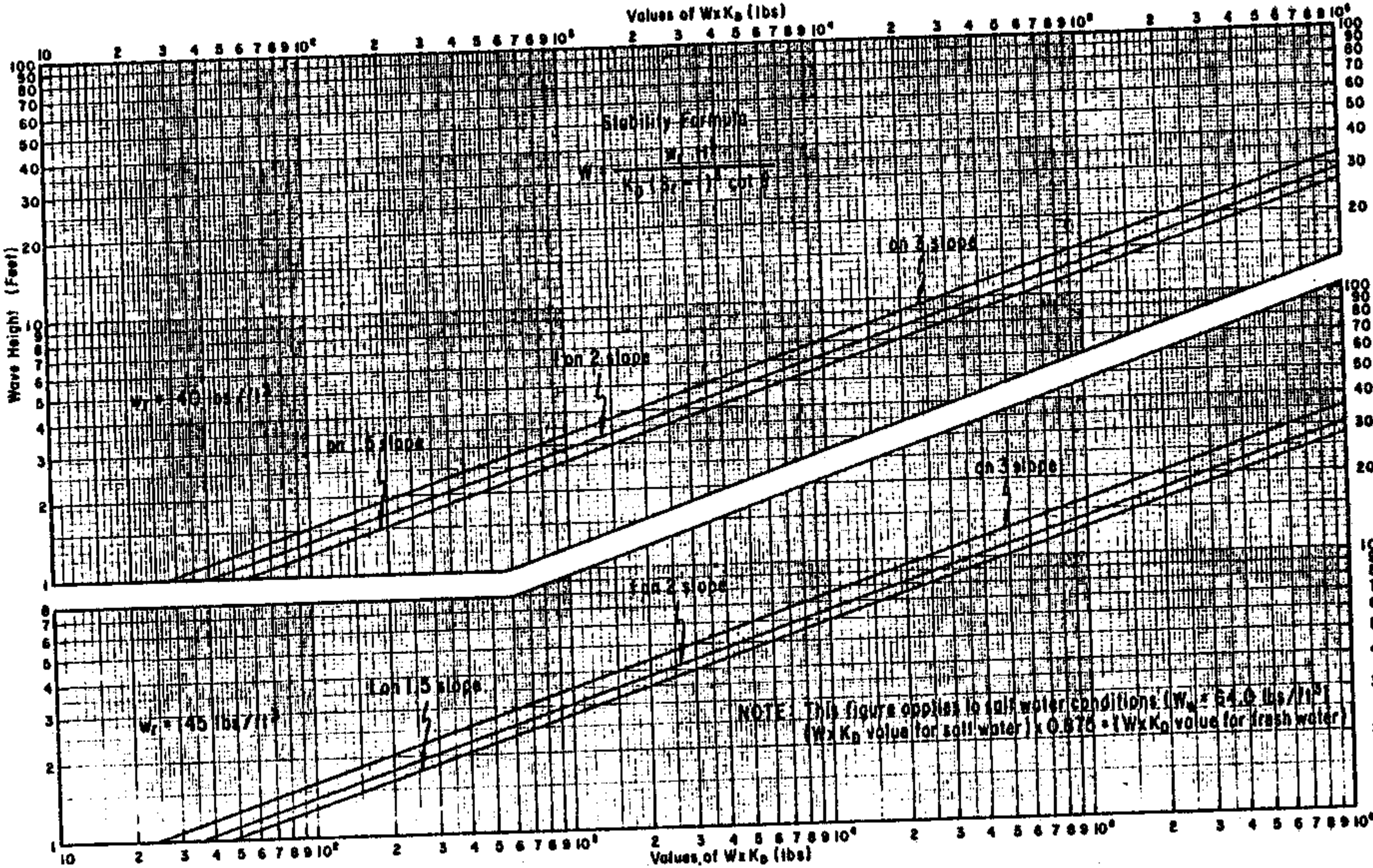


Figure 7.2 Armor Unit Weight x K_D versus Wave Height for Various Slope Values (γ_r = 140 pcf and 145 pcf) (CERC, 1977, p. 7-182)

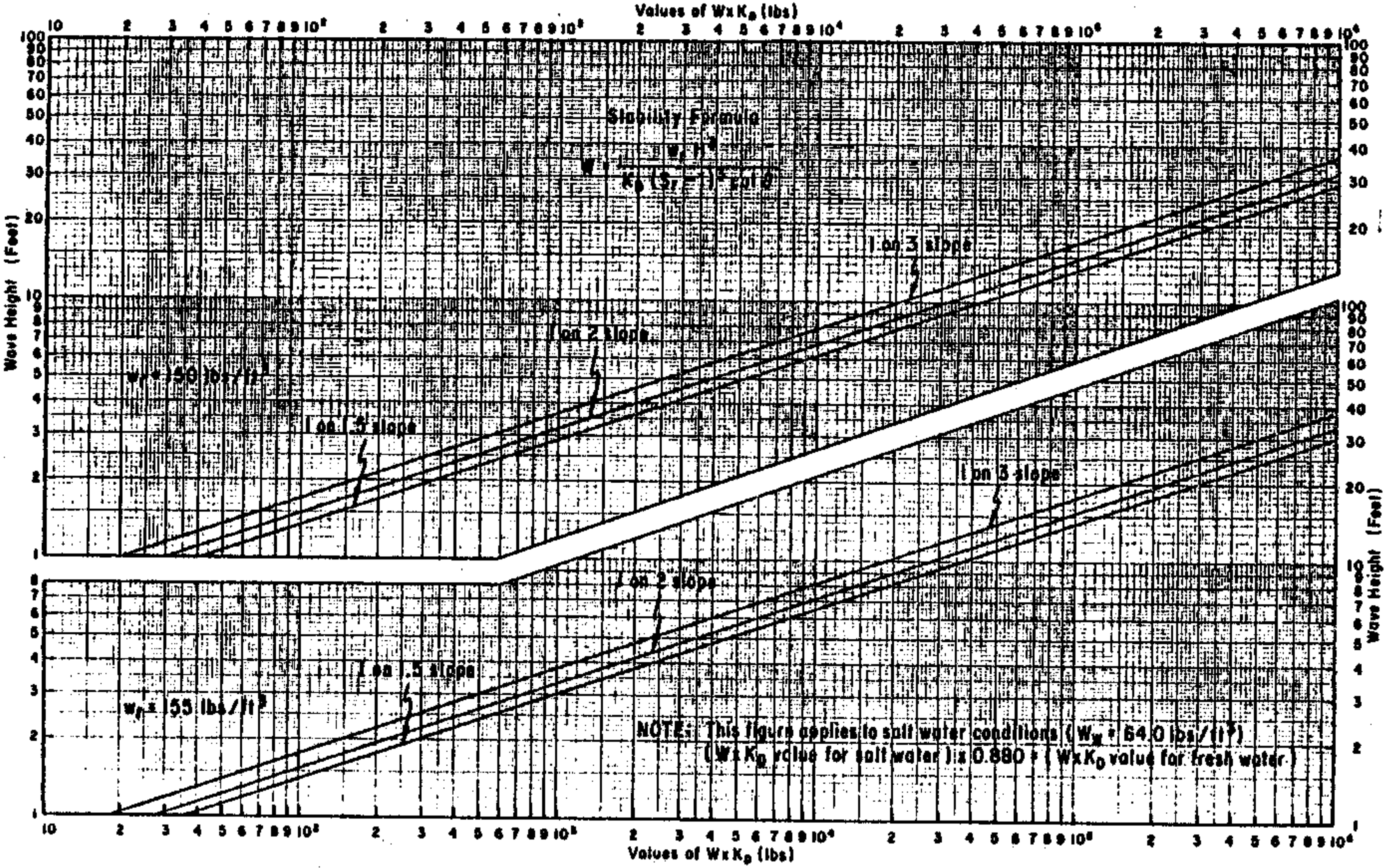


Figure 7.3 Armor Unit Weight x K_D versus Wave Height for Various Slope Values (γ_r = 150 pcf and 155 pcf) (CERC, 1977, p. 7-183)

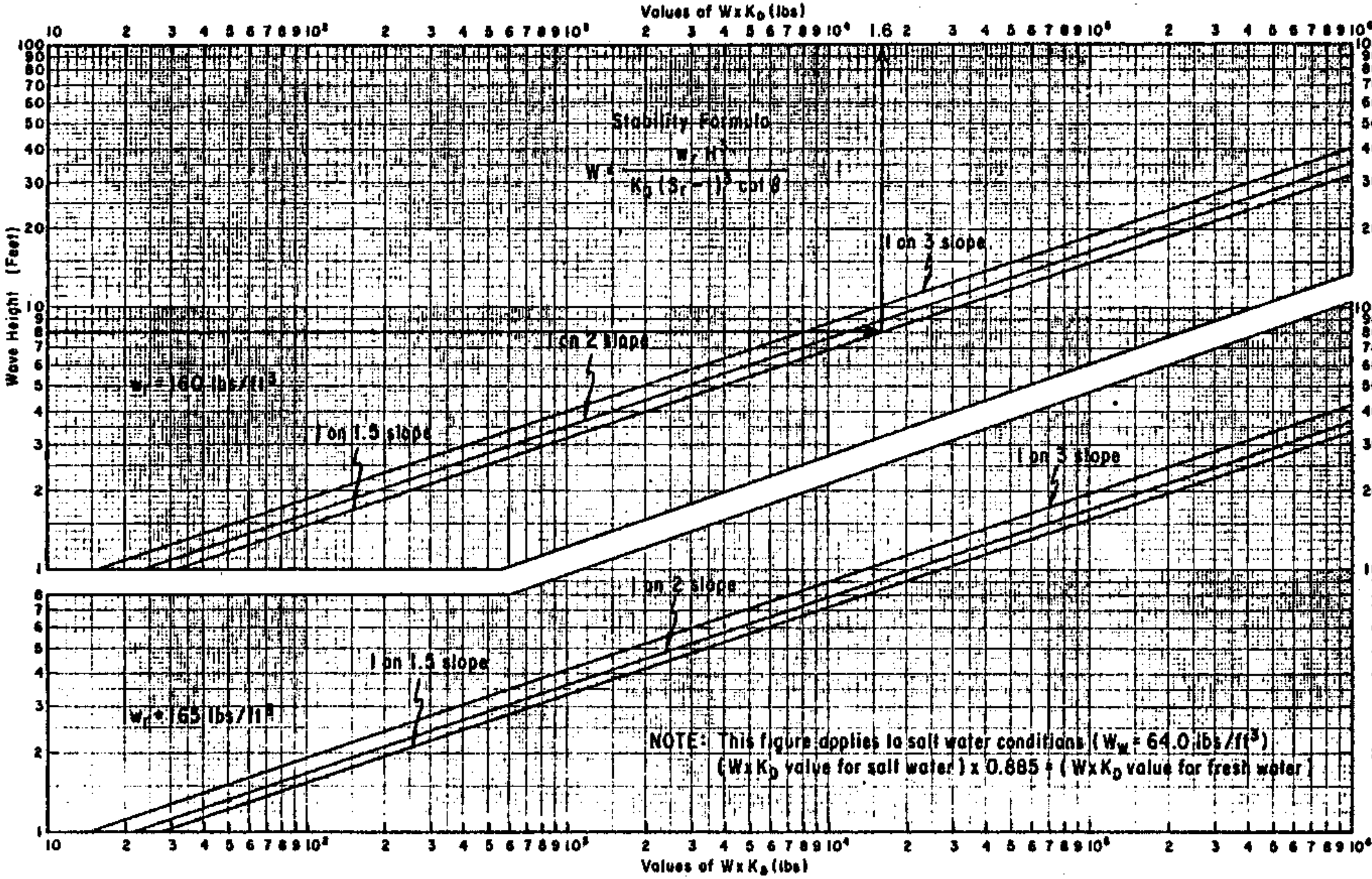


Figure 7.4 Armor Unit Weight x K_D versus Wave Height for Various Slope Values (γ_r = 160 pcf and 165 pcf) (CERC, 1977, p. 7-184)

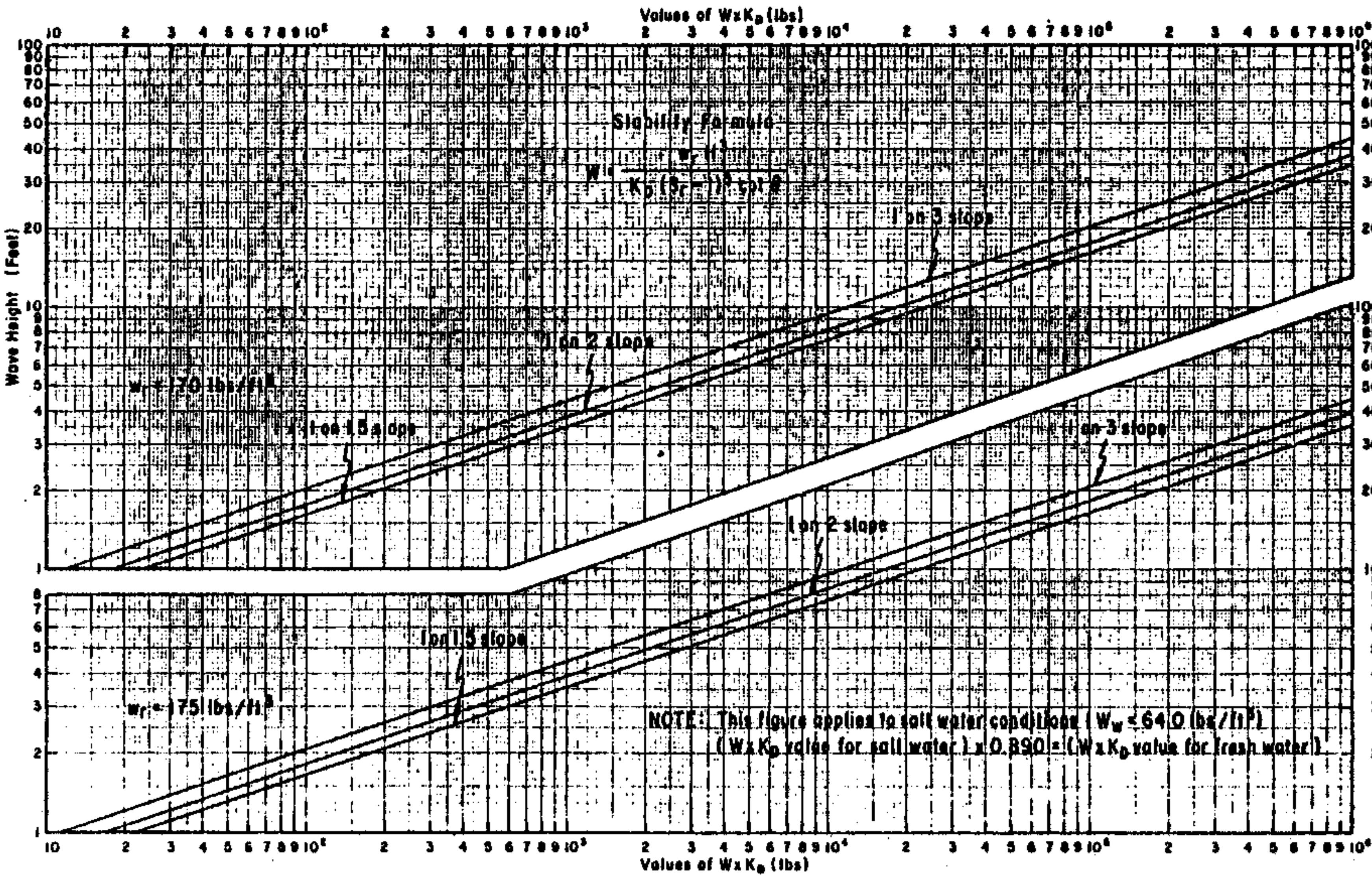
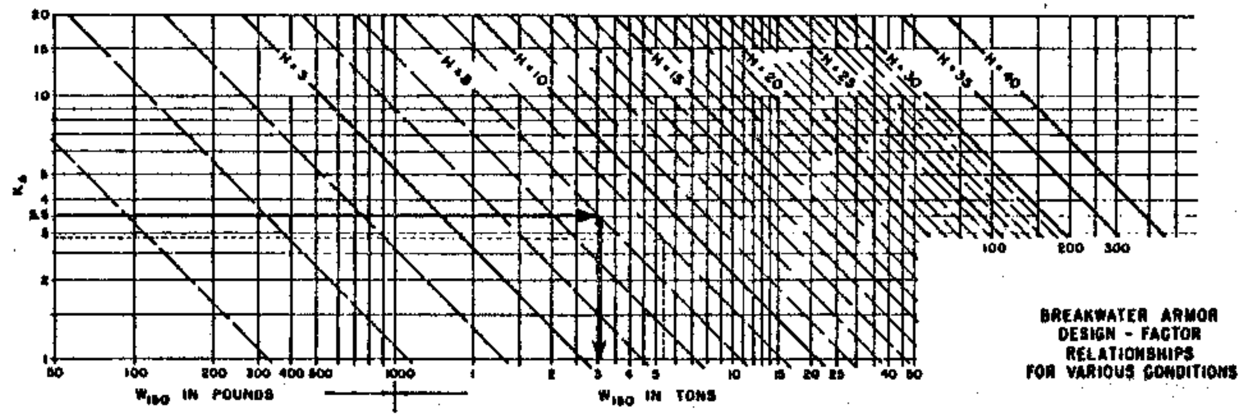


Figure 7.5 Armor Unit Weight x K_D versus Wave Height for Various Slope Values (γ_r = 170 pcf and 175 pcf) (CERC, 1977, p. 7-185)



γ , lb/ft^3

Cot θ	150	152	154	156	158	160	162	164	166	168	170	172	174	176	178	180
1.5	100	95	90	85	81	77	73	70	67	64	61	58	55	53	51	49
2.0	75	71	67	64	61	58	55	52	50	47	45	43	41	40	38	37
2.5	60	57	54	51	48	46	44	42	40	38	36	35	33	32	31	29
3.0	50	47	45	42	40	38	37	35	33	31	30	29	28	27	26	25
3.5	43	40	38	36	34	33	31	30	28	27	26	25	24	23	22	21

Percentage of W_{150}

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

$\cot\ \theta = 1.5$ (1 ON 1.5 SLOPE)

FRESH WATER, $\gamma_w = 62.4$ PCF

REQD : COVER LAYER ARMOR UNIT WEIGHT, W, BY :

a) EQUATION 7.2 DIRECTLY

b) CERC GRAPHICAL METHOD (FIGURES 7.2 TO 7.5)

c) ASCE (1969) GRAPHICAL METHOD (FIGURE 7.6)

SOLUTION :

FROM TABLE 7.1, $K_D = 3.5$

a) $$W = \frac{\gamma_r H^3}{K_D (S_r - 1)^3 \cot\theta}$$

$$= \frac{160\ (8^3)}{3.5\left(\frac{160}{62.4} - 1\right)^3\ 1.5} = 4078^{\#} = 2.04 \text{ TONS}$$

$\therefore \underline{W \simeq 2.0 \text{ TONS}}$

b) USING FIGURE 7.4, TOP (FOR $\gamma_r = 160$ PCF),

$H = 8$ FT , $\cot\ \theta = 1.5$,

READ $W \times K_D = 1.6 \times 10^4\ ^{\#}$

FOR FRESH WATER, MULTIPLY BY 0.875 $\rightarrow W \times K_D = 1.4 \times 10^4$

$$W = \frac{W \times K_D}{K_D} = \frac{1.4 \times 10^4}{3.5} = 4000^{\#} = \underline{2.0 \text{ TONS} = W}$$

c) USING FIGURE 7.6, ENTER GRAPH AT LEFT FOR $K_D = 3.5$, $H = 8$ FT ; READ $W_{156} = 3.1$ TONS

USING BOTTOM CHART, FOR $\gamma_r = 160$ PCF, COT $\theta = 1.5$;

READ % OF $W_{156} = 77$

$\therefore W = 77\% \; W_{156} = (0.77)(3.1) = 2.39$ TONS

FOR FRESH WATER, MULTIPLY BY 0.875 $\longrightarrow$ $\underline{\underline{W = 2.1 \text{ TONS}}}$

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

No-Damage Criteria and Minor Overtopping							
Armor Units	n *	Placement	Structure Trunk		Structure Head		
			K_D §		K_D		Slope
			Breaking wave	Nonbreaking wave	Breaking wave	Nonbreaking wave	cot θ
Quarrystone							
Smooth rounded	2	random	2.1	2.4	1.7	1.9	1.5 to 3.0
Smooth rounded	>3	random	2.8	3.2	2.1	2.3	‖
Rough angular	1	random †	†	2.9	†	2.3	‖
Rough angular	2	random	3.5	4.0	2.9 2.5 2.0	3.2 2.8 2.3	1.5 2.0 3.0
Rough angular	>3	random	3.9	4.5	3.7	4.2	‖
Rough angular	2	special ‡	4.8	5.5	3.5	4.5	‖
Tetrapod and Quadripod	2	random	7.2	8.3	5.9 5.5 4.0	6.6 6.1 4.4	1.5 2.0 3.0
Tribar	2	random	9.0	10.4	8.3 7.8 7.0	9.0 8.5 7.7	1.5 2.0 3.0
Dolos	2	random	22.0	25.0	15.0 13.5	16.5 15.0	2.0 ¶ 3.0
Modified Cube	2	random	6.8	7.8	—	5.0	‖
Hexapod	2	random	8.2	9.5	5.0	7.0	‖
Tribar	1	uniform	12.0	15.0	7.5	9.5	‖
Quarrystone (K_{RR}) Graded angular	-	random	2.2	2.5			

* 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

Unit		Damage (D) in Percent						
		0 to 5	5 to 10	10 to 15	15 to 20	20 to 30	30 to 40	40 to 50
Quarrystone (smooth)	$H/H_{D=0}$	1.00	1.08	1.14	1.20	1.29	1.41	1.54
	K_D	2.4	3.0	3.6	4.1	5.1	6.7	8.7
Quarrystone (rough)	$H/H_{D=0}$	1.00	1.08	1.19	1.27	1.37	1.47	1.56
	K_D	4.0	4.9	6.6	8.0	10.0	12.4	15.0
Tetrapods & Quadripods	$H/H_{D=0}$	1.00	1.09	1.17	1.24	1.32	1.41	1.50
	K_D	8.3	10.8	13.4	15.9	19.2	23.4	27.8
Tribar	$H/H_{D=0}$	1.00	1.11	1.25	1.36	1.50	1.59	1.64
	K_D	10.4	14.2	19.4	26.2	35.2	41.8	45.9

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)

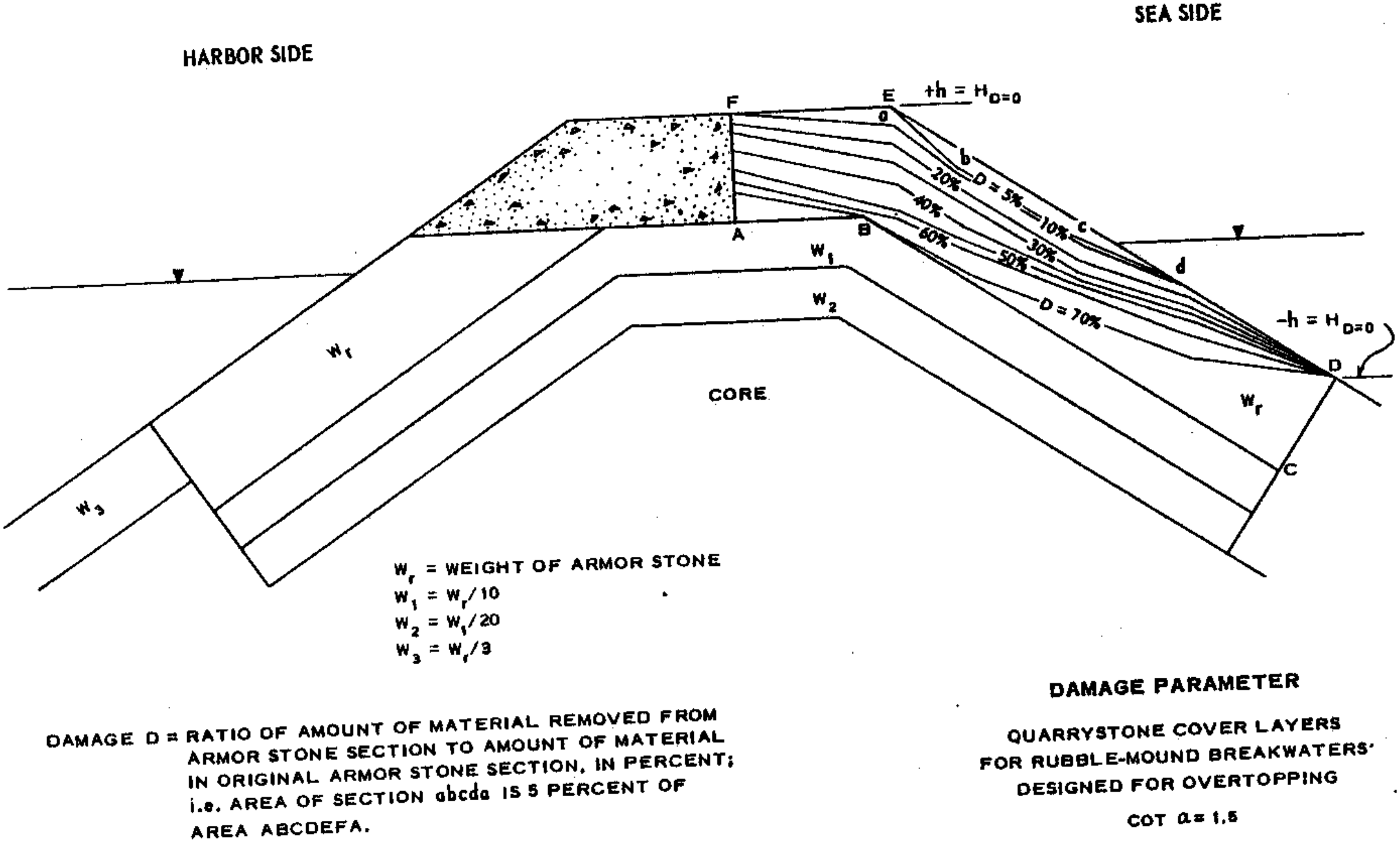


Figure 7.7 Damage Parameter, D, for a Typical Model Test Section (Jackson, 1968a, pl. 21)

1/2

DESIGN EXAMPLE 7.2

SELECTION AND USE OF K_D COEFFICIENTS

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

REQD : a) FOR NO-DAMAGE DESIGN, DETERMINE K_D. DETERMINE THE DAMAGE ANTICIPATED FROM 12 FT WAVES.

b) IT IS DESIRED TO ALLOW 20 TO 30 % DAMAGE. DETERMINE H AND K_D TO BE USED IN THIS DESIGN.

SOLUTION :

a) EXCEEDING NO-DAMAGE CONDITION :

$K_D = 2.4$ (FROM TABLE 7.1 OR 7.2)

$$H = 12 \text{ FT}, \quad \frac{H}{H_{D=0}} = \frac{12}{10} = 1.2$$

FROM TABLE 7.2, FOR SMOOTH QUARRYSTONE, THIS VALUE CORRESPONDS TO 15 TO 20 % DAMAGE.

∴ IF THE STRUCTURE IS DESIGNED FOR NO-DAMAGE $H_{D=0} = 10$ FT AND $K_D = 2.4$, 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 %,

$\frac{H}{H_{D=0}} = 1.29 \quad \therefore H = 1.29\,(10) = 12.9$ FT

AND $K_D = 5.1$

∴ IF THE STRUCTURE IS DESIGNED FOR $H_{D=0} = 10$ FT AND $K_D = 2.4$, WAVES OF $H = 12.9$ FT COULD CAUSE COVER LAYER DAMAGE OF 20 TO 30 %.

ALTERNATIVELY, IF THE DESIGN USED $H = 10$ FT AND $K_D = 5.1$, 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 quarrystone 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

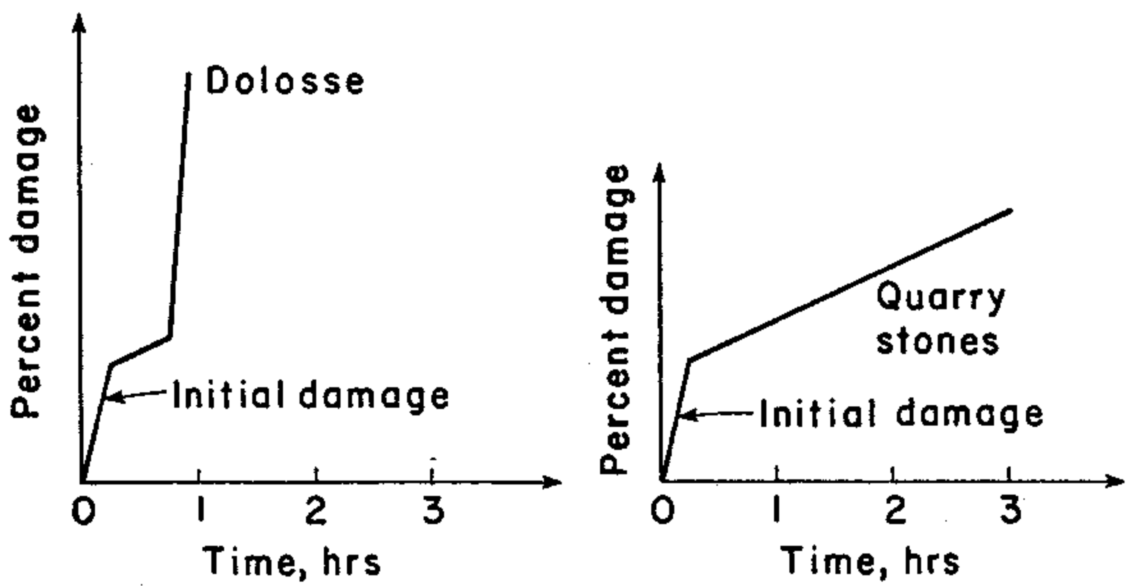


Figure 7.8 Damage Development for Quarrystones and Dolosse for Oblique (70-80°) Incident Waves (Gravesen, Jensen and Sorensen, 1979, p. 16)

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 concrete 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.

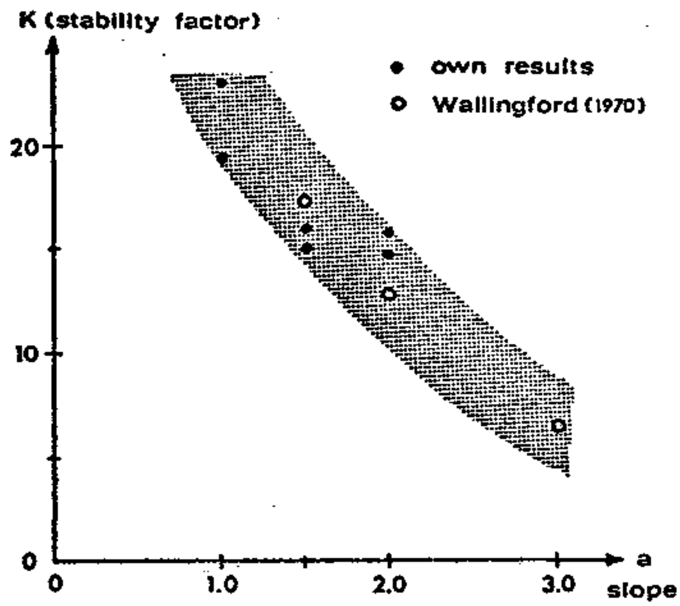


Figure 7.9 Variation of the Stability Coefficient with Structure Slope for Dolosse (Brorsen, Burcharth and Larsen, 1974, p. 1699)

Table 7.3 Representative Values of Angle of Repose for Various Armor Units and Model Units (Gravesen, Jensen and Sorensen, 1979, p. 9)

Armor Unit	Angle of Repose $\mu = \tan \phi$	ϕ	ϕ for Model Units
Quarrystones	1.1	48°	
Cubes	1.2	50°	
Tetrapods	∿1.4	∿55°	54° plastic 58° concrete
Dolos	∿2.7	∿70°	79° concrete mortar 70° octangular plastic 68° round plastic

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:

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

where ϕ is a variable quantity. Based on preliminary, inconclusive tests, ϕ 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, γ_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:

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

where γ_a corresponds to the standard values in Figure 7.10:

γ_a concrete = 150 pcf

γ_a quarrystone = 165 pcf

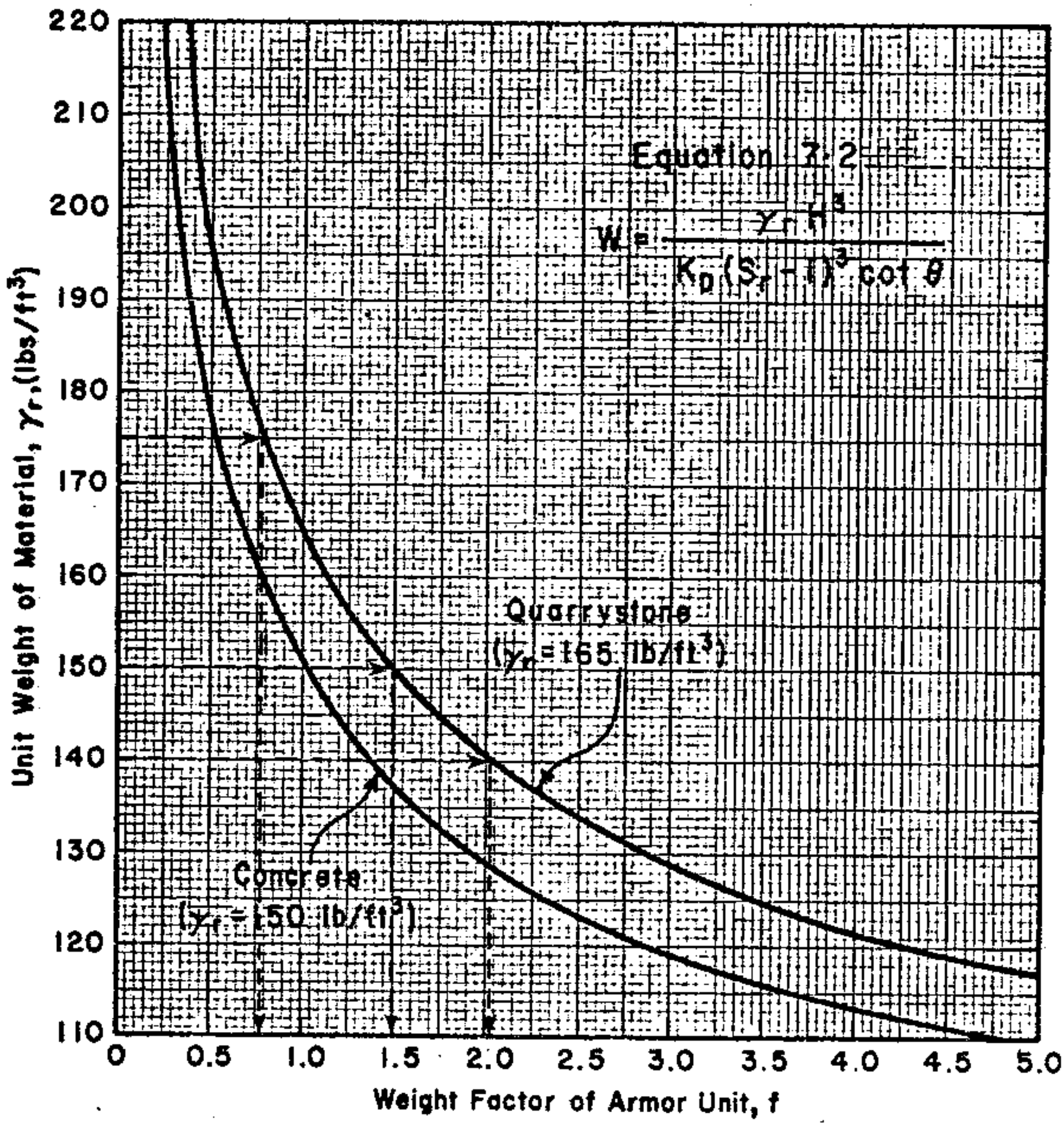


Figure 7.10 Effect of Unit Weight Changes on Required Armor Unit Weight (CERC, 1977, p. 7-192)

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. Quarrystone 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

1/1

DESIGN EXAMPLE 7.3

EFFECT OF VARYING ARMOR UNIT SPECIFIC WEIGHT

GIVEN: ROUGH QUARRYSTONE ARMOR LAYER, $\gamma_r = 150$ PCF
REQUIRED ARMOR UNIT WEIGHT, $W = 12$ TONS

REQD: FOR THE SAME WAVE ACTION, DETERMINE THE QUARRYSTONE WEIGHT IF
a) $\gamma_r = 140$ PCF
b) $\gamma_r = 175$ PCF

SOLUTION:

OBTAIN THE WEIGHT FACTORS FROM THE UPPER CURVE IN FIGURE 7.10:

$f\ (\gamma_r = 140 \text{ PCF}) = 2.03$
$f\ (\gamma_r = 150 \text{ PCF}) = 1.49$
$f\ (\gamma_r = 175 \text{ PCF}) = 0.78$

a) $\gamma_r = 140$ PCF:

$$W_{140} = W_{150} \times \frac{f_{140}}{f_{150}} = 12 \times \frac{2.03}{1.49} = 16.3 \simeq 16 \text{ TONS}$$

FOR $\gamma_r = 140$ PCF, $W = 16$ TONS

b) $\gamma_r = 175$ PCF:

$$W_{175} = 12 \times \frac{0.78}{1.49} = 6.3 \simeq 6 \text{ TONS}$$

FOR $\gamma_r = 175$ PCF, $W = 6$ 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.

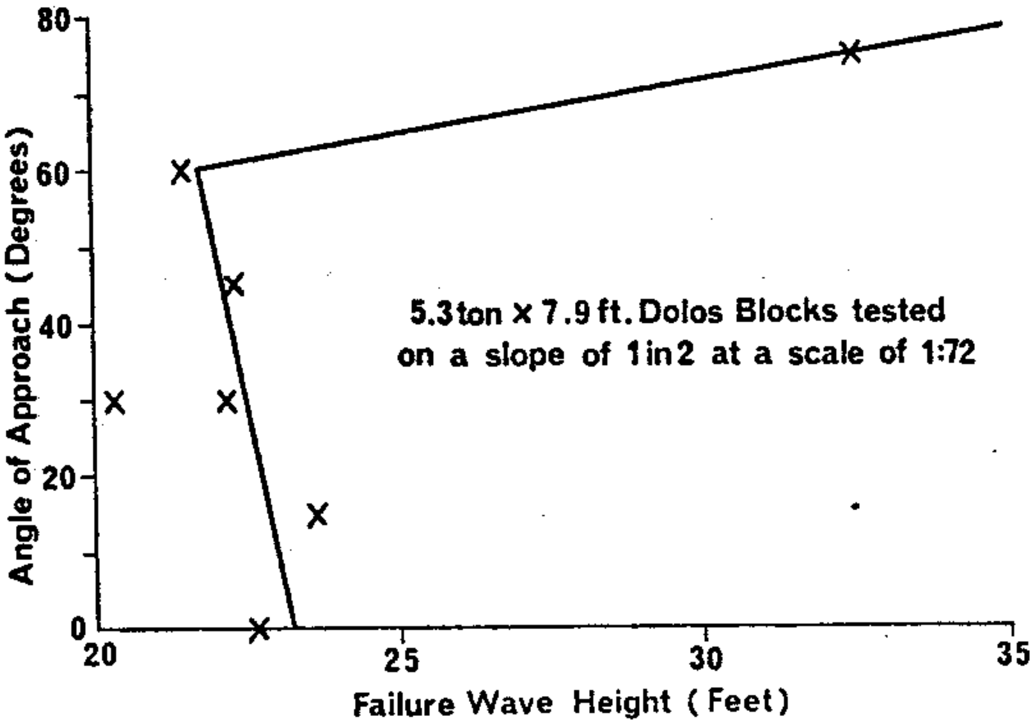


Figure 7.11 Effect of Angle of Wave Approach on Dolos Slope Stability (Whillock and Price, 1976, p. 2571)

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", ξ, was proposed to describe flow characteristics:

$$\xi = \frac{tg\theta}{\sqrt{H/L_o}} \tag{7.5}$$

in which: t = wave period, g = gravitational acceleration, θ = mound slope angle with the horizontal, H = wave height in front of structure, and L_o = 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 ξ 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 ξ 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

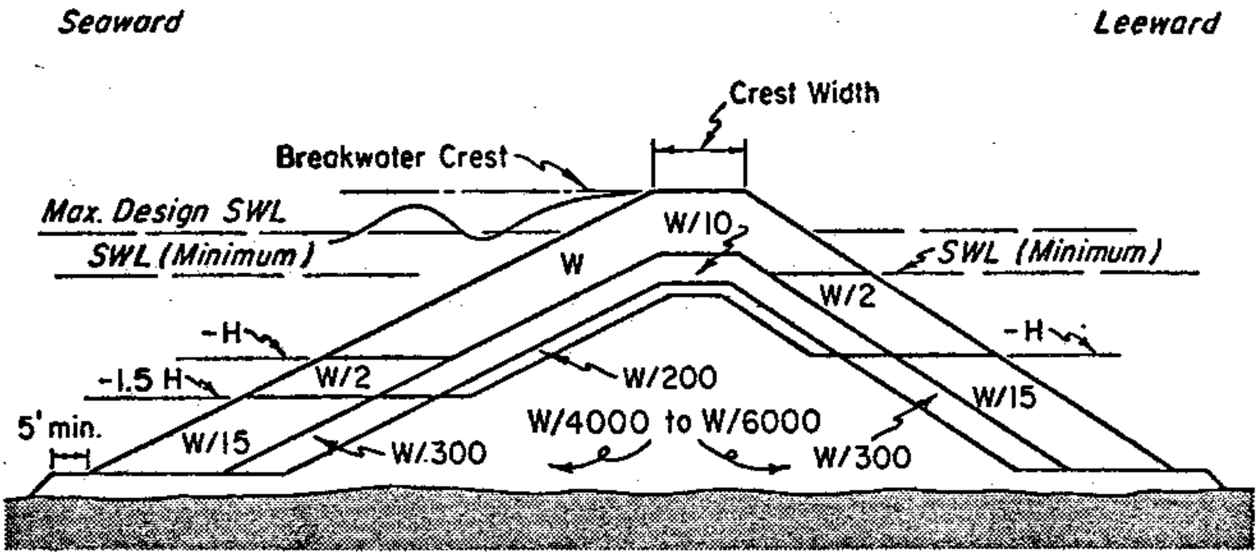
It has been demonstrated that stability formulas, although presently used exclusively in rubble mound design, are not wholly satisfactory. A proposed force balance method (Wang, 1977, after Bruun and Johannesson, 1974), while neither widely accepted nor extensively tested, seems promising as it is based 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).

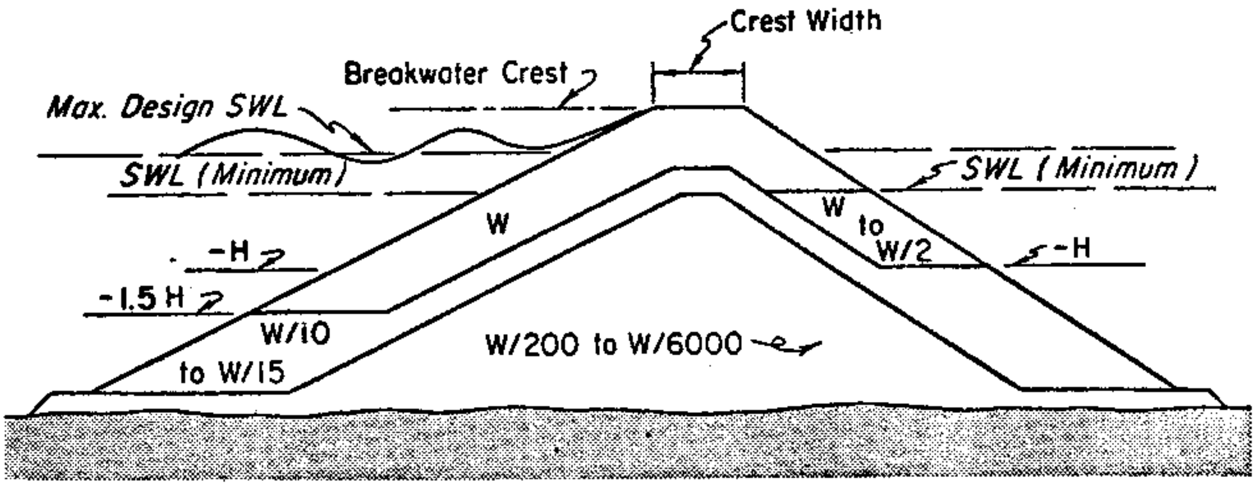
7.2 CROSS-SECTION DESIGN

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:



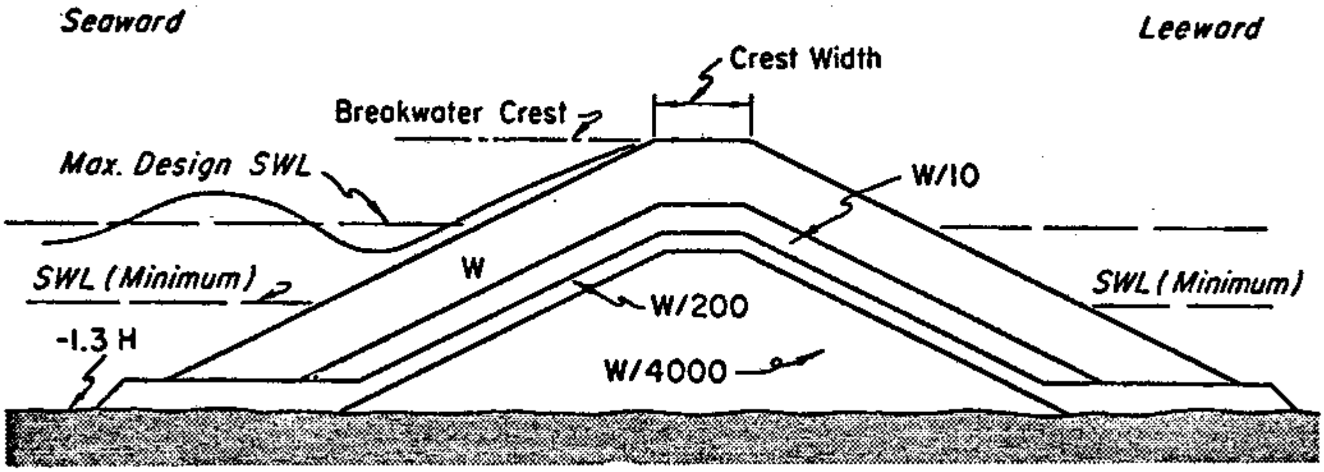
Idealized Multilayer Section

Rock Size	Layer	Rock Size Gradation (%)
W	Primary Cover Layer	125 to 75
W/2 and W/15	Secondary Cover Layer	125 to 75
W/10 and W/300	First Underlayer	130 to 70
W/200	Second Underlayer	150 to 50
W/4000–W/6000	Core and Bedding Layer	170 to 30



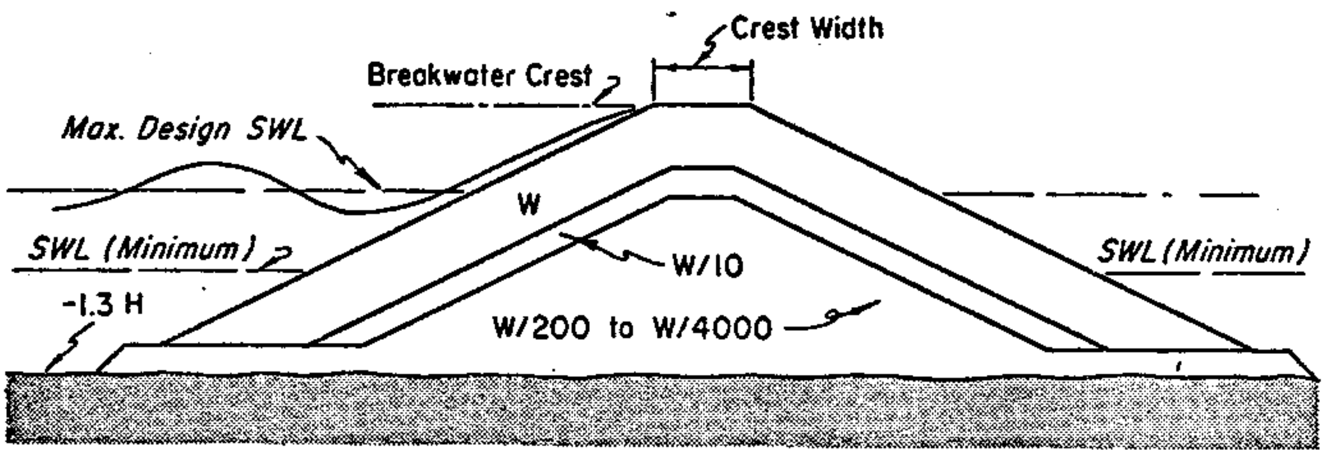
Recommended Three-layer Section

Figure 7.12 Rubble Mound Cross-Section for Nonbreaking Wave Condition with No- to Moderate Overtopping (CERC, 1977, p. 7-203)



Idealized Multilayer Section

Rock Size	Layer	Rock Size Gradation (%)
W	Primary Cover Layer	125 to 75
W/10	First Underlayer	130 to 70
W/200	Second Underlayer	150 to 50
W/4000	Core and Bedding Layer	170 to 30



Recommended Three-layer Section

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:

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

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.

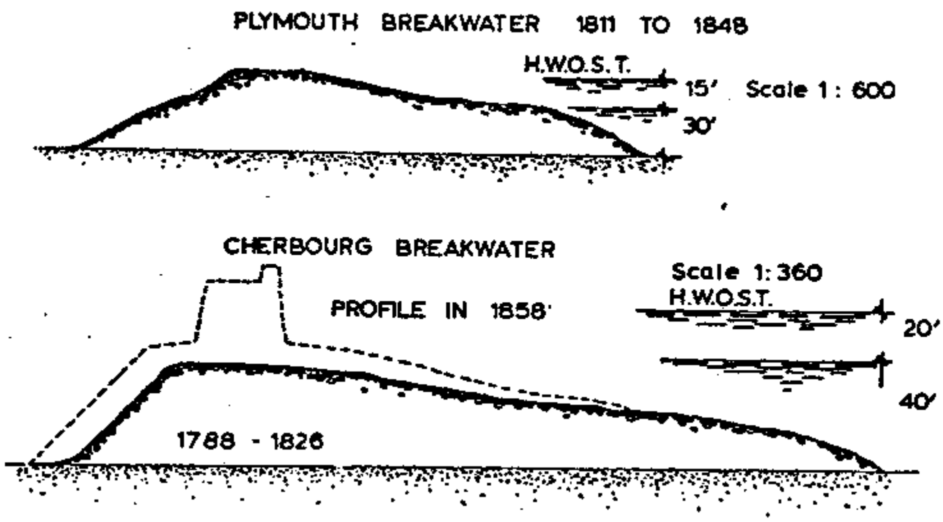


Figure 7.14 S-Shape Profile of "Mature" Breakwaters at Plymouth, England and Cherbourg, France (Bruun and Kjelstrup, 1981, p. 172)

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:

$$B = nk_{\Delta} \left(\frac{W}{\gamma_r}\right)^{1/3} \tag{7.7}$$

in which: B = crest width, n = number of stones (n=3 minimum), k_{Δ} = layer coefficient, W = cover layer armor unit weight, and γ_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)

	Water Depth at Structure $\leq 1.3H$ (Figure 6.13)		Water Depth at Structure $> 1.3H$ (Figure 6.12)	
	Overtopping	Nonovertopping	Overtopping	Nonovertopping
Seaward slope, Extension	To Bottom	To Bottom	To – H Below Minimum SWL	To – H Below Minimum SWL
Leeward slope, Extension*	To Bottom	Determined By Wave Action Within The Harbor	Minimum SWL	Determined By Wave Action Within The Harbor

* 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 armoring 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 quarrystone 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 quarrystone 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, $H = 13$ FT
$\cot\theta = 2.0$ (1 ON 2 SLOPE)
SEA WATER $\gamma_w = 64.0$ PCF
PRIMARY ARMOR UNITS - QUADRIPODS, $\gamma_c = 140$ PCF
SECONDARY ARMOR UNITS - SMOOTH QUARRYSTONE, $\gamma_Q = 165$ PCF

REQD : 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,

$$W = \frac{\gamma_r H^3}{K_D (S_r - 1)^3 \cot\theta} \qquad \text{FROM TABLE 7.1, } K_D = 8.3$$

$$= \frac{.140\,(13)^3}{8.3\left(\frac{140}{64} - 1\right)^3 2.0} = 11065^{\#} = 5.53 \text{ TONS}$$

$\therefore$ W QUADRIPODS $\simeq$ 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 W EQUIVALENT FOR QUARRYSTONE MUST BE COMPUTED.

FROM TABLE 7.1, $K_D = 2.4$

$$W_{EQUIV} = \frac{165\,(13)^3}{2.4\left(\frac{165}{64} - 1\right)^3 2.0} = 19215^{\#} = 9.61 \text{ TONS} \approx 9.5 \text{ TONS}$$

THEN, $\frac{W}{2} = \frac{9.5}{2} = 4.8$ TONS

$\frac{W}{15} = \frac{9.5}{15} = 0.63$ TONS $\approx 1270^{\#}$

QUARRYSTONE : $W/2 = 4.8$ T ; $W/15 = 1270^{\#}$

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 quarrystones (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 quarrystone 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, γ_r; N_r = the required number of armor units for a given surface area, A; k_{Δ} = layer coefficient; P = average porosity of cover layer. The magnitudes of k_{Δ} 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 γ_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,

Armor Unit	n	Placement	Layer Coefficient k_{Δ}	Porosity (P) percent
Quarrystone (smooth)	2	random	1.02	38
Quarrystone (rough)	2	random	1.15	37
Quarrystone (rough)	>3	random	1.10	40
Cube (modified)	2	random	1.10	47
Tetrapod	2	random	1.04	50
Quadripod	2	random	0.95	49
Hexapod	2	random	1.15	47
Tribar	2	random	1.02	54
Dolos	2	random	1.00	63
Tribar	1	uniform	1.13	47
Quarrystone	graded	random	——	37

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

Weight (tons)	Dimen. (ft.)	Weight (lbs.)	Dimen. (ft.)	Weight (lbs.)	Dimen. (in.)	Weight (lbs.)	Dimen. (in.)	Weight (lbs.)	Dimen. (in.)
1	2.64	100	0.97	5	4.30				
2	3.33	200	1.23	10	5.42	0.5	2.00	0.025	0.74
3	3.81	300	1.40	15	6.21				
4	4.19	400	1.54	20	6.83	1.0	2.52	0.050	0.93
5	4.52	500	1.66	25	7.36				
6	4.80	600	1.77	30	7.82	1.5	2.88	0.075	1.06
7	5.05	700	1.86	35	8.23				
8	5.28	800	1.95	40	8.60	2.0	3.17	0.100	1.17
9	5.49	900	2.02	45	8.95				
10	5.69	1000	2.10	50	9.27	2.5	3.41	0.125	1.26
11	5.88	1100	2.16	55	9.57				
12	6.05	1200	2.23	60	9.85	3.0	3.63	0.150	1.34
13	6.21	1300	2.27	65	10.12				
14	6.37	1400	2.35	70	10.37	3.5	3.82	0.175	1.41
15	6.51	1500	2.40	75	10.61				
16	6.66	1600	2.45	80	10.84	4.0	3.99	0.200	1.47
17	6.79	1700	2.50	85	11.06				
18	6.92	1800	2.55	90	11.28	4.5	4.15	0.225	1.53
19	7.05	1900	2.60	95	11.48				
20	7.17	2000	2.64	100	11.63	5.0	4.30	0.250	1.59

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 W_p, NUMBER OF LAYERS $n_p = 2$.

SECONDARY COVER LAYER OF SAME MATERIAL. BETWEEN $-H$ AND $-1.5H$, ARMOR WEIGHT $W_s = W_p/2$ BELOW $-1.5H$, REQUIRED WEIGHT $W_s = W_P/15$, AS SHOWN IN FIGURE 7.12.

REQD : 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.

$$r = n k_\Delta \left(\frac{W}{\gamma_r}\right)^{1/3}$$

HERE, $r_{SECONDARY} \geq r_{PRIMARY}$ $\therefore$ SET $r_s = r_p$

$$n_s\, k_{\Delta s} \left(\frac{W_s}{\gamma_{r_s}}\right)^{1/3} = n_p\, k_{\Delta p} \left(\frac{W_p}{\gamma_{r_p}}\right)^{1/3}$$

BECAUSE THE MATERIALS ARE THE SAME, $k_{\Delta s} = k_{\Delta p}$ AND $\gamma_{rs} = \gamma_{rp}$, AND THESE TERMS CANCEL OUT.

$$n_s \left(W_s\right)^{1/3} = n_p \left(W_p\right)^{1/3}$$

$\therefore$ AS W_s DECREASES, n_s MUST INCREASE. SOLVING FOR n_s:

$$n_s = n_p \frac{W_p^{1/3}}{W_s^{1/3}} \quad , \quad n_p = 2$$

2/2

THEN, FOR $W_s = \frac{W_P}{2}$,

$$n_s = 2\,\frac{W^{1/3}}{\left(\frac{W}{2}\right)^{1/3}} = \frac{2}{\left(\frac{1}{2}\right)^{1/3}} = 2.52 \simeq 2.5$$

BETWEEN $-H$ AND $-1.5\,H$, $n_s = 2.5$

FOR $W_s = \frac{W_P}{15}$,

$$n_s = \frac{2}{\left(\frac{1}{15}\right)^{1/3}} = 4.93 \simeq 5.0$$

BELOW $-1.5\ H$, $n_s = 5$

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 stones 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

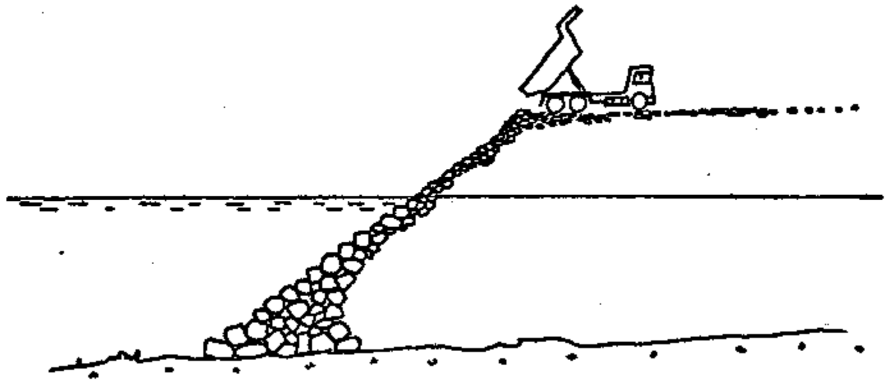
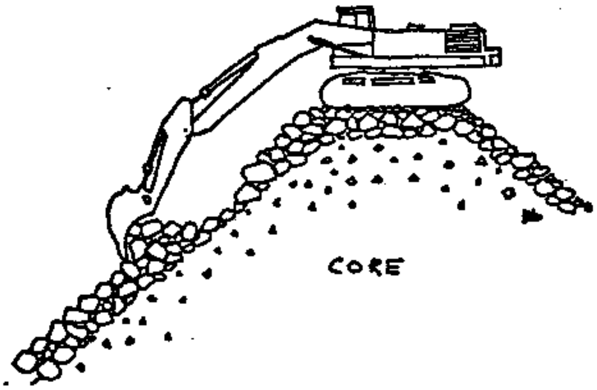
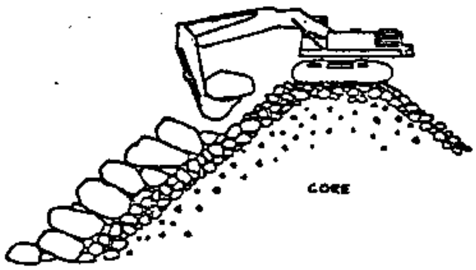


Figure 7.15 Separation of Large Rubble (Kjelstrup, 1979, p. 139)



a. Smoothing the underlayer



b. Placing the armor stone

Figure 7.16 Back-Hoe Method to Finish Rubble Mound Layers (Kjelstrup, 1979, p. 140)

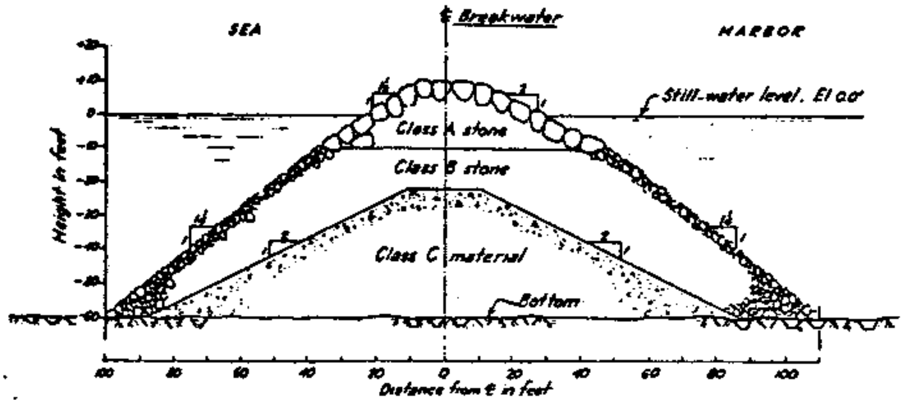


Figure 7.17 Design Cross-Section, Submerged Core Rubble Mound Breakwater (Quinn, 1972, p. 176)

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.

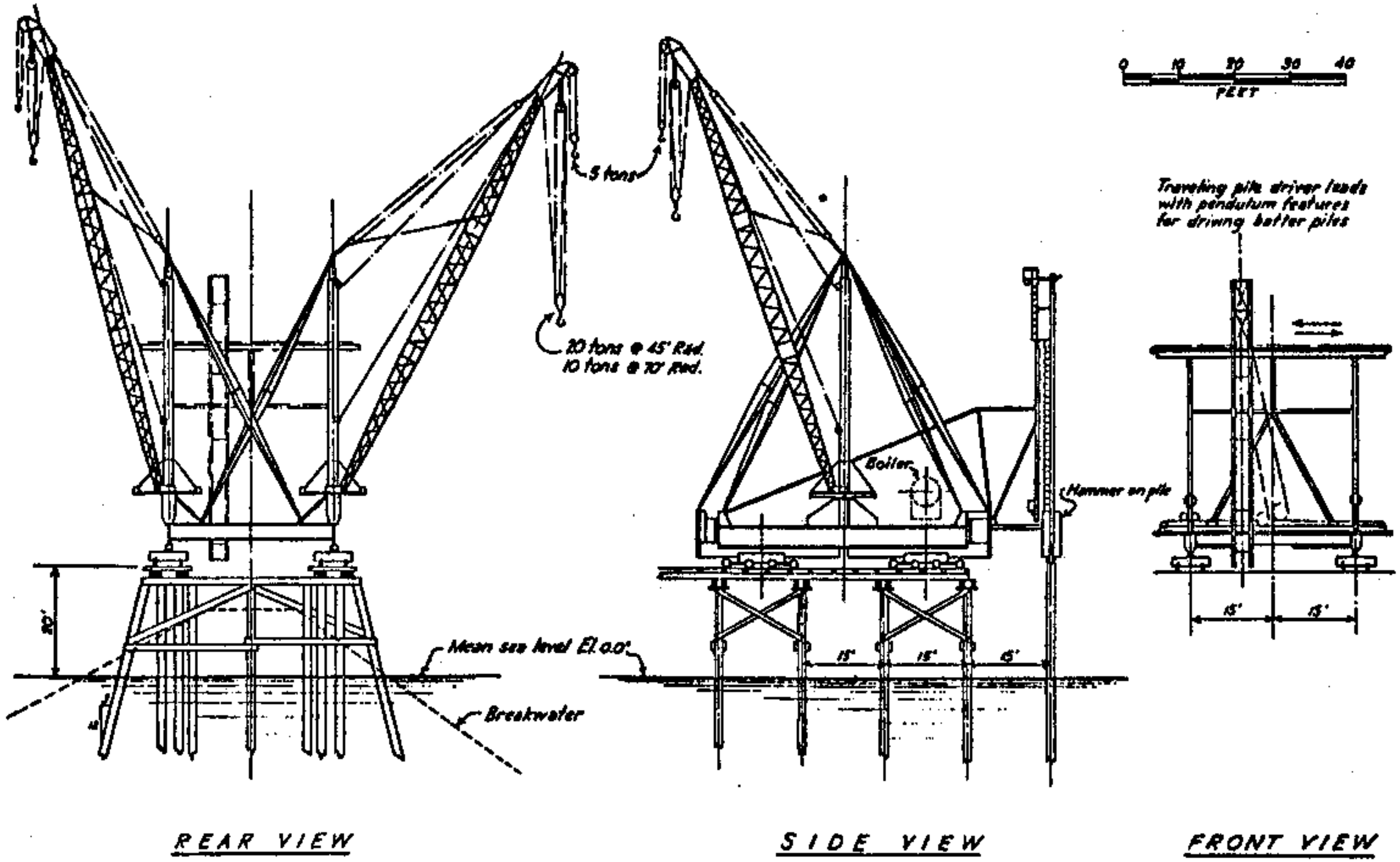


Figure 7.18 Typical Cross-Section of Trestle and Traveler used for Construction of Coco Solo Breakwater (Quinn, 1972, p. 204)

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,

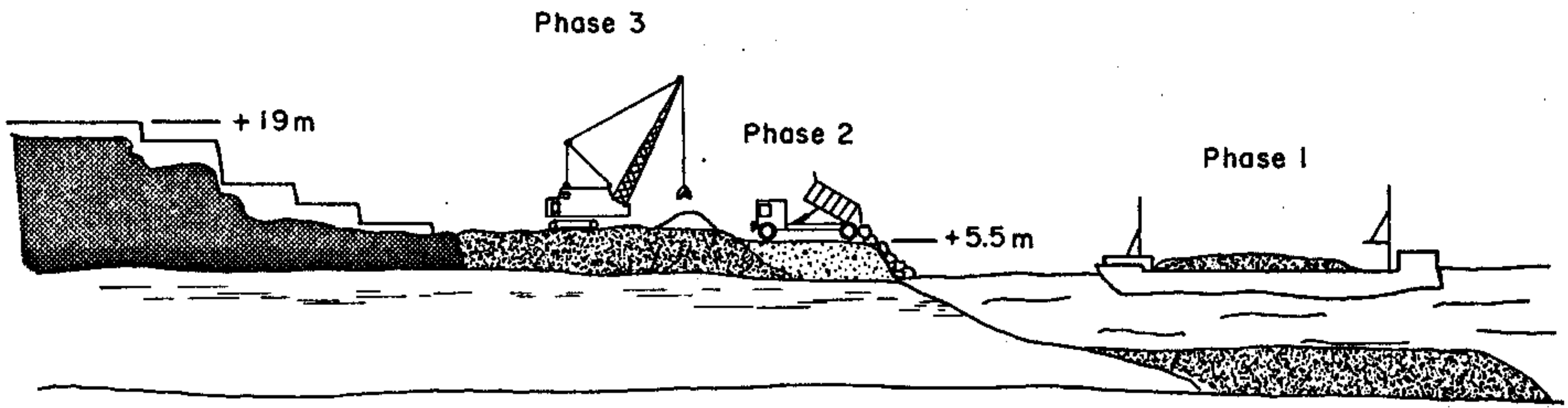


Figure 7.19 Staged Construction, Sines Breakwater, Portugal (Zwamborn, 1979, p. 428)

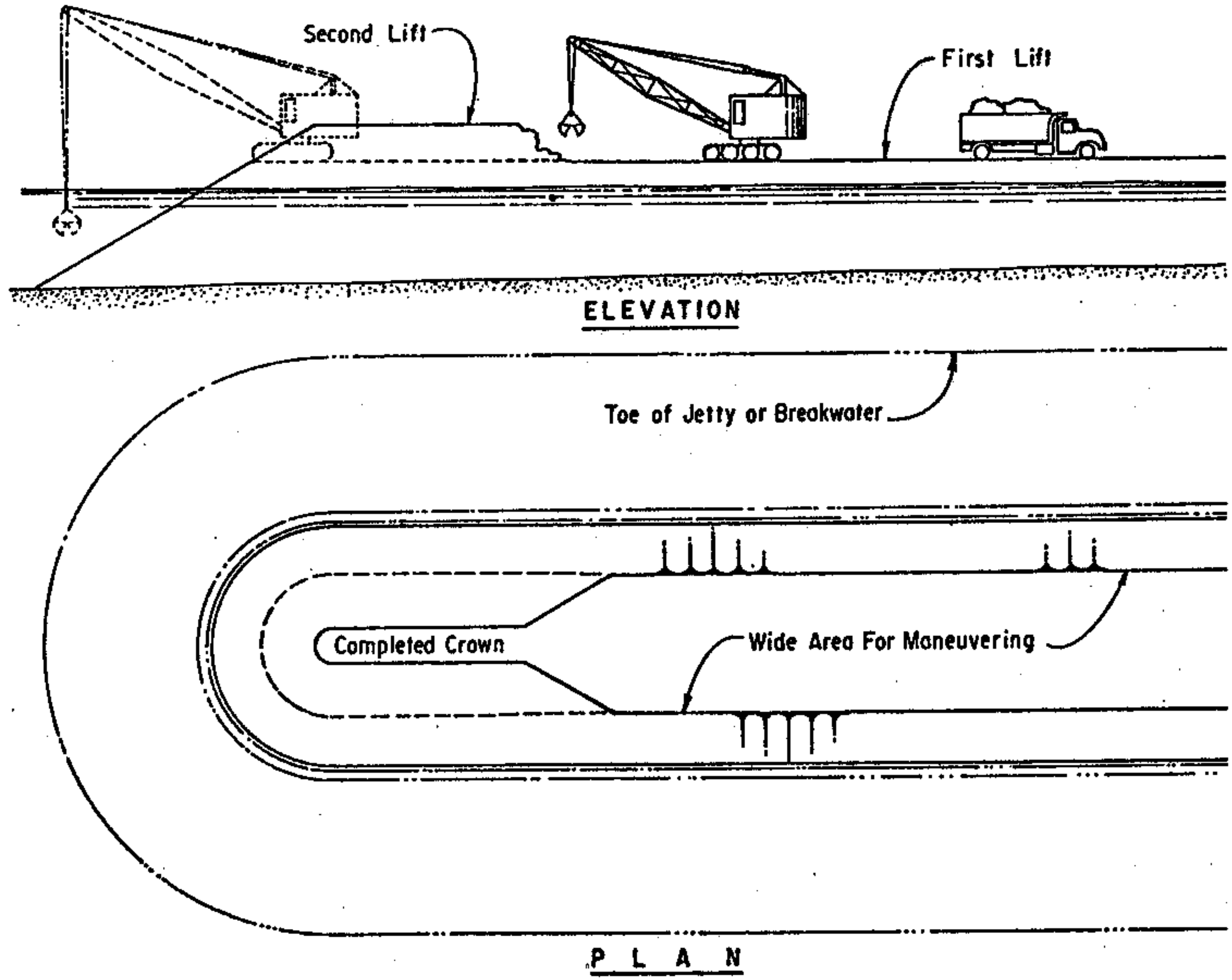


Figure 7.20 Two-Stage Construction of a Shore-Connected Rubble Mound Structure (Dunham and Finn, 1974, p. 57)

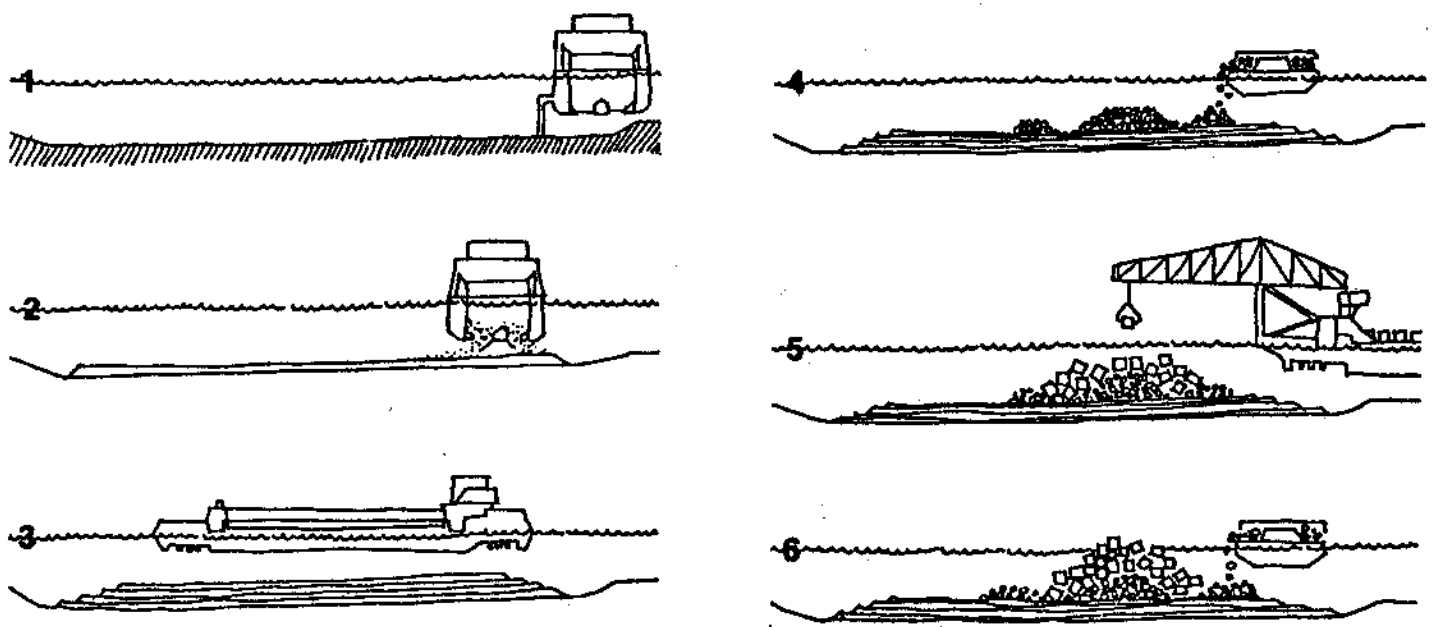


Figure 7.21 Lift Construction of Breakwater at Rotterdam, Europort. See Text for Descriptions (TAMU, 1971, p. 71)

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,

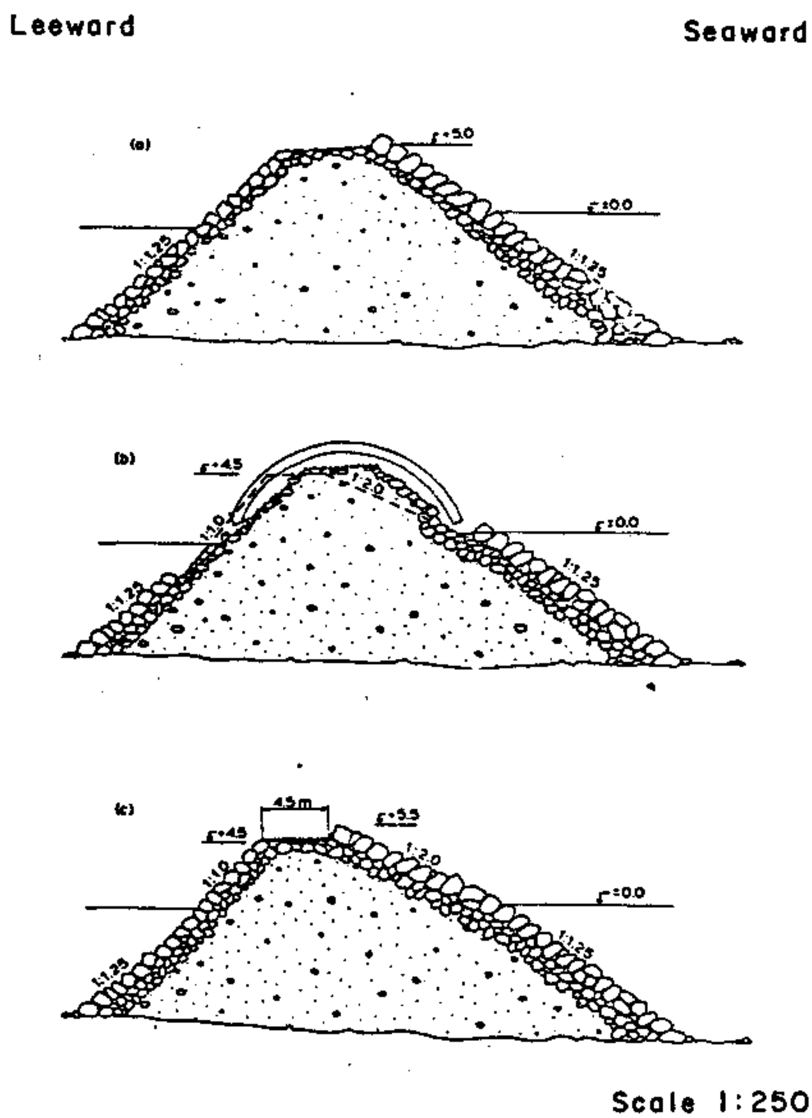


Figure 7.22 Breakwater at Gryllefjord, Troms after a) Initial construction, b) One or two years, c) Four years. See Text for Descriptions (Bruun and Kjelstrup, 1981, p. 189)

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

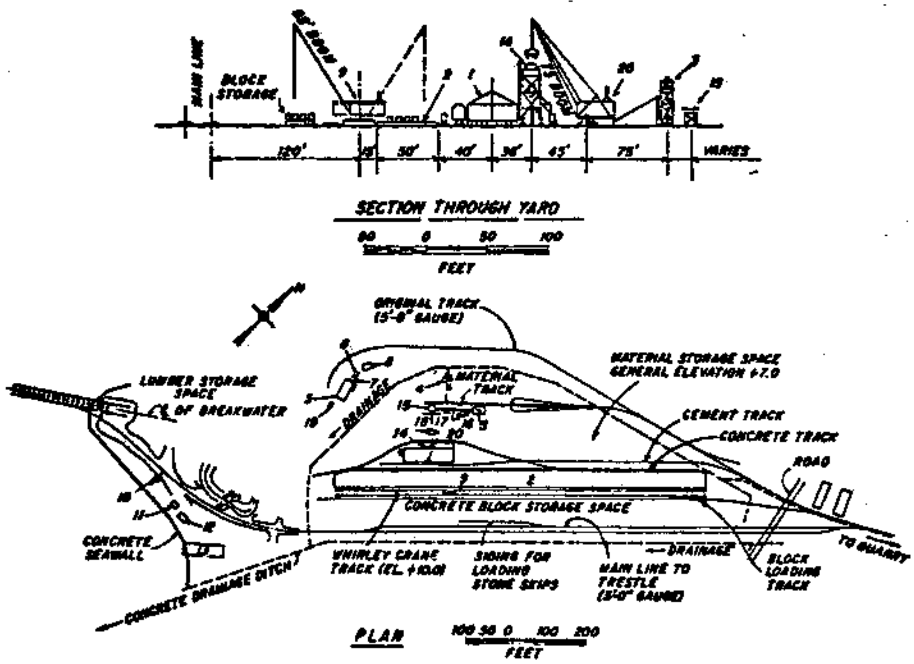


Figure 7.23 Casting Yard for Coco Solo Breakwater Concrete Armor Units. 1) Cement storage shed, 2) Block casting platform, 3) Aggregate hopper, 4) Settling tank, 5) Storehouse, 6) Saw shed, 7) Carpentry shed, 8) Concrete building, 9) Whirley crane, 10) Water tank, 11) Oil storage, 12) Concrete building, 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

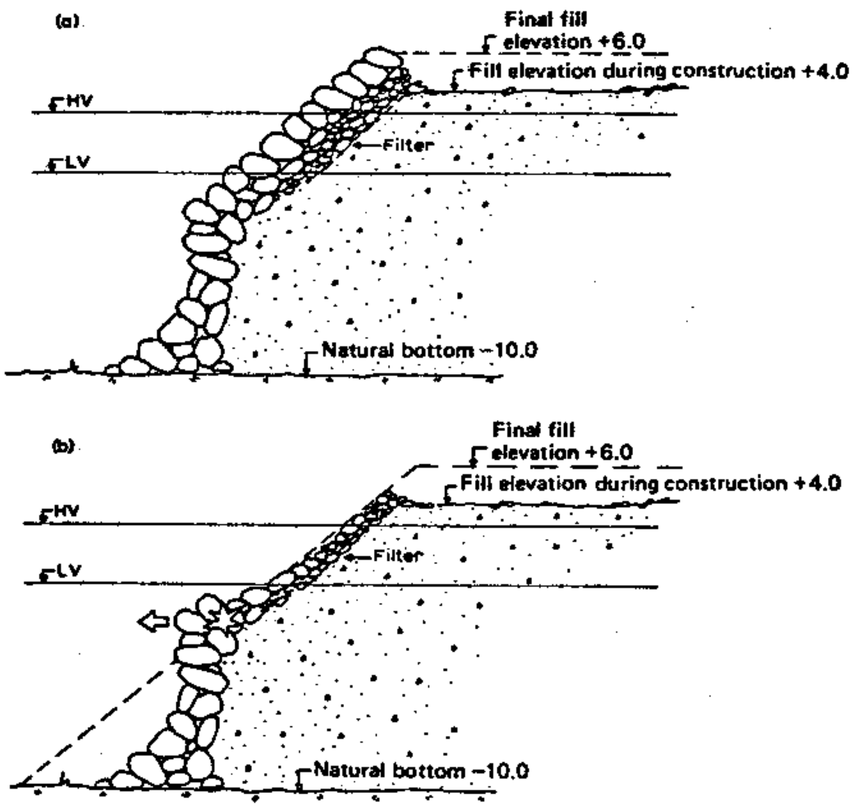


Figure 7.24 Down-Blasting of Steep, Unstable Rubble Slope Below Water Level (Bruun and Kjelstrup, 1981, p. 190)

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.

CHAPTER 8

SUMMARY AND CONCLUSIONS

The shoreline is the site of complex interactions between water, wind and land. Coastal erosion occurs when the natural forces remove more material than they accrete. Coastal protection methods are implemented to control or stop unacceptable erosion. Process alteration structures, breakwaters, jetties and groins, are among the means of engineered protection.

A basic appreciation of the coastal environment and processes is a prerequisite to rational design in the coastal zone. Mechanisms of littoral transport and inlet stability are vital aspects of coastal dynamics. Site data are used to quantify the net longshore transport rate and sediment budget, and the sedimentation and hydraulic characteristics of affected inlets. A detailed site analysis encourages engineering design which conforms to, rather than opposes, the natural forces.

The functional design of breakwaters, jetties and groins is detailed in Chapter 3. Each structure has different characteristics of orientation and geometry that affect the precise manner in which it accretes littoral drift and attenuates wave action. These elements must be carefully planned for each project to provide the necessary protection.

Negative effects of construction, such as downdrift erosion, must be anticipated as well during the analysis and design cycle. Inadequate and improperly applied shore protection represents a waste of time, effort and capital, and may result in the loss of unrecoverable coastal property. If it appears that the proposed structure will cause significant damage as it performs its intended function, the design is unacceptable. Redefinition of the design objectives, modification of the structure configuration, incorporation of non-structural protection methods, or abandonment of the project are among the available options in reevaluation.

Breakwaters, jetties and groins each have a unique shore protection purpose, but all act in some respect to bar drift and attenuate wave forces. Because they have this general function in common, they share structural configurations. Mounds and walls are the two conventional types. Mounds are gravity structures composed of layers of discrete elements, usually rock. They effectively attenuate waves through runup and dissipation within the intertices of their rough surfaces. Walls reflect wave energy. Common wall construction materials are steel, timber and concrete. A third category, low cost shore protection, includes small-scale, low cost devices typically of simple and innovative design. It is anticipated that many promising shore protection methods will result from current studies in this field.

The structural type selected depends largely on the scale of the project. The most common configuration of process alteration structures is the mound; "rubble mound" is almost synonomous with "breakwaters, jetties and groins." For this reason, the design of rubble mounds is treated in special detail. Mounds are widely adaptable to most water depths and foundation conditions. A prime advantage is that structural damage is progressive, rather than sudden and catastrophic. The elements of the structure can settle and readjust under wave action a great deal before the structural or functional integrity is threatened.

The major environmental loading parameter in rubble mound design is wave loading. It is extremely important to characterize accurately the wave type, and select and quantify the design wave height. A thorough analysis of wave data and diffraction and refraction effects is warranted. The forces of ice, earthquakes and tsunamis can be significant on a site-specific basis. Although standard design procedures do not incorporate their magnitudes, design modifications can be made when necessary based on experience and judgment. The economics of small-scale protection generally preclude costly modifications.

Foundation conditions have a major influence on structural stability. Excessive settlement and insufficient soil bearing capacity are geotechnical problems which can induce failure of the overlying rubble mound. Similarly, toe scour can remove foundation support, dislodge the stones, and allow crevices to open in the mound. In this state, the underlayers and core are exposed to direct wave attack and can easily erode away. Proper design and installation of a foundation blanket is an essential element of the foundation scheme. Filter design specifications are detailed in Chapter 5. The overall foundation system should be carefully devised based on thorough geotechnical analyses.

The use of rock as a coastal engineering construction material has not been widely addressed in the literature. The weatherability of rock in the coastal zone is an important concern in the design of mounds armored with rock. The rock must be durable for the life of the structure, and sufficient quantities of the rock must be available at a reasonable cost. The marine environment is extremely aggressive and can degrade materials which seem sound on initial examination. Laboratory and field investigations are necessary for a complete assessment of rock quality. Exploratory methods are suggested in Chapter 6.

Characteristics of concrete armor units are discussed as well. These elements have superior hydraulic stability; smaller concrete shapes can provide the same wave protection as larger rock armor units. Their use is predominantly on larger rubble mound installations exposed to more severe wave attack, where high armor unit weights are required by design.

Actual mound design commences with definition of the structure geometry. Because mound stability depends on the weight of the armor layer, computation of cover layer armor unit weight constitutes a major part of the design effort. Empirical methods, as Hudson's formula (Equation 7.2), give a satisfactory representation of cover layer stability. Quick graphical solutions are available in Chapter 7. Planners must recognize the limitations of these approaches and be careful not to extrapolate the results beyond their intended context. The effects of variables not integrated into empirical procedures should be studied through hydraulic model investigations whenever possible.

There are as many possible mound cross-sections as there are mound structures. The recommendations presented in Chapter 7 are intended as guidelines to be adapted to the particular site conditions and shore protection needs.

Construction equipment and methods can be the limiting factor in rubble mound design. Practical limitations and potential difficulties will be apparent only if construction planning occurs concurrently with the analysis and design phases. Attention to the proper sequencing of operations and to construction supervision assures the good quality of the final product.

Of necessity the discussions presented here have classed various design considerations and procedures in discrete groups. It is emphasized that evaluation of environmental and geotechnical conditions, assessment of materials, computational design, and construction planning are inherently interrelated and must be treated simultaneously. When one of these factors is altered it is likely that others are affected as well. The analysis-design cycle, with its many facets, should be reevaluated for each set of design parameters. The optimum design affords the necessary protection at the lowest cost, with the minimum of negative environmental effects.

The decision to erect shore protection structures cannot be taken lightly. Thorough analyses provide the necessary data base. The anticipated effects of a proposed structure on adjacent shorelines as well as on the property of interest must be objectively evaluated. If it is decided to build, design must be undertaken and executed with the same seriousness of purpose. The use of guidelines presented in this study and consultation with knowledgeable professionals can help to ensure successful protection against coastal erosion.

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Appendix A

Sources of Information on Low Cost Shore Protection

The review of low cost shore protection given in Chapter 4 can be supplemented with information available from the U.S. Corps of Engineers. The following is cited from Rogers, Golden and Halpern (1981), an introductory brochure on the subject. To obtain the reports mentioned, write to:

John G. Housley
Section 54 Program
U.S. Army Corps of Engineers
USACE (DAEN-CWP-F)
Washington, DC 20314

Where to Go From Here

Three in-depth reports have been prepared as supplements to this introductory brochure. **Low Cost Shore Protection: A Property Owner's Guide** gives detailed information about the subjects covered briefly in this brochure, as well as a list of other helpful publications (many of them free) and the addresses of government agencies that have jurisdiction or expertise in waterfront areas. The other two reports, **Low Cost Shore Protection: A Guide for Local Government Officials** and **Low Cost Shore Protection: A Guide for Engineers and Contractors**, include information pertinent to these groups, as well as lists of information sources and government agencies.

These reports are the latest products of the long-term commitment to coastal planning and engineering of the U.S. Army Corps of Engineers. In the Shoreline Erosion Control Demonstration Act of 1974, Congress authorized the Corps of Engineers to develop and demonstrate low cost methods of shoreline erosion protection in the sheltered and inland waters of the United States and disseminate the results of the demonstration program. The Corps of Engineers has produced other publications, many of which are available through the U.S. Government Printing Office, Washington, DC 20402. In addition, the Corps conducts research at the Coastal Engineering Research Center in Fort Belvoir, Virginia.

Information on local situations may be obtained from the district offices of the Corps of Engineers and from state and local agencies responsible for water, natural resources, or coastal management. Federal and state offices of the Soil Conservation Service and Fish & Wildlife Service will be able to help you locate professional advice about the ecology of your area and find ways to evaluate and minimize the environmental impact of any erosion control measure you decide to use. The National Ocean Survey can provide hydrographic charts and tide tables for all U.S. coastal areas; and lake level information for the Great Lakes is available from the Detroit District of the Corps of Engineers.

Your state's board of higher education can help you locate agricultural and marine extension services, another source of information about local conditions and ecology. These services are usually associated with "land-grant" or "sea-grant" units of state or private colleges and universities.

Local businesses and associations concerned with waterfront uses may be able to refer you to competent professionals familiar with your area and its special characteristics.

Appendix B

Filter Fabric Information

Manufacturer or Distributor	Product Name	Comment*	Cost/sq ft (1978 Estimate)
1. Celanese Fibers	Mirafi 140		$0.07
2. Dupont	Typar 3401	Pervious	0.08
	Typar T063	Impervious	0.155
3. Monsanto Textiles Co.	Bidim C-22		0.07
	Bidim C-34		0.124
4. Phillips Fibers Corp.	Supac 4P		
	Supac 5P	140 Sieve EOS	0.067
	Supac (New)	80 Sieve EOS	
5. Terrafix Erosion Control Products, Inc.	Terrafix 300N		0.04
	Terrafix 500N		0.55
	Terrafix 750B		0.37
	Terrafix 370RS		0.30

* Most nonwoven manufacturers prefer performance flow tests over the Equivalent Opening Size (EOS) sieve tests used for comparison with woven fabrics

Table B.1 Material Costs of Selected Nonwoven Filter Fabric (Keown and Dardeau, 1980, p. 51)

Manufacturer or Distributor	Product Name	EOS* Sieve No.	Cost/sq ft (1978 Estimate) Small Quantities (10,000 sq ft)	Large Quantities (>1 Million sq ft)
1. Advanced Construction Specialties	(Laurel Erosion Control Cloth I)	100	$0.15	$0.102
	(Laurel Erosion Control Cloth II)	30-50	0.22	0.18
2. Carthage Mills	Filter-X	100	0.255	0.20
	Poly-Filter X	70	0.17	0.12
	Poly-Filter GB	40	0.245	0.185
3. Koch Brothers, Inc.	Zenith		0.139**	
4. Nicolon Corp.	Nicolon 70	70	0.12**	
	Nicolon 40	40	0.12**	
5. J. P. Stevens, (Menardi-Southern)	Monofilter 40	40	0.10**	
	Monofilter 70	70	0.10**	
	Monofilter 100	100	0.10**	

* Equivalent Opening Size.
** Manufacturer does not distinguish between cost categories for small and large quantities.

Table B.2 Material Costs of Selected Woven Filter Fabric (Keown and Dardeau, 1980, p. 50)

Fabric Manufacturers

(after Koerner and Welsh, 1980, pp. 252-254
and Keown and Dardeau, 1980, pp. A1-A2)

Advance Construction Specialties Company
P. O. Box 17212
Memphis, TN 38117

American Enka Company
Enka, NC 28728

Amoco Fibers Company
Patchague Plymouth Division
550 Interstate North
Atlanta, GA 30339
(404) 691-4081

Bay Mills Midland, Ltd.
Midland, Ontario L4R 4G1
Canada
(705) 526-7867

Bradley Materials
P. O. Box 254
Valparaiso, FL 32580

Carthage Mills
Erosion Control Division
124 W. 66th Street
Cincinnati, OH 45216
(513) 242-2740

Celanese Fibers Marketing Company
Box 1414
Charlotte, NC 28232
(704) 554-2000

Crown Zellerbach
Nonwoven Fabrics Division
P. O. Box 877
Camas, WA 98607
(206) 834-5954

DuPont de Nemours and Company
Textile Research Laboratory
1007 Market Street
Wilmington, DE 19898
(302) 774-0650

Eastman Chemical Products, Inc.
Kingsport, TN 37662
(212) 262-7187

GAF Corporation
Glenville Station
Greenwich, CT 06830
(203) 324-5418

ICI Fibres: "Terram"
Pontypool, Gwent, NP4 8YD
Great Britain
(04955) 58150

J. P. Stevens and Co., Inc.
Stevens Tower
1185 Avenue of the Americas
New York, NY 10036

Kenross-Naue, Inc.
131 Golf Terrace
Daphne, AL 36526

Koch Brothers, Inc.
35 Osage Avenue
Kansas City, KS 66105

Menardi-Southern Corp.
3908 Colgate
Houston, TX 77087
(713) 643-6513

Monsanto Textiles Company
800 N. Lindbergh Blvd.
St. Louis, MO 63166
(314) 694-7179

Nicolon Corporation
4229 Jeffery Drive
Baton Rouge, LA 70816
(504) 292-3010

Owens-Corning Fiberglas Corp.
Technical Center
P. O. Box 415
Granville, OH 43023
(614) 587-0610

Phillips Fibers Corp.
P. O. Box 66
Greenville, SC 29602
(803) 242-6600

PPG Industries, Inc.
One Gateway Center
Pittsburgh, PA 15222

Staff Industries, Inc.
78 Dryden Road
P. O. Box 797
Upper Montclair, NJ 07043
(201) 744-5367

Tex-el, Inc.
485, Des Erables
St. Elzear, Beauce Nord
Quebec GOS 2J0
Canada

Wellington Sears Co.
Marketing Subsidiary of West Point Pepperell, Inc.
111 W. 40th Street
New York, NY 10018
(212) 354-9150