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 4.5 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)
Concrete, rock, or asphalt cell cap may be used to cover sand or rock filled cells.

Note:
Dimensions and details to be determined by particular site conditions.

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)
Figure 4.10 Cellular Steel Sheet Pile Breakwater at Marsa al 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).

**Concrete Piles.** 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

![Diagram of timber groin with annotations]

b. Timber Breakwater

![Diagram of timber breakwater with annotations]

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)

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 Lasnik, 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

<table>
<thead>
<tr>
<th>Material</th>
<th>Erosion Control Structure</th>
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<th>Groin</th>
<th>Revetment</th>
<th>Bulkhead and Seawall</th>
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*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 sheathing. 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 Demonstration 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
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
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, \( H_b \):

\[
X_p = (4.0-9.25m) H_b
\]  

(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 \( d_b \) 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:

\[
d_b(max) = \alpha H_b
\]  

(5.2)

\[
d_b(min) = \beta H_b
\]  

(5.3)

Figure 5.5 is used to evaluate 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
(GERC, 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, \( m = 0.02 \)
- Design wave period, \( T = 8 \) sec

**READ:**
- a) Maximum breaker height, \( H_b \)
- b) Limiting breaker depths, \( d_b (\text{min}) \) and \( d_b (\text{max}) \)

**SOLUTION:**

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

Enter Figure 5.5 to the curve for \( m = 0.02 \),

Read \( \frac{H_b}{d_s} = 0.93 \)

\[
H_b = 0.93 \times d_s = (0.93)(10) \quad \therefore H_b = 9.3 \text{ ft}
\]

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 \( \beta \) curve for \( m = 0.02 \),

Read \( \beta = 1.15 \)

\( \alpha = 1.52 \)
THEN,

\[ d_b \ (\text{MIN}) = \beta H_b = (1.15)(9.3) = 10.7 \ \text{FT} = d_b \ (\text{MIN}) \]

\[ d_b \ (\text{MAX}) = \alpha H_b = (1.52)(9.3) = 14.1 \ \text{FT} = d_b \ (\text{MAX}) \]

THE CALCULATED DESIGN WAVE WILL INITIATE BREAKING IN DEPTHS BETWEEN 10.7 AND 14.1 FT.
Figure 5.5 $\alpha$ and $\beta$ 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 $d_b (\text{min})$ (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 $a$ curve, yields
Figure 5.6  Breaker Height Index versus $\frac{H_b}{gT^2}$ (CERC, 1977, p. 7-11)
DESIGN EXAMPLE 5.2

DETERMINATION OF DEEPWATER WAVE HEIGHT

GIVEN: \[ H_b = 9.3 \text{ FT} \]
\[ T = 8 \text{ SEC} \]
\[ m = 0.02 \] (FROM DESIGN EXAMPLE 5.1)
REFRACTION COEFFICIENT, \( K_R = 0.90 \) FOR A SPECIFIED
DEEPWATER DIRECTION OF WAVE APPROACH (SEE
HUBBELL AND KULHAVY, 1979b)

REQUIRED: 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 \( H_0 = \frac{H_b}{1.15} = 9.3 = 8.1 \text{ FT} \)

\( H_0' \) IS THE UNREFRACTED DEEPWATER WAVE HEIGHT.

\( H_0 = \frac{H_0'}{K_R} = \frac{8.1}{0.9} = H_0 = 9.0 \text{ FT} \)

\( H_b \) 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