# 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

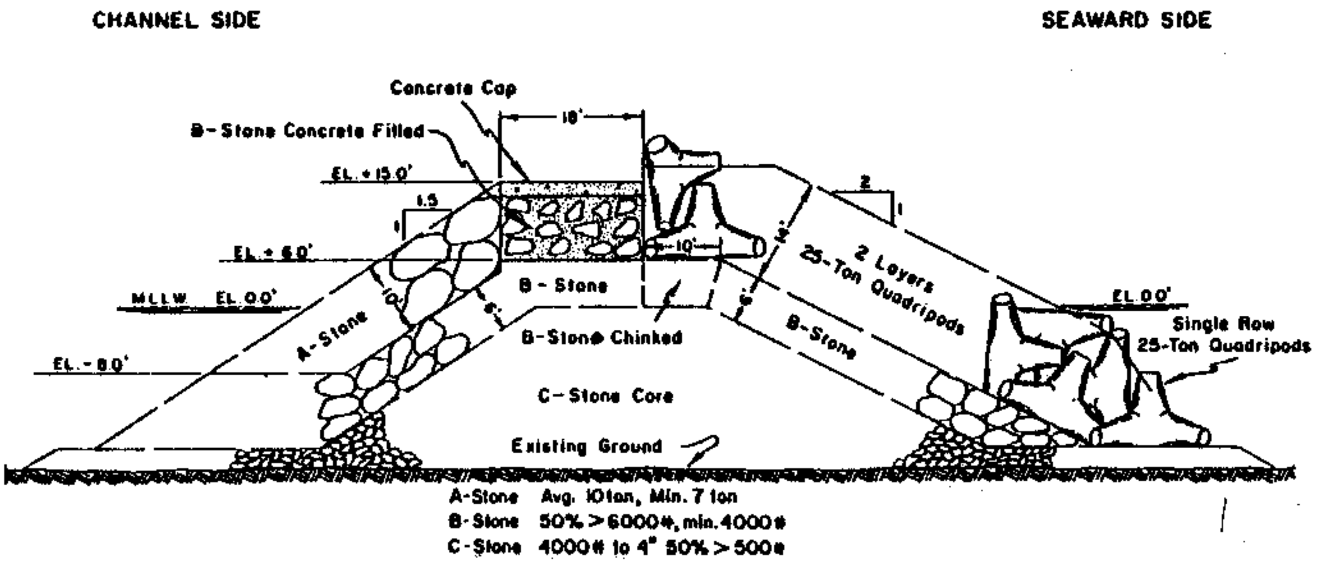


Figure 4.1 Rubble Mound Jetty (CERC, 1977, p. 6-85)

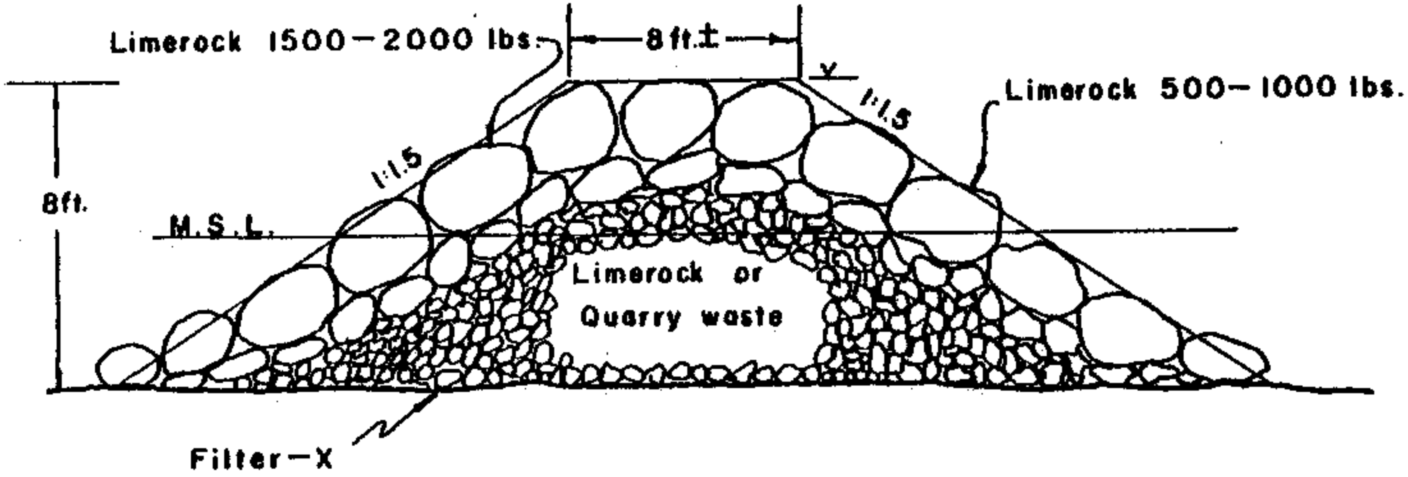


Figure 4.2 Rubble (Limestone) Mound Groin (Bruun and Manohar, 1963, p. 29)

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).

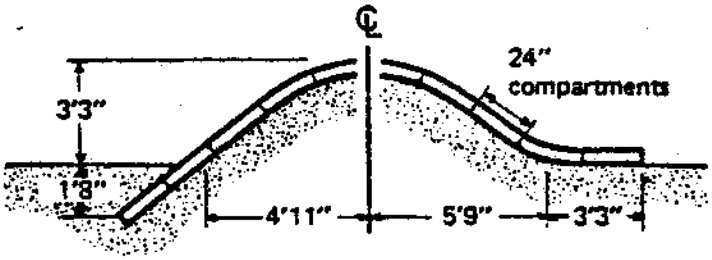


Figure 4.3 Revet Mattress Groin (Maccaferri Revet Mattress Catalog, undated, p. 11)

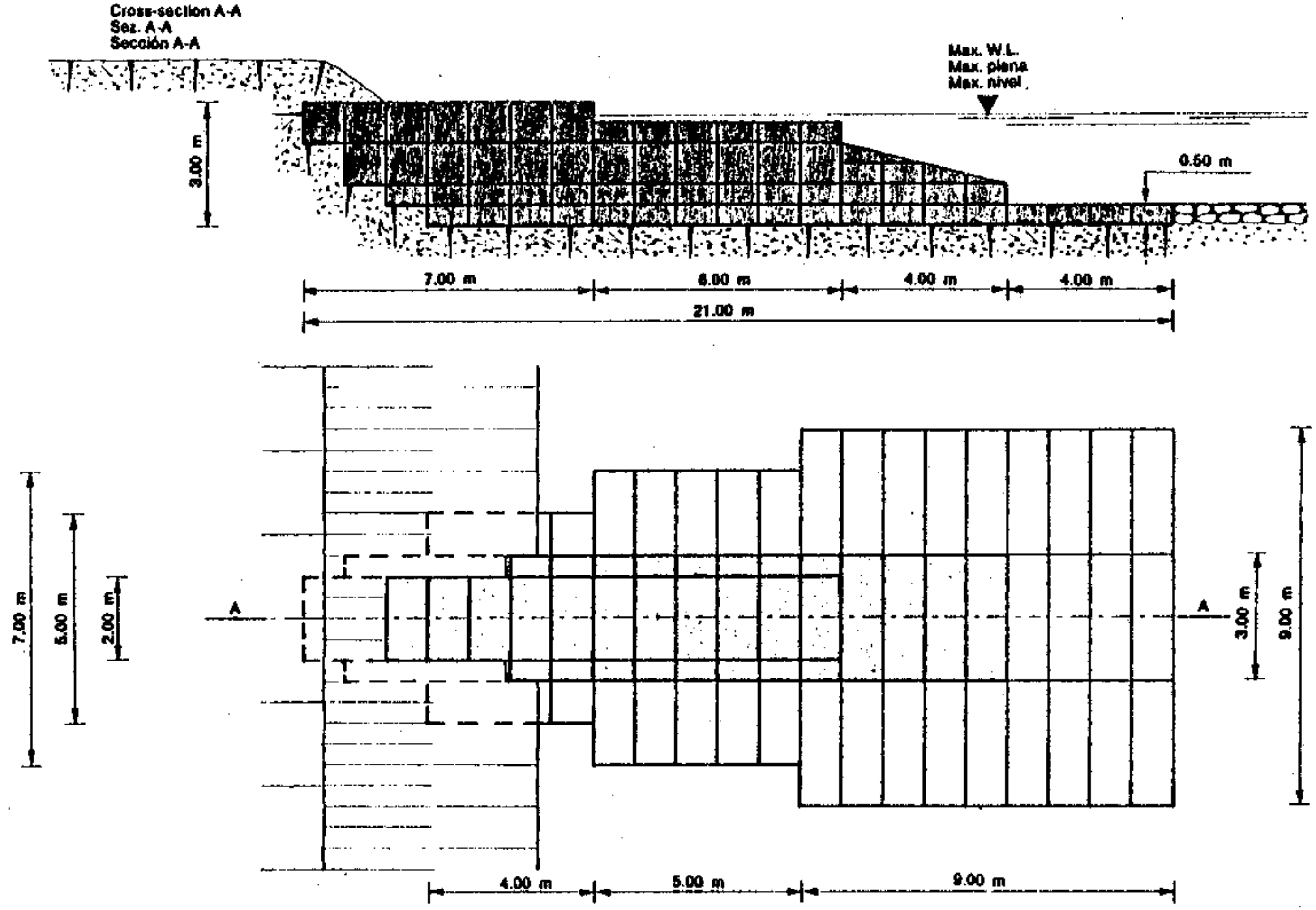


Figure 4.4 Stepped Mound Gabion Groin (Maccaferri Gabions Inc., 1979, p. 40)

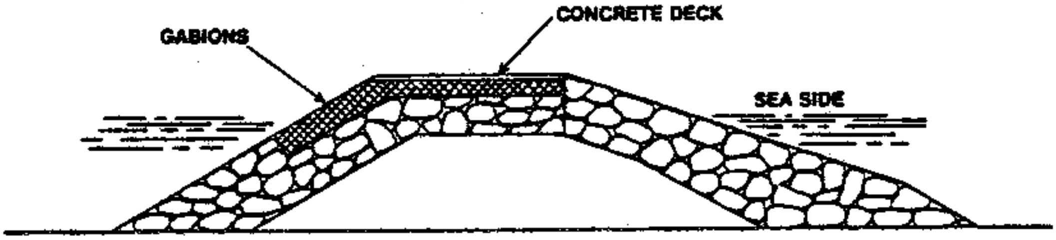


Figure 4.5 Gabion Reinforcement on Shoulder of Rubble Mound Breakwater (Bekaert Gabions, 1977, p. 54)

## 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.

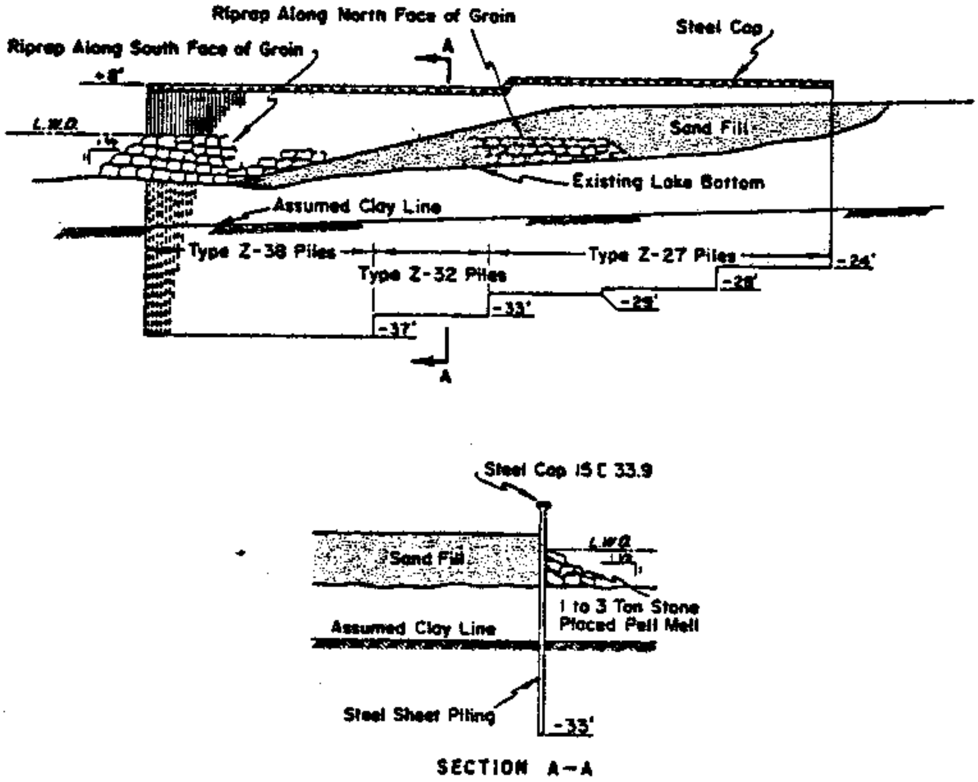


Figure 4.6 Cantilever Steel Sheet Pile Groin (CERC, 1977, p. 6-79)

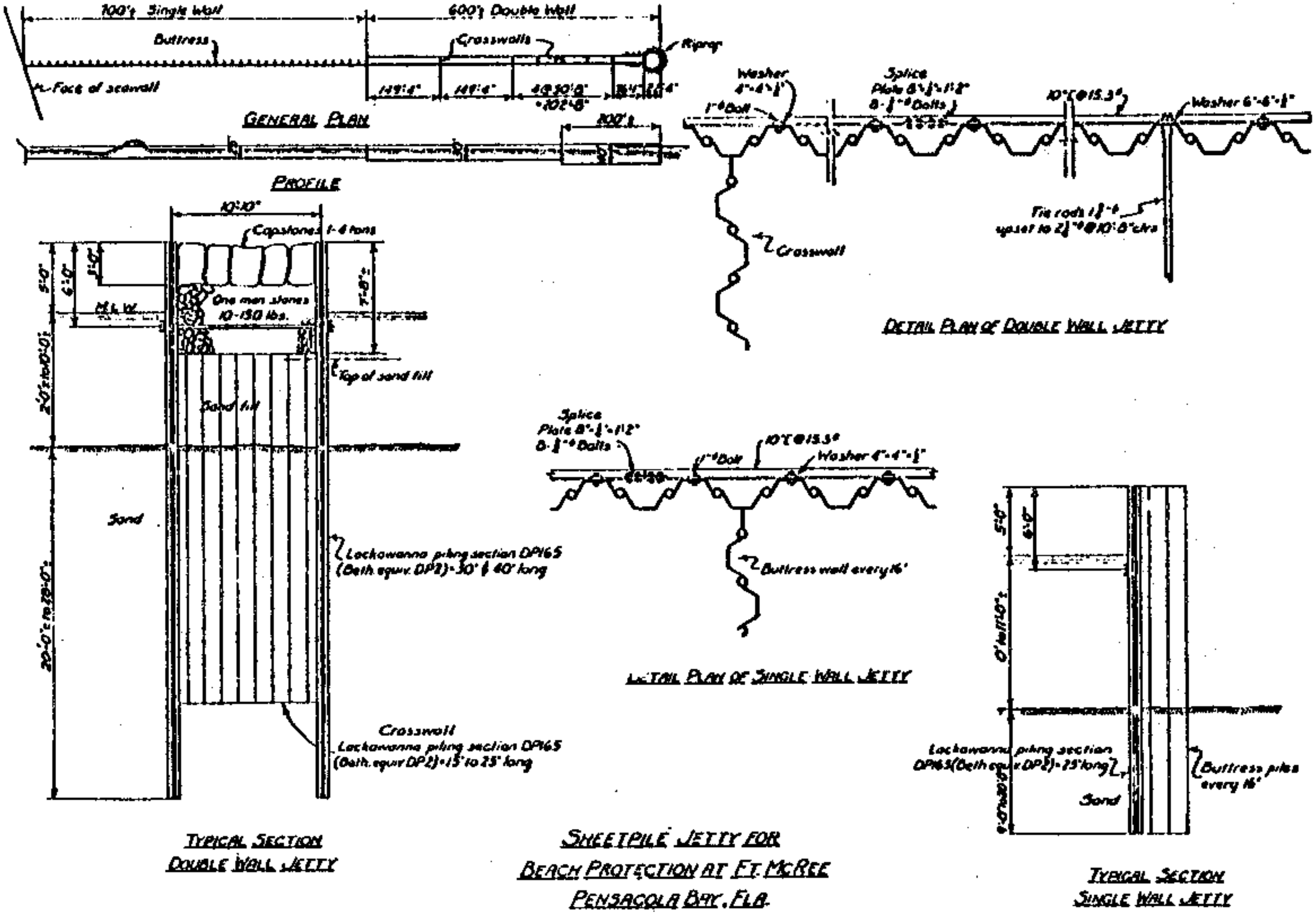


Figure 4.7 Buttressed Single Wall and Double Wall Steel Sheet Pile Jetty (USCOE, 1963, pl. 31)

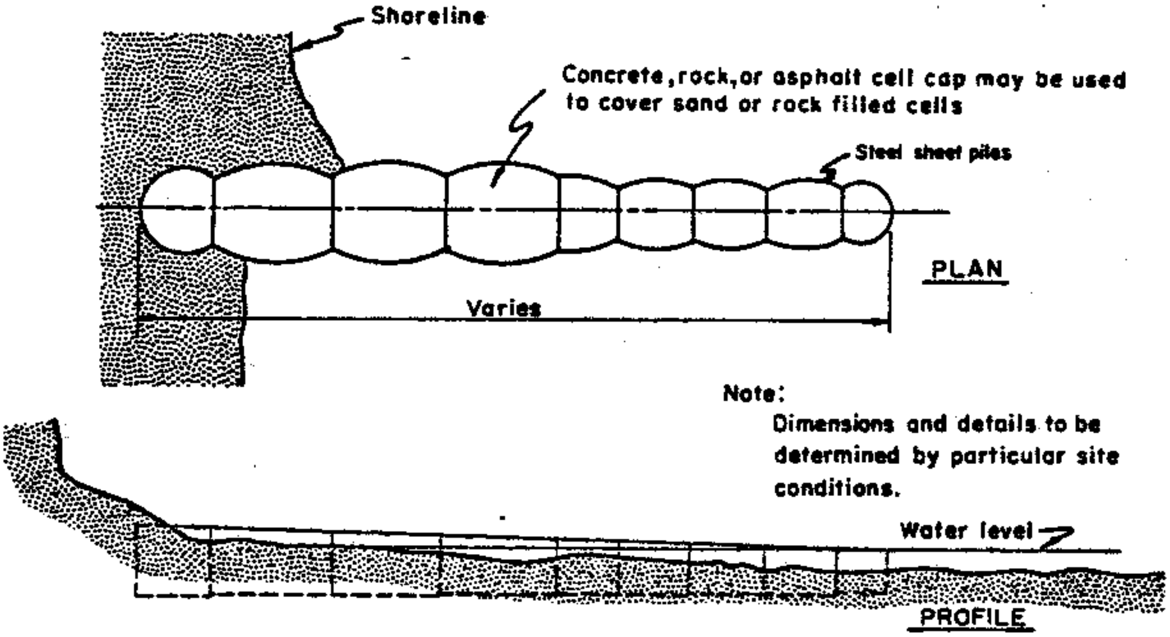
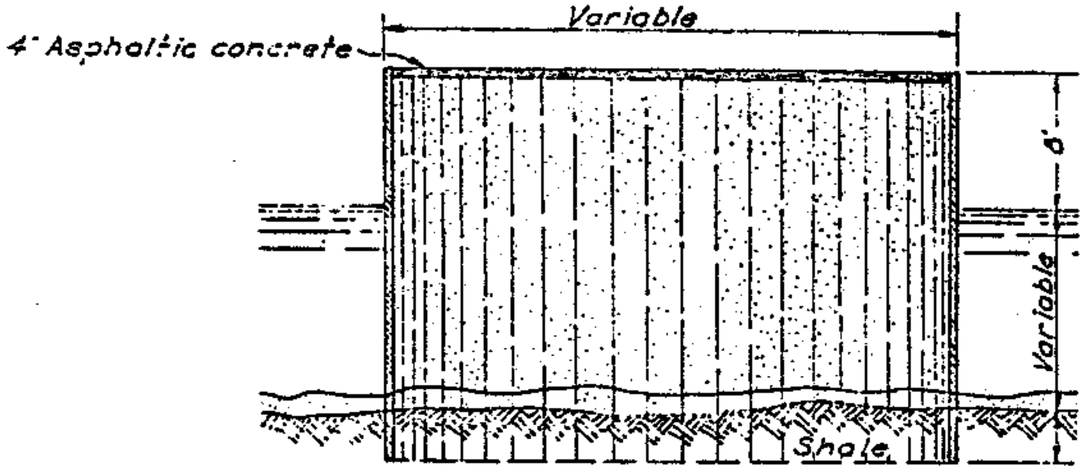


Figure 4.8 Diaphragm Type of Cellular Groin (CERC, 1977, p. 6-80)

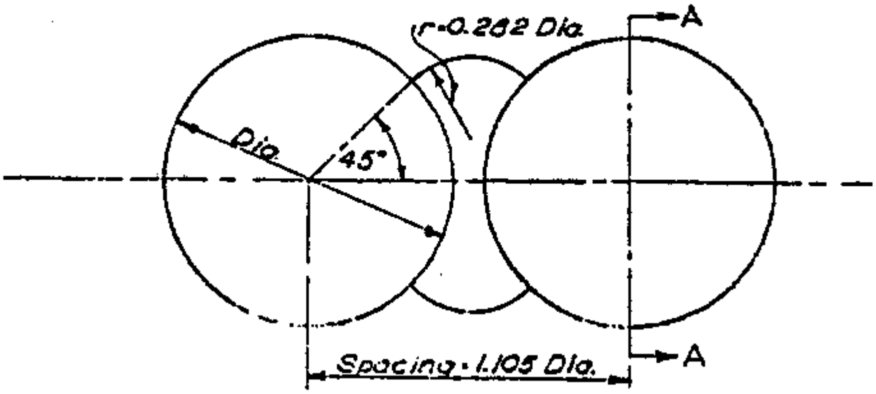
The circular type (Figure 4.9) consists of complete circles connected by shorter arcs. Figures 4.10 and 4.11 typify the designs of two large-scale structures. Each cell must be stable against sliding, overturning, and rupture in the web and interlocks. Rupture is often traced to driving the piles out 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

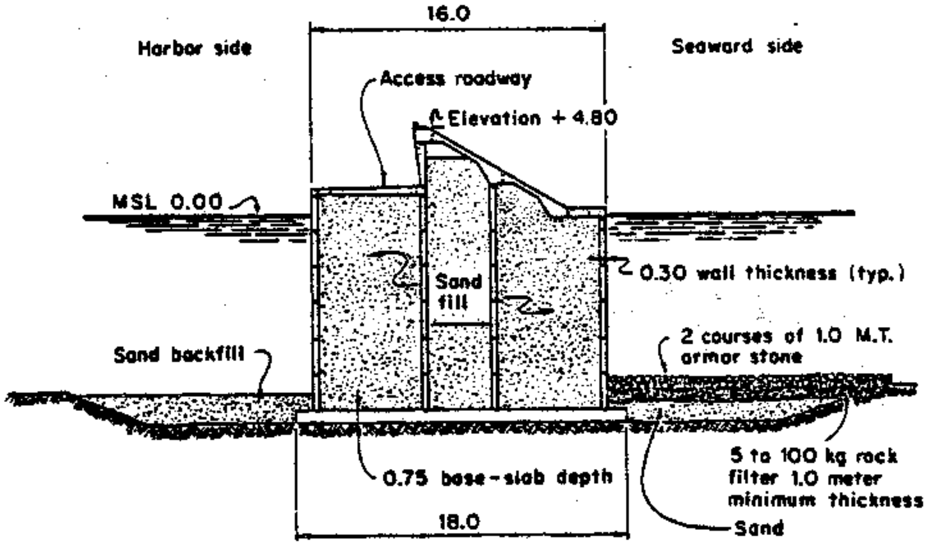


TYPICAL SECTION A-A



TYPICAL PLAN

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)

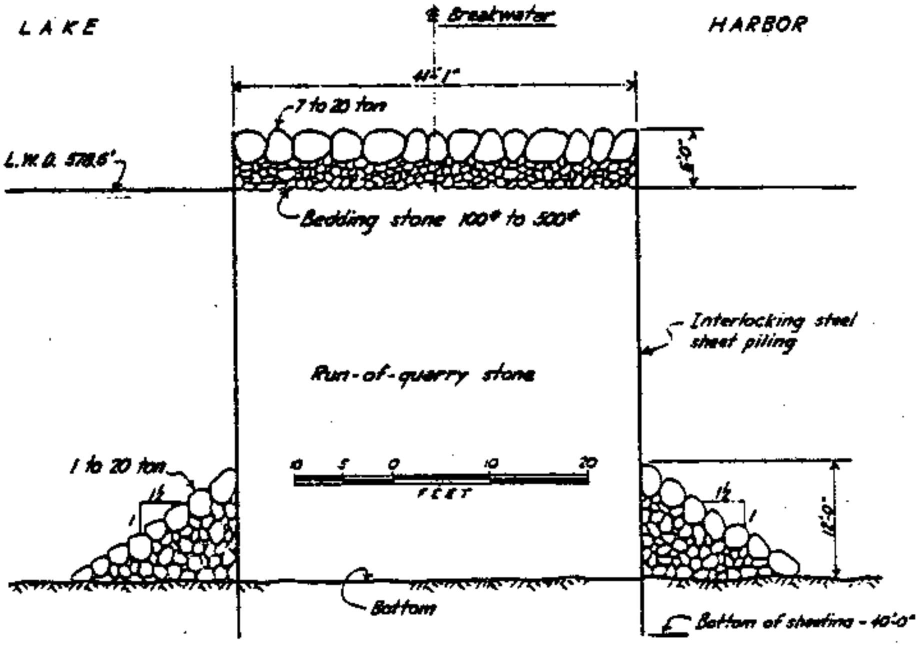


Figure 4.11 Cross-Section through Cellular Sheet Pile Breakwater at Calumet on Lake Michigan (Quinn, 1972, p. 252)

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

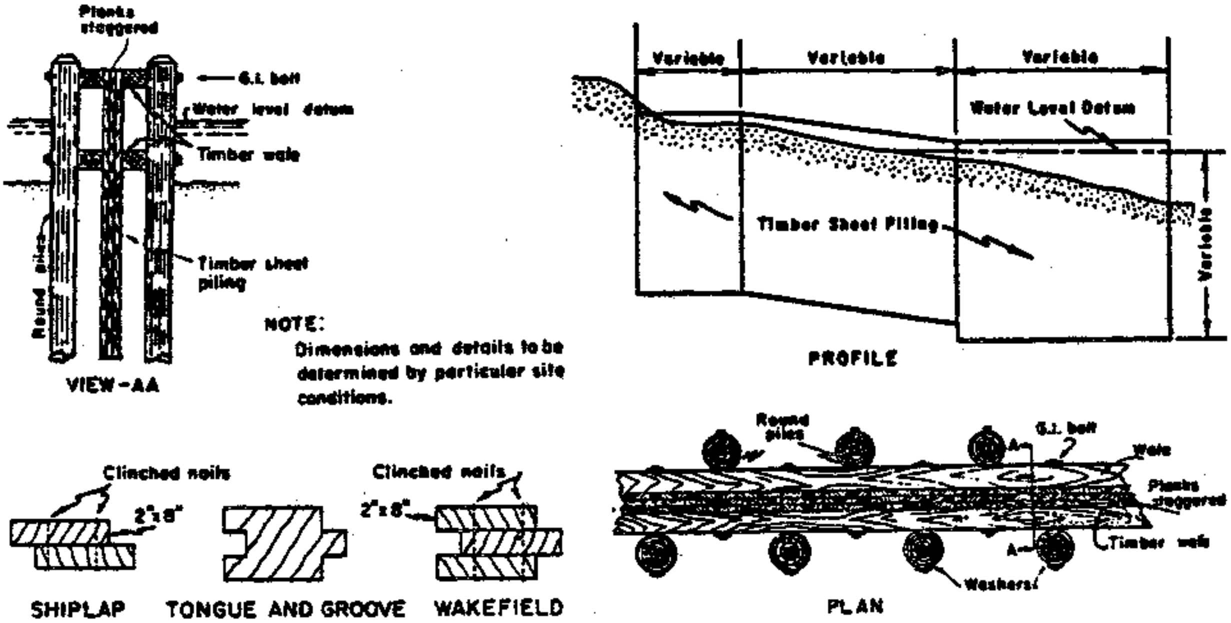
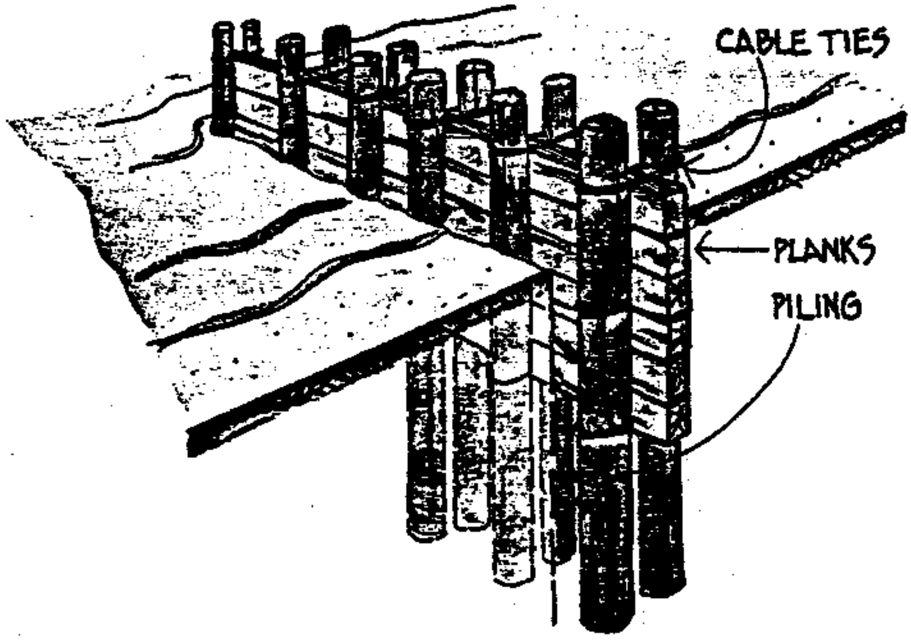
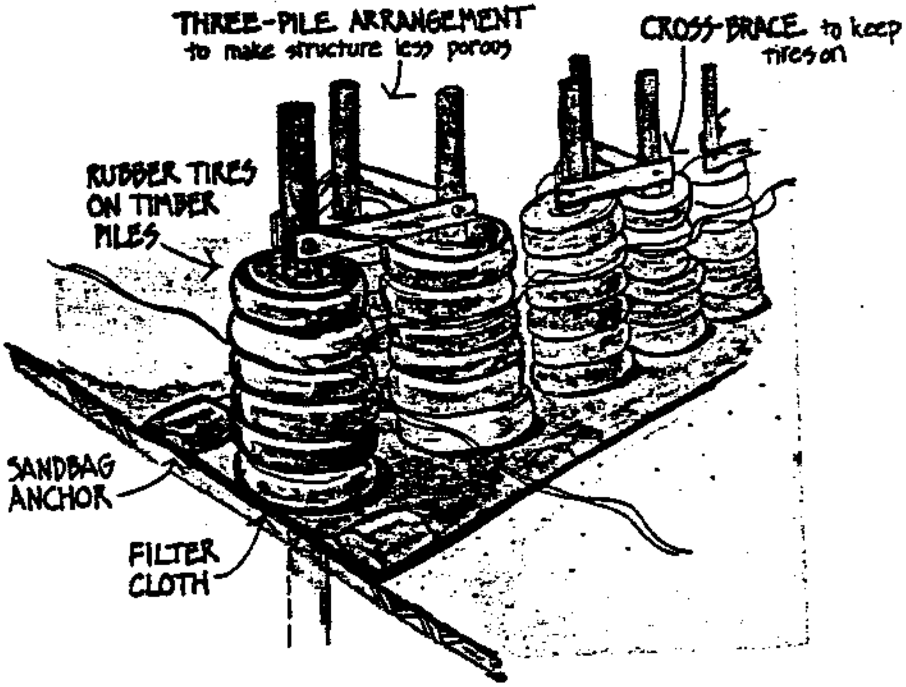


Figure 4.12 Typical Timber Sheet Pile Groin (CERC, 1977, p. 6-77)



a. Timber Groin



b. Timber Breakwater

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

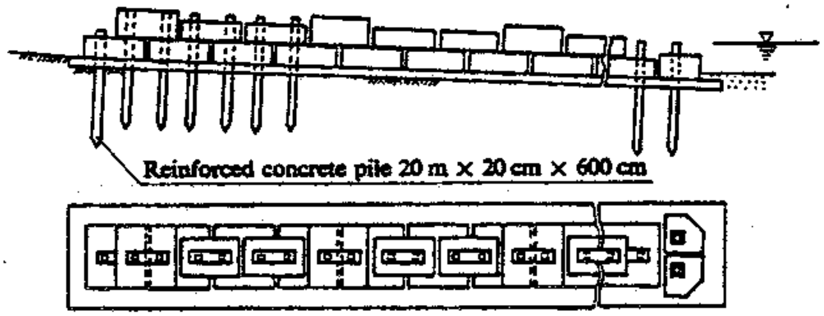


Figure 4.14 Concrete Block Groin, Niigata, Japan (Horikawa, 1978, p. 331)

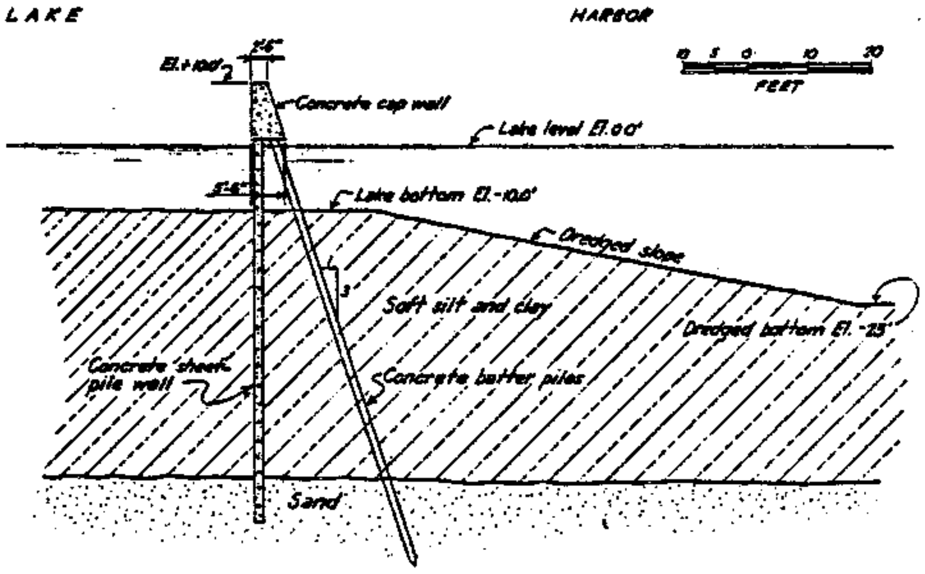


Figure 4.15 Concrete Sheet Pile Breakwater (Quinn, 1972, p. 256)

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

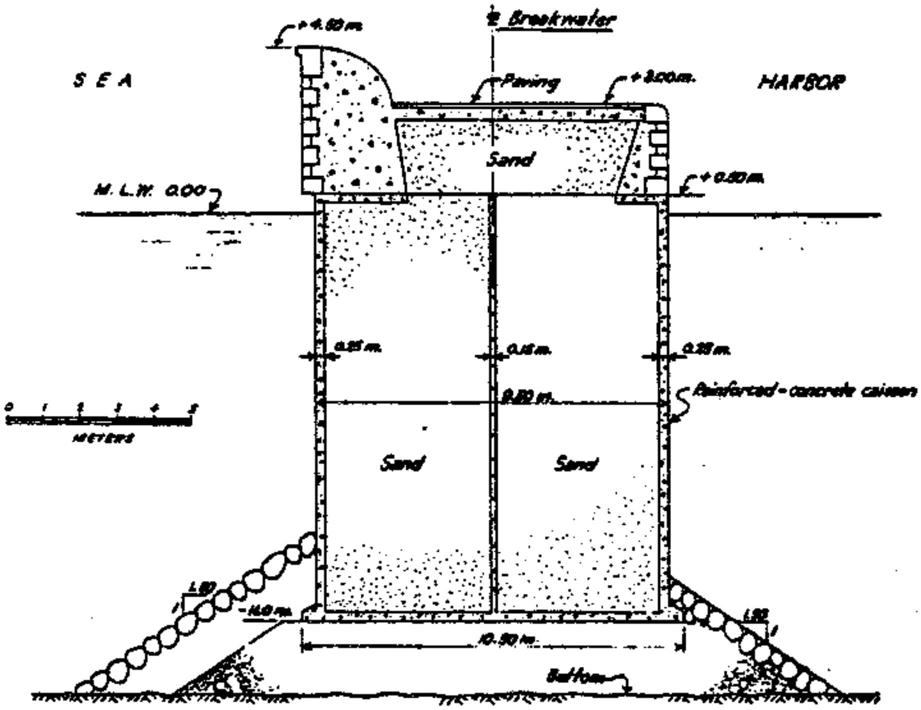
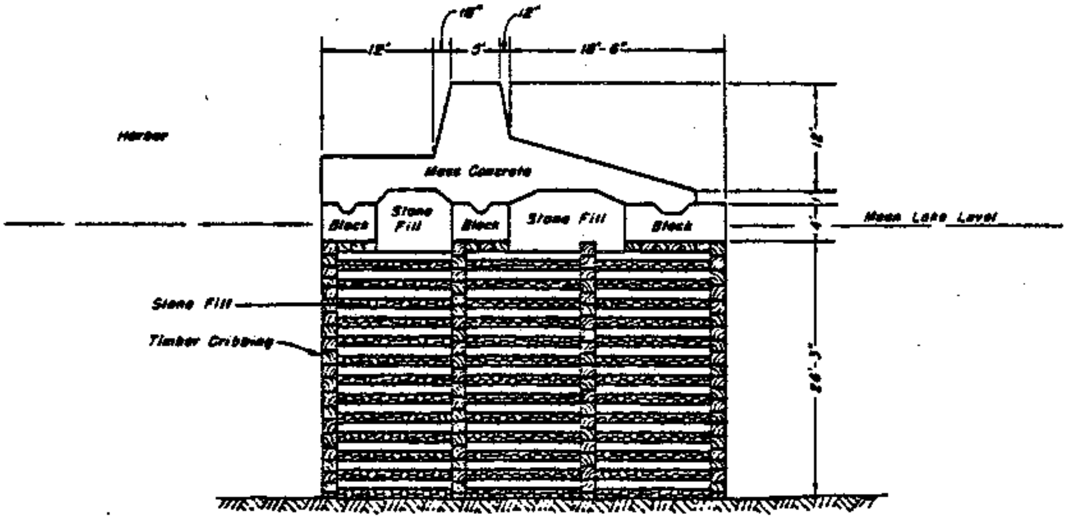


Figure 4.16 Concrete Caisson Breakwater, Helsingborg Harbor, Sweden (Quinn, 1972, p. 249)



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

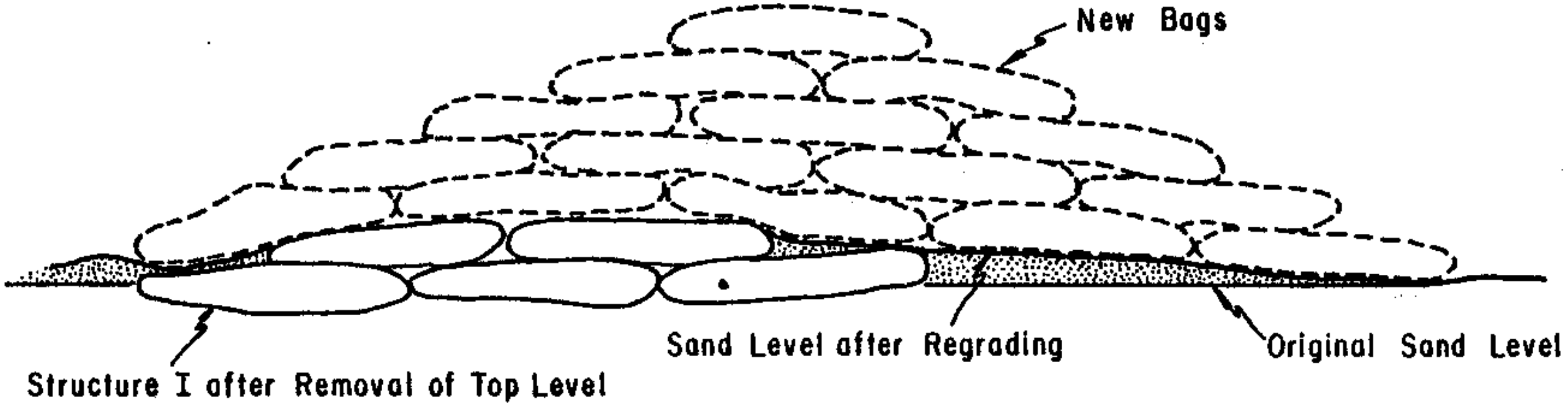


Figure 4.18 Sandbag Groin Experimental Cross-Section (Ray, 1977, p. 22)

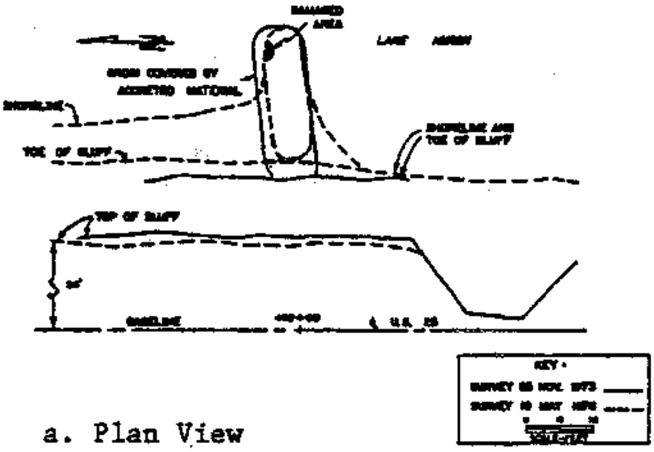
are given by Ray (1977) and additional information is expected in the Shoreline Erosion Control Demonstration Program report.

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).

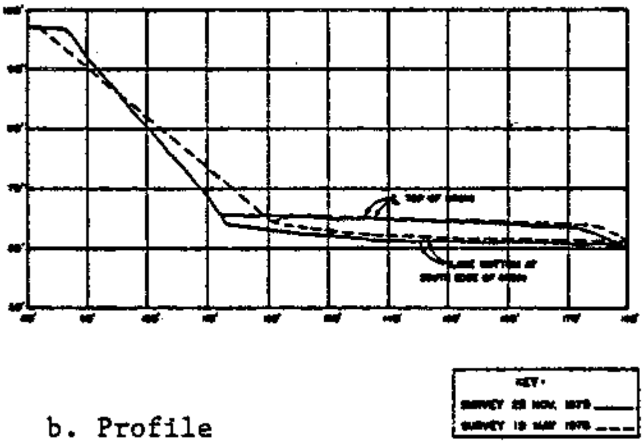
### Rock Mastic

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

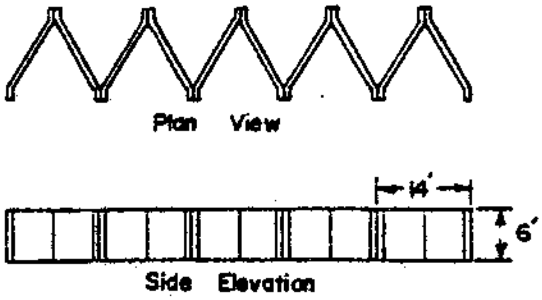


Figure 4.20 Precast Concrete Inshore Breakwater (Hanson, Perry and Wallace, 1978, p. 26)

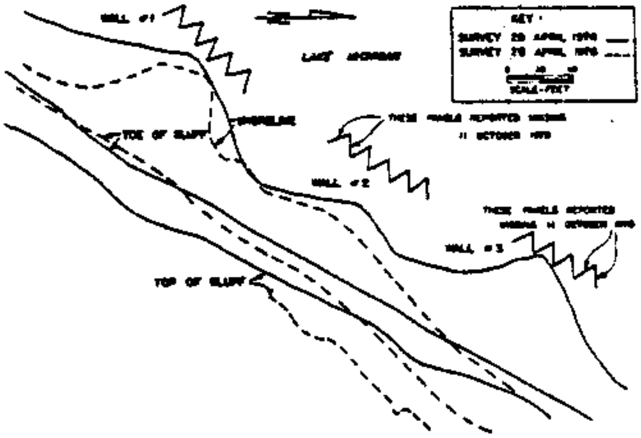


Figure 4.21 Concrete Zig-Zag Wall Breakwater, Pere Marquette Township, Michigan (Brater, et. al., 1977, p. 43)

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

## Other Materials

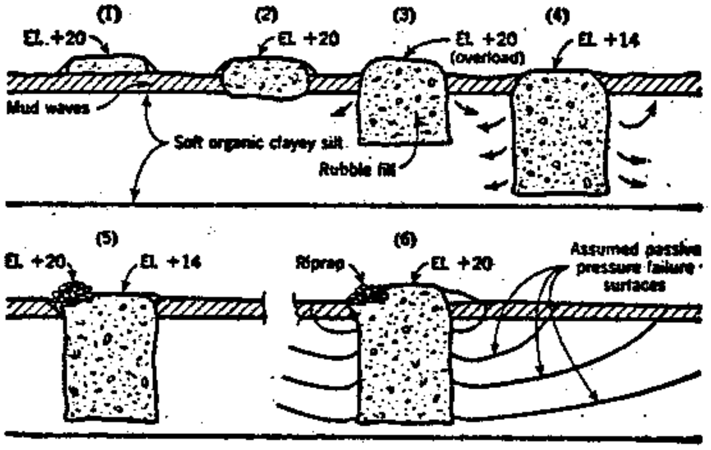
Any material that has an acceptable lifespan, is non-polluting and will remain stable under the imposing environmental forces has potential for shore protection construction. Low cost surplus ships, barges and drydocks are nontraditional building materials, yet can suitably perform as offshore portions 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).

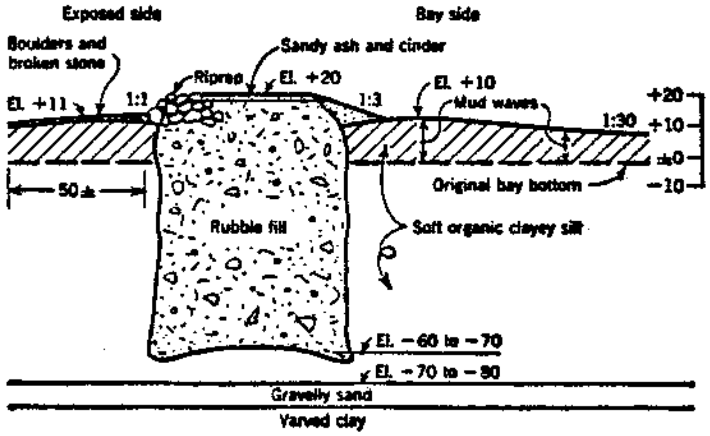
A rubble dike breakwater to protect small craft at the New York World's Fair Marina was built entirely of no-cost fill. Truckers paid a premium for the privilege of convenient disposal of heavy construction debris and rubble. The only method of achieving a stable embankment was to displace the 70 to 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.