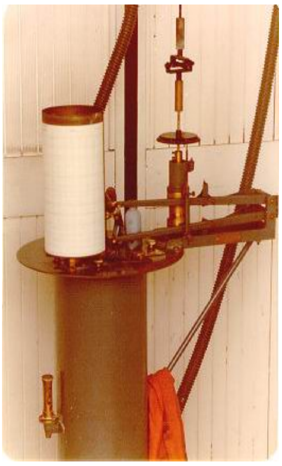
JABATAN KERJA RAYA MALAYSIA



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SHORE PROTECTION GUIDELINES

UNIT MARITIM
CAWANGAN PANGKALAN UDARA DAN MARITIM
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KUALA LUMPUR

CHAPTER 1

SHORE PROTECTION: INTRODUCTION

1.0 OBJECTIVES

This database is intended for planning, regulatory, and etc for local government officials whose duties include some involvement in shoreline erosion prevention measures. The discussion is limit to the shorelines of sheltered waters that are not subject to the direct action of undiminished oceanic waves.

The erosion problems that are experienced today are often caused by failure to recognize that shorelines have the areas of continuous and sometimes dramatic change. This lack understanding of shoreline processes has been catastrophic for both private and public. The objective of the report is to show that the situation is not without remedies and large variety of reasonably low cost alternatives are available Therefore before any action is taken, it is important to recognize and understand the natural forces at work in the general area of a propos project. By considering the overall view rather than condition just at the site, a broader perspective of the problem and possible solutions is developed, and a more informed decision can be made.

The database is aimed to slow or arrest erosion problems. However, successful use of the material presented depends on numerous factors that are peculiar to individual situations.

1.1 SHORELINE PROCESSES

The first requirement in solving an erosion problem or reviewing a proposed solution is to understand the processes and forces at work. Without such basic knowledge, any solutions are likely to be misguided and inappropriate. The following presents basic information about shoreline processes as a foundation for the subsequent discussion.

1.1.1 Wave Action

While waves are always present on the open coast, they are not continuously active in sheltered waters. Nonetheless, they are still the major cause of erosion in all coastal areas. Understanding how wave action influences shoreline processes requires familiarity with several basic characteristics of waves: height, period, and length (Figure 1.1). Wave height is the vertical distance between the wave crest and trough. Wave period is the time it takes two successive wave crests to pass a stationary point, and wavelength is the distance between successive crests.

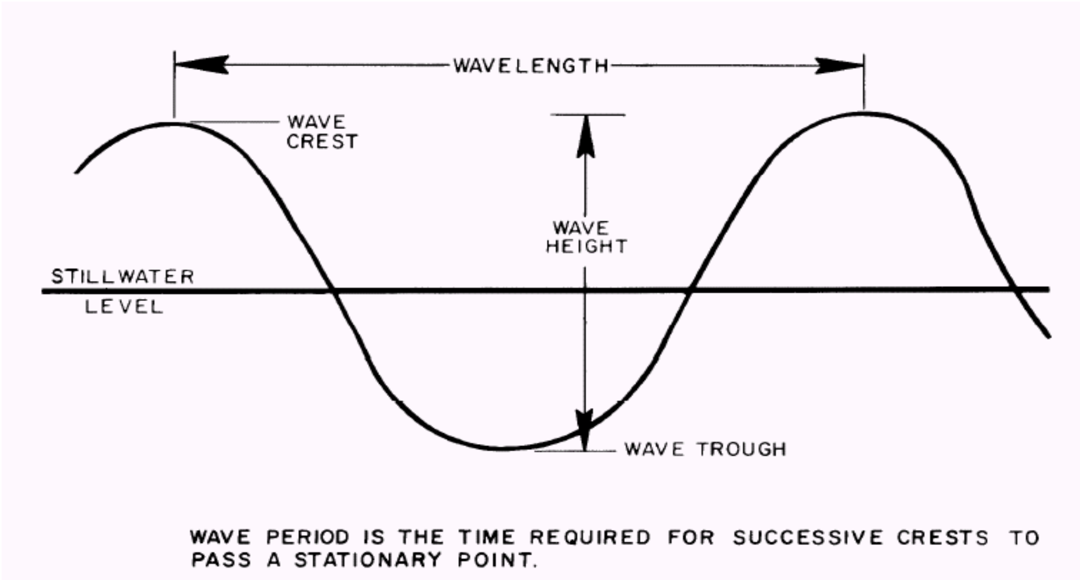

Figure 1.1: Characteristic of Waves

As a wave moves through deep water (depths greater than one half the wavelength), these basic characteristics do not change. When a wave approaches shallower water near the shore, the period remains constant, the forward speed and wavelength decrease, and the height slightly increases. The wave begins to feel the “bottom", and its profile steepens as its gently rolling shape sharpens to a series of pointed crests with intervening flat troughs. When the wave height is about 80 percent of the water depth, the wave can no longer steepen and it breaks. For example, a 5-foot wave breaks in a water depth of about 6.5 feet.

Important wave properties are demonstrated when a series of regular waves meet a solid barrier, such as a breakwater (Figure 1.2). Wave diffraction occurs when the waves pass the barrier, and part of their energy is transferred along the crests to the quiet area in the shadow of the structure. Diffraction causes waves to form in the shadow zone that are smaller than waves in the adjacent unprotected zones.

Wave reflection occurs on the offshore side of the breakwater. While waves passing the structure are diffracted, the portions striking the breakwater are reflected like a billiard ball from a cushion. If the structure is a smooth vertical wall, the reflection is nearly perfect, and if the wave crests are parallel with the breakwater, the reflected and incoming waves will reinforce each other to form standing waves, which are twice as high as the incoming waves. These can cause considerable scouring of the bottom. If the waves approach at an angle, no standing waves form, but the resulting sea-state is choppy because the reflected waves cross the path of incoming waves. This could also contribute to bottom scour.

The final important wave characteristic is evident when waves break either on a beach or structure. The uprush of water after breaking is called runup and it expends the wave’s remaining energy. The runup height depends on the roughness and steepness of the structure or beach and the characteristics of the wave.

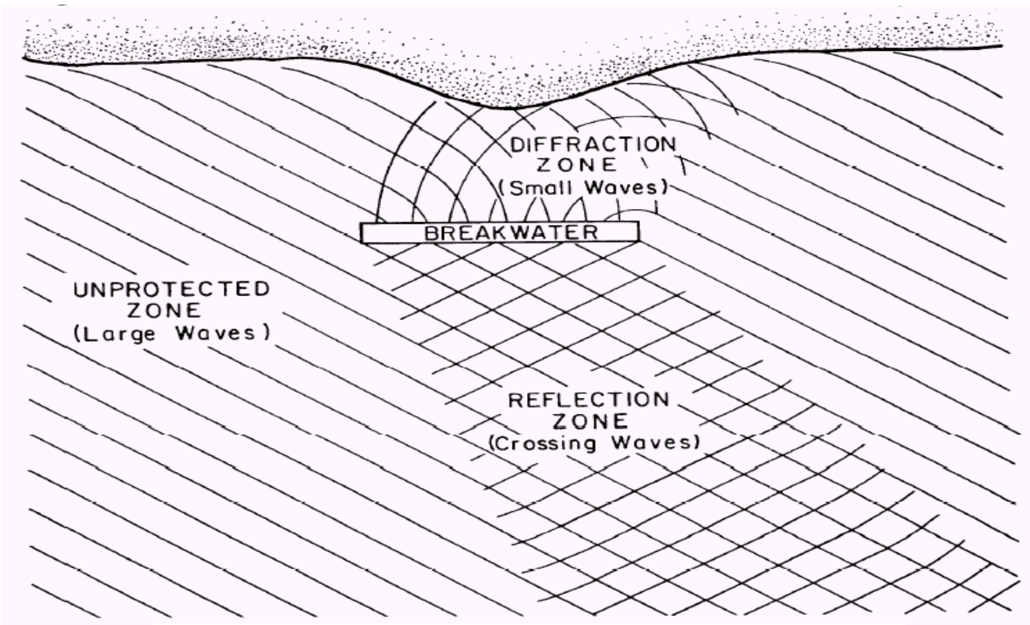

Figure 1.2: wave Diffraction and Reflection

The wave generation process depends on several important factors, the most prominent being wind, although the movements of pleasure craft and large vessels are also significant sources of wave activity. The height of wind-driven waves depends on the wind speed, duration, fetch length, and water depth. Wind speed is obviously important, but duration (length of time the wind blows) must also be considered because wind action must be sustained for wave growth. Fetch is the over-water distance wind travels while generating waves. At a given site, the maximum fetch length, or longest over-water distance, is generally the most important. Less important, but still critical, is the average water depth along the fetch. Deeper water allows for somewhat larger waves because of decreased bottom friction.

1.1.2 Sediment Transport

The large variety of shoreline materials ranges from rock cliffs to boulders, cobbles, gravel, sand, silt, and clays. Geologists and engineers have developed several classification systems for these materials and an example is given in Table 1.1.

Table 1.1: Classification of Shoreline Materials

Size Description	Particle Size Range	
	(Inches)	(mm)
Boulder	greater than 10	greater than 256
Cobble	10 – 3	256 - 76
Gravel	3 - 0.18	76 – 4.8
Sand	0.18 - 0.003	4.8 -0.07
Silt	0.003 - 0.00015	0.07 -0.004
Clay	smaller than 0.00015	smaller than 0.004

Littoral (shoreline) materials are derived from the deterioration and erosion of coastal bluffs and cliffs; the weathering of rock materials found inland and transported to the shore by rivers and streams; the disintegration of shells, coral or algae and the production of organic material (generally peat) by coastal wetlands.

Failure or erosion of a bluff causes material to be deposited at the base. Waves sort this material and carry the fine-grained silts and clays far offshore where they settle to the bottom. The original deposit is eventually reduced to sand and gravel, which form a beach. If no other littoral material is carried to the sit by waves, even the sand and fine gravel will eventually disappear down the coast or offshore, leaving only coarse gravel behind However, a new supply of material may be deposited on the beach by a fresh failure of the bluff, and the process begins again. In most cases, littoral materials comprising beaches are derived from erosion of the shoreline itself.

Rivers and streams carry sediments eroded from mountain forests, and fields, particularly during floods. The sediments are usually smaller than sand because the coarser particles are not easily transported by the streams. Except where streams traverse sand drainage basins, the contribution to beach building from this source is usually smaller than from the first source.

Coral reefs, shells, and other plant or animal matter are another material source. They gradually break and weather in carbonate particles, which are, for instance, the primary component of beaches south of Palm Beach, Florida. Swamps, marshes, a coastal wetlands produce peats and other organic matter. Too light to remain in place under continued wave action, they are ultimately washed offshore unless stabilized.

Littoral materials are transported along the shore by wave action. As waves approach the shore, they move to progressive shallower water where they bend or refract until finally breaking at an angle to the beach. The broken wave creates considerable turbulence, lifting bottom materials into suspension and carrying them up the beach face in the general direction of wave approach. A short distance up the beach, the motion reverses direction back down the beach slope. In this case, the downrush does not follow the path of the advancing wave but instead, moves down the slope in response to gravity. The next wave again carries the material upslope, repeating the process, so that each advancing wave and the resulting downrush move material along the beach in the downdrift direction. As long as waves approach from the same direction, the alongshore transport direction remains the same.

Littoral materials are also moved by the longshore current. Arising from the action of breaking waves, this current is generally too weak, alone, to move sand. However, the turbulence of breaking wave’s places sand temporarily in suspension and permits the longshore current to carry it downdrift. The sand generally settles out again within a short distance, but the next wave provides the necessary turbulence for additional movement. The downdrift movement of material is thus caused by zigzag motion up and down the beach, and the turbulence and action of the wave-generated longshore current.

1.1.3 Currents

The water at the shore is constantly in motion due to currents as well as waves. Tides produce currents in sheltered bays connected to the open sea. As the tide begins to rise in the ocean (flood tide), the bay's water surface elevation lags behind, generating a current into the bay. As the tide falls (ebbs), the ocean surface drops more quickly so that the bay surface becomes higher and current flows out of the bay. Tidal currents are generally not strong enough to cause erosion problems except in the throat area of tidal inlets connecting bays to the ocean.

1.1.4 Seasonal Change

The most notable seasonal change at sheltered sites is the frequency, direction, and severity of high winds. Storms generate strong winds that often approach from entirely different directions than winter squalls. The manners in which storm winds align with fetch lengths at the site figures prominently in evaluating the potential for wave damage. If the most severe winds striking a site are generally along the longest fetch length, structures should be built more strongly than if severe winds rarely approach from that direction.

1.1.5 Water Level Variations

The water surface elevation itself constantly changes with time. The Stillwater level, the water level with no waves present, changes because of astronomical tides and storms.

1.1.5.1 Astronomical Tides

Tides are caused by the gravitational attraction between the earth, moon, and sun, and are classified as diurnal, semidiurnal, or mixed. Diurnal tides have only one high and low each day. Semidiurnal tides have two approximately equal highs and two approximately equal lows daily. Mixed tides, on the other hand, exhibit a distinct difference in the elevation of either the two successive highs or two successive lows. In addition, at locations with mixed tides, the characteristics of the tide may change to diurnal or semidiurnal at certain times during the lunar month (Figure 1.3).

In addition, the tidal range, or difference between the high and low, tends to fluctuate throughout the lunar month. Spring tides have larger than average ranges with higher high and lower low tides. Neap tides are exactly opposite with smaller ranges, lower highs, and higher lows. Spring tides occur with full and new moons because the gravitational attraction of both the sun and moon act along the same line, tending to exaggerate the difference between the high and low tides. At neap tides (during quarter moons), the pull of the sun and moon are out of phase, somewhat cancelling their individual effects and causing correspondingly smaller tidal ranges. Differences in tidal range are also caused by the varying distance to the moon as it orbits the earth, the declination of the moon's orbit, and the declination of the earth's orbit about the sun.

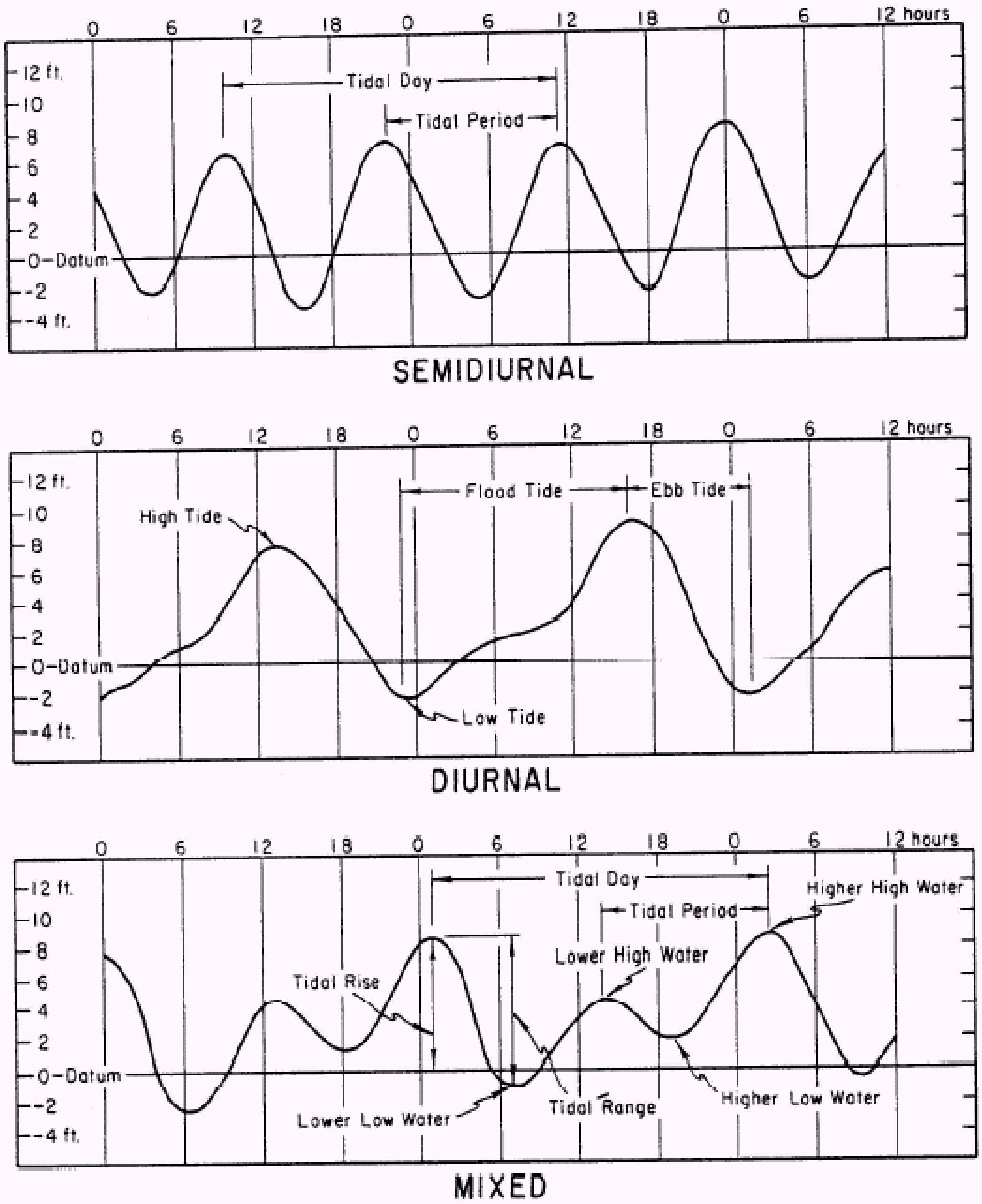

Figure 1.3: Types of Tides [Wiegle 1953]

Tide levels are used as reference elevations on maps, charts, and engineering drawings. Key reference elevations or datum (Figure 1.4), which is important because of their wide use, is defined in the *Glossary*.

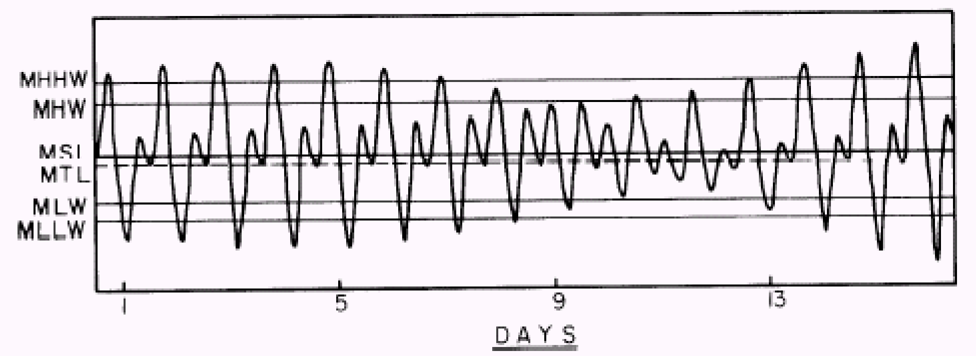

Figure 1.4: Illustration of Tides Datums [After Harris 1981]

1.1.5.2 Storm Effects

Storms tend to increase the stillwater level because of atmospheric pressure differences, high winds, and the effects of large breaking waves. Atmospheric pressure differences across a large water body can commonly cause one or two foot rises in the water level in the lower pressure area. The stress on the water surface from high storm winds also tends to drive the water on shore to above normal levels (storm setup) until balanced by the tendency for the water to flow back to a lower level. These high winds also generate large waves, which tend to pile water on shore as they break, raising the stillwater level further.

Enclosed water bodies can also respond to storm forces by seiching. This occurs when storm winds drive the water surface higher at the downwind end of a lake. As the storm passes, this pent-up water is released, causing it to move toward the opposite end of the lake, resulting in oscillations. This back and forth movement (seiching) will noticeably continue for several cycles.

1.2 EROSION PROBLEM

1.2.1 Importance of Shoreform

The land-sea boundary is characterized by many shapes and configurations. Geologists have devised elaborate classification systems to describe these various features. For the purpose of understanding basic shoreline processes and for designing appropriate corrective measures, however it will only be necessary to informally classify shorelines as either bluffs, low erodible plains (including sandy beaches), or wetlands. Many shorelines, of course contain two or even all three basic features. For instance, a shoreline may be a high bluff with a sand beach at the base, or a gently sloping plain -fronted by a marsh. In that case, one must consider the interaction of these features with the erosive forces and then single out the most important for further study.

1.2.1.1 Bluff and Cliff Shorelines

Cliff shorelines consist primarily of resistant rock. On the other hand, bluff shorelines are composed of such sediments as clays, sands and gravels, or erodible rock. Cliffs rarely suffer severe or sudden erosion but undergo slow, steady retreat under wave action over a long period.

Erosion problems are most common along bluff shorelines where a variety of forces and processes act together. The most prevalent causes of bluff erosion and recession are scour at the toe (base) by waves and instability of the bluff materials themselves. Slope stability problems are highly technical and can only be analyzed correctly using methods of geotechnical engineering. Therefore, they are beyond the scope of this report. A brief discussion of factors affecting slope stability and how to recognize potential problems is presented below. It is suggested that if there are slope stability problem, a registered professional geotechnical engineer should be contacted.

Soils are not generally stable at a vertical face, but form a slope that varies with the soil and groundwater conditions. This slope forms as a result of a series of failures whose nature depends on whether the soil is cohesive (clay) or granular (sand, silt, gravel, etc.). Cohesive soils generally slide along a circular or curved arc, the soil moving downward as it rotates along the failure surface. Granular soils, on the other hand, fail when vertical-sided blocks drop to the bottom or when the soil suddenly flows down an inclined plane. Height is a factor because high bluffs (over 20 feet) impose greater stresses and are likely to suffer more severe stability problems than low bluffs. The internal strength of soils can be decreased by groundwater and seepage flows within the bluff. For instance, rainwater is naturally absorbed and seeps down to lower levels. Soils, such as coarse sand, which allow rapid and free passage of water, are permeable. On the other hand, impermeable soils, such as clay, do not allow the free flow of water except through cracks or other openings. The large tree's roots penetrate the clay layer and provide a path for seepage to the sand layer beneath. Likewise, as the clay fails, cracks form at the surface which provides a path for seepage to penetrate the soil, further weaken it, and accelerate the failure process. Water can also enter the clay along the existing circular failure surface, leading to further movement.

Once seepage penetrates the clay and reaches the permeable sand layer, it passes freely to the lower clay layer where it flows along the clay's surface and exits the bluff face. This seepage can increase the risk of slope failure. In addition, surface runoff can erode the bluff face, causing gullies and deposits of eroded material on the beach below. The seepage exiting the bluff at the clay layer can also cause surface erosion.

The added weight or loading of buildings and other structures can increase soil stresses and contribute to slope failure. Structures located near the top edge of the bluff have the greatest impact. An in-ground pool, even when filled, weighs less than the soil it replaces and would not adversely affect stability, provided no leakage exists and splashing is minimized.

The other major cause of bluff shoreline problems is wave action at the toe. Waves move clays and silts offshore while leaving sands and gravels for the beach. During storms, however, waves can reach the bluff itself and erode or undercut the toe. Depending on the bluff soil characteristics, only a short time may be needed under such conditions for the entire bluff face to fail.

The slope of the offshore bottom is important to wave action on a bluff. If the offshore slopes are steep, deep water is closer to shore, more severe wave activity is possible, and maintenance of a protective beach is more difficult. Flat offshore slopes, on the other hand, result in shallower water near the shoreline, which inhibits heavy wave action at the bluff and provides for potentially better protective beaches.

1.2.1.2 Low Erodible Plains and Sand Beaches

Beaches and erodible plains are composed of loose sediments ranging from silts to gravel that slope gently up and away from the water's edge. Because they seldom reach more than five to ten feet above Stillwater level, such shorelines are susceptible to flooding as well as erosion. Erosion problems are caused by wave action, although wind can be important in some cases.

Figure 1.5 depicts an idealized beach profile. Waves approach from offshore, finally breaking and surging up the foreshore. At the crest, the profile flattens considerably, forming a broad berm inaccessible to normal wave activity. The beach berm is often backed by a low scarp formed by storm waves, a second berm, and eventually a bluff or dune.

During periods of either increased water levels or wave heights, the sand above the low water level is eroded, carried offshore, and deposited in a bar. Eventually, enough sand collects to effectively decrease the depths and cause the storm waves to break farther offshore. This reduces wave action on the beach, and helps re-establish equilibrium. At open coast sites, the process eventually reverses, and long-period swells again return the sand to the beach after storms. At sheltered sites where no exposure to oceanic swells exists, the recovery does not occur, and storm caused erosion becomes permanent.

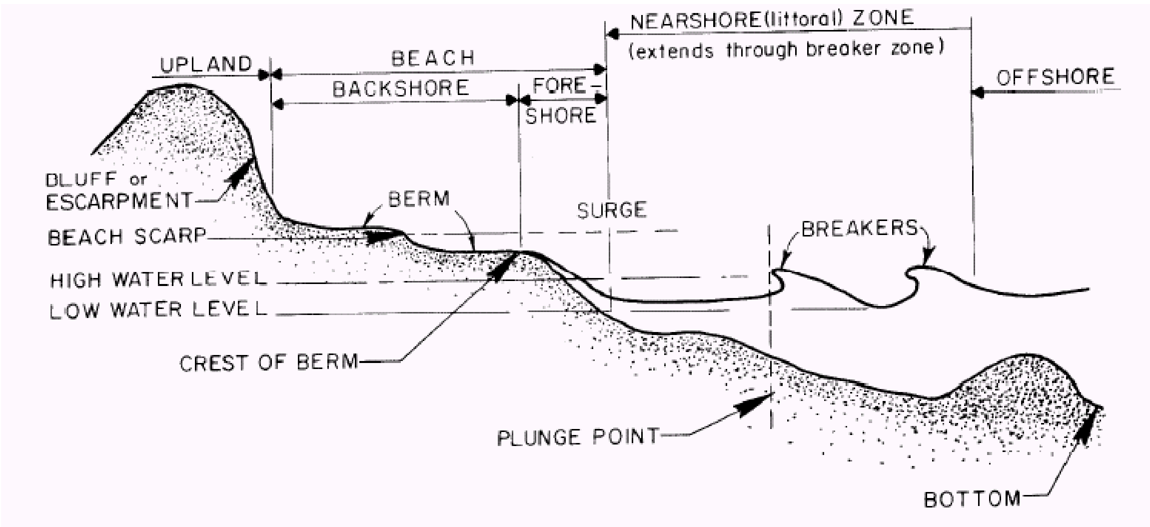

Figure 1.5: An Idealized Beach Profiles [After U.S. Army Corps of Engineers 1977c]

1.2.1.3 Wetlands

Wetlands usually occur in combination with sand beaches or low erodible plains. Wetlands are defined as:

Those areas that are inundated or saturated by surface or groundwater at a frequency and duration sufficient to support, and that under normal circumstances do support, a prevalence of vegetation typically adapted for life in saturated soil conditions. Wetlands generally include swamps, marshes, bogs, and similar areas" [U.S. Army Corps of Engineers (1977b).

Marsh plants are primarily herbaceous (lacking woody stems), such as grasses, sedges, and rushes. The species present depend on location and whether the marsh is low (regularly flooded) or high (irregularly flooded).

Until recently, wetlands were regularly drained and filled for new development or agriculture. They are now recognized as a vital link in the food chain of the aquatic community and for their capacity to absorb water-borne pollutants. However, more importantly, they provide shore protection by absorbing energy of approaching waves and trapping sediment carried along by currents.

The shore protection qualities of wetlands are particularly important when they provide a buffer zone in front of a sandy beach or other area vulnerable to erosion. While not providing full protection, they effectively diminish wave energy and allow for less massive and costly backup protection.

1.2.2 The Causes of Erosion

1.2.2.1 Wave Action

Wave action is the most obvious cause of erosion.

1.2.2.2 Littoral Material Supply

Waves keep the littoral materials constantly moving downdrift. As long as equal quantities of material are transported from the updrift direction, the shoreline remains stable. When the updrift supply exceeds the amount moving downdrift, the shoreline accretes (material accumulates). However, when the updrift supply is deficient, the shoreline retreats.

Much of the littoral material supplied to shorelines results from updrift erosion. Therefore, if large amounts of updrift shoreline are suddenly protected, material is lost to the littoral system. This decreases the supply to the downdrift shores, resulting in erosion problems unless they are also protected.

Determining the transport direction is necessary in some cases but usually difficult because of variations in wave directions throughout the year. Summer winds (and waves) may be primarily from one direction, while winter storm winds may come from an entirely different quadrant. When winds and waves change direction, the transport direction also will changes (transport reversal). The gross longshore transport rate is the total amount of sand that annually moves past a point regardless of direction. The net transport rate is the quantity moved in one direction minus that moved in the other direction. For example, if the amount of sand moved in one direction in one year was equal to the amount moved in the other direction, the net transport rate would be zero.

1.2.2.3 Wind

Wind is a problem where large volumes of sand may be transported by prevailing breezes to form dunes. This mechanism seldom occurs along sheltered shorelines.

1.2.3 The Effects of Erosion

The most obvious and noticeable effect of erosion is the loss of shor6front property. Less apparent are the increases in sedimentation caused by erosion in adjoining areas since all materials eroded from a shoreline at one point are eventually deposited elsewhere. It is likely this will occur in deeper water such as a navigation channel crossing or closely paralleling the shore. This can be as serious a problem, in terms of total utilization of the shoreline, as the eroding property. All possible effects of increasing or decreasing sediment movement by any actions should be carefully considered. Significant effects of either kind will probably make it impossible to obtain required federal and state permits.

CHAPTER 2

SHORE PROTECTION STRUCTURES

2.0 AVAILABLE OPTION

Three options are available when confronted with an erosion problem: take no action, relocate endangered structures, or take positive action to halt the erosion. The latter includes devices that armor the shoreline, intercept or diminish wave energy offshore, or retain earth slopes against sliding. Any alternative requires evaluation of the shoreform, planned uses of the land, money and time available, and other effects of the decision.

a. No Action

The no action alternative is used to help evaluate different options. When confronted with an erosion problem, the first, reaction is to act immediately. What is not realized at first is the expense of even low cost solutions. Therefore, it is advisable to estimate the losses involved in doing nothing, particularly if only undeveloped land or relatively inexpensive structures are threatened

b. Relocation

No action is generally unacceptable, and in most cases, steps must be taken. Before investing in shore protection, however, physical relocation of endangered structures should be considered. This could involve moving them either to a different area or farther from the water on the same lot. Moving a building involves considerable expense which could be wasted if it is not moved back far enough. Therefore, it is necessary to evaluate the erosion rate (feet/year) and the likelihood that this rate will continue at or below historical levels through the required life of the setback.

c. Take positive action to halt the erosion

Take positive action to construct shore protection structures such as bulkheads, seawalls, revetment, vegetation, infiltration, drainage controls and etc to protect the shore from erosion.

2.1 BULKHEADS AND SEAWALLS

"Bulkheads" and "seawalls" are terms often used interchangeably in referring to shore protection structures. Bulkheads are retaining walls, however, whose primary purpose is to hold or prevent sliding of the soil. While they also provide protection from wave action, large waves are usually beyond their capacity. Seawalls, on the other hand, are massive structures used to protect backshore areas from heavy wave action. Their size generally places them beyond the range of low cost shore protection. They are also not generally needed in sheltered waters where large waves do not occur.

Bulkheads may be employed to protect eroding bluffs by retaining soil at the toe, thereby increasing stability, or by protecting the toe from erosion and undercutting. Bulkheads are also used for reclamation projects where a fill is needed at a position in advance of the existing shore. Finally, bulkheads are used for marina and other structures, where water depth is needed directly at the shore (Figure 2.1).

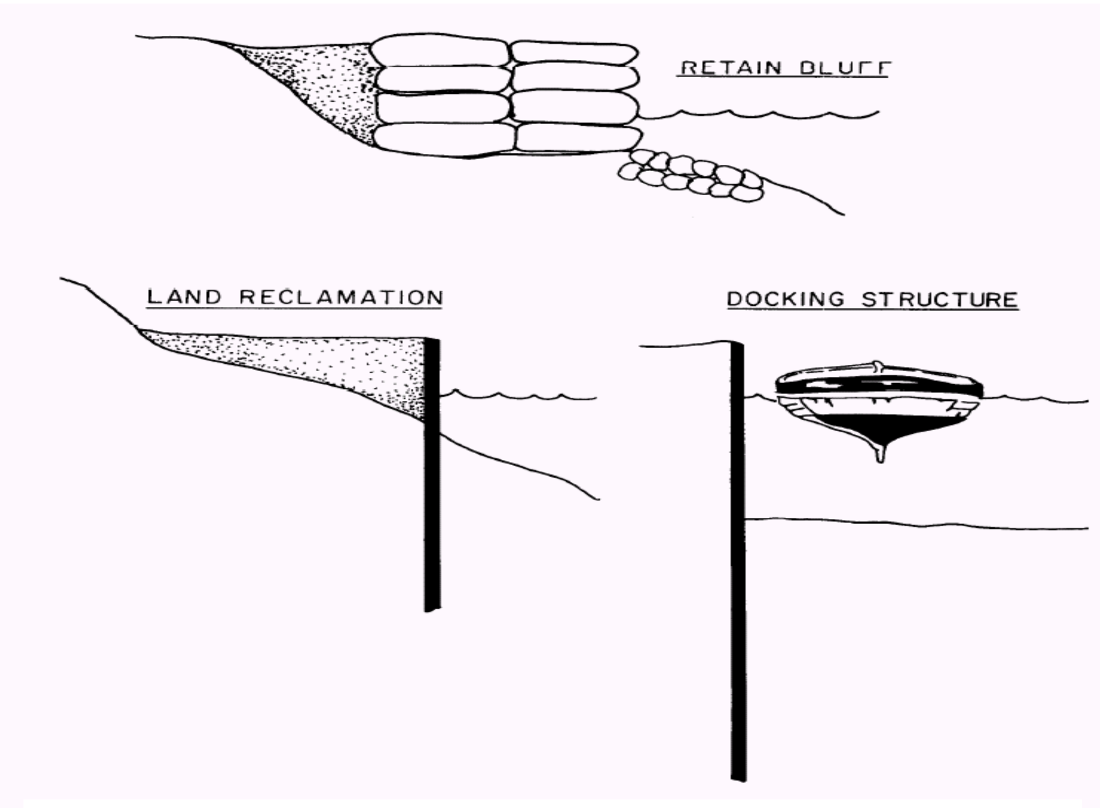

Figure 2.1: Uses of Bulkheads

Construction of a bulkhead does not insure stability of a bluff. If a bulkhead is placed at the toe of a high bluff steepened by erosion to the point of incipient failure, the bluff above the bulkhead may slide, burying the structure or moving it toward the water. To increase the chances of success, the bulkhead should be placed somewhat away from the bluff toe, and if possible, the bluff should be graded to a flatter, more stable slope.

Bulkheads may be either thin structures penetrating deep into the ground (e.g., sheet piling) or massive structures resting on the surface (e.g., sand or grout filled bags). Sheet pile bulkheads require adequate ground penetration to retain soil. Stacked bag structures do not require heavy pile-driving equipment and are appropriate where subsurface conditions hinder pile penetration. However, they need firm foundation soils to adequately support their weight. Because they do not generally penetrate the soil, they often cannot prevent slides where failure occurs beneath the surface. This limits their effectiveness to sites where the backfill and structure are low.

Bulkheads protect only the land immediately behind them and offer no protection to adjacent areas up and down the coast or to the fronting beach. In fact, because bulkheads normally have vertical faces, wave reflections are maximized, wave heights and overtopping may increase, and scour in front of the structure is more likely. In addition, if downdrift beaches were previously nourished by the erosion of land now protected, they may erode even more quickly. If a beach is to be retained adjacent to a bulkhead, additional structures such as groins or breakwaters may be required.

Since scour can be a serious problem, toe protection is necessary for stability. Typical toe protection consists of quarrystone large enough to resist movement by wave forces, with an underlying layer of granular material or filter cloth to prevent the soil from being washed through voids in the scour apron. Flanking (erosion of the shore around the ends of the structure) can also be a problem. This can be prevented by tying each end into existing shore protection devices or the bank.

Bulkheads may be either cantilevers or anchored (like sheet piling), or gravity structures (like sand-filled bag). Cantilever bulkheads require adequate embedment to retain soil and are used where low heights are sufficient. Toe scour reduces their effective embedment and can cause failure. Anchored bulkheads are usually used where high structures are needed. They also require adequate embedment (although less than cantilever bulkheads) to function properly, but they tend to be less susceptible to toe scour.

Gravity structures eliminate the need for heavy piling driving equipment and are often appropriate where subsurface conditions hinder pile penetration. However, they require strong foundation soils to adequately support their weight, and they normally do not sufficiently penetrate the ground to develop reliable soil resistance on the offshore side. Therefore, they depend primarily on shearing resistance along the base of the bulkhead to support applied loads. Gravity bulkheads also cannot prevent rotational slides in materials where the failure surface passes beneath the structure. Their use, therefore, is generally limited to relatively where their cost is comparable to cantilever sheet pile bulkheads.

2.2 REVETMENTS

A revetment is a heavy facing (armor) on a slope to protect it and the adjacent upland against wave scour (Figure 2.2). Revetments depend on the soil beneath them for support and should, therefore, be built only on stable shores or bank slopes. Slopes steeper than 1 on 1.5 (1 vertical on 1.5 horizontal) are unsuitable for revetments unless flattened. Fill material, when required to achieve a uniform slope, must be properly compacted.

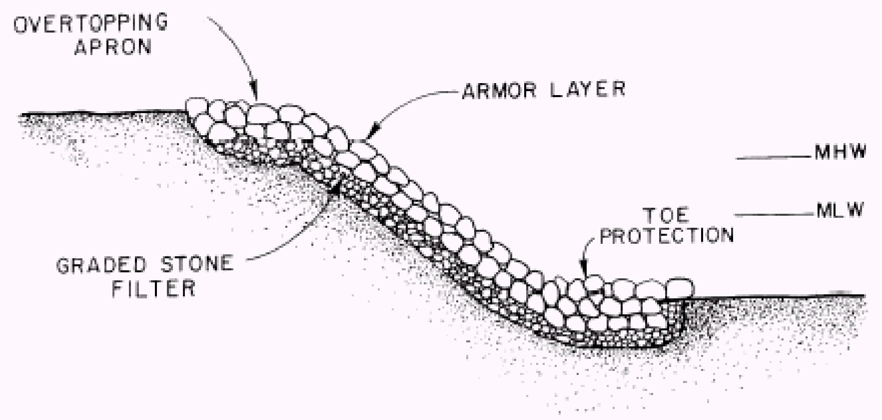

Figure 2.2: Typical Revetment Section

Like bulkheads, revetments protect only the land immediately behind them and provide no protection to adjacent areas. Erosion may continue on adjacent shores and may be accelerated near the revetment by wave reflection from the structure, although not as seriously as with vertical-faced bulkheads. Also, a downdrift shore may experience increased erosion if formerly supplied with material eroded from the now protected area. If a beach is to be retained adjacent to a revetment, additional structures such as groins or breakwaters may be required. Of the revetment's three components, the primary one, which determines the characteristics of the other two, is the armor layer, which must be stable against movement by waves. The second component, the underlying filter layer, supports the armor against settlement, allows groundwater drainage through the structure, and prevents the soil beneath from being washed through the armor by waves or groundwater seepage. The third component, toe protection, prevents settlement or removal of the revetment's seaward edge.

Overtopping by green water (not white spray) which may erode the top of the revetment can be limited by a structure height greater than the expected runup height, or by protecting the land at the top of the revetment with an overtopping apron. Flanking, a potential problem with revetments, can be prevented by tying each end into adjacent shore protection structures or the existing bank. As the bank retreats, however, the ends must periodically be extended to maintain contact.

The armor layer of a revetment maintains its position under wave action either through the weight of, or interlocking between, the individual units. Revetments are either flexible, semi-rigid, or rigid. Flexible armor retains its protective qualities even with severe distortion, such as when the underlying soil settles or scour causes the toe of the revetment to sink. Quarrystone, riprap, and gabions are examples of flexible armor. A semi-rigid armor layer, such as interlocking concrete blocks, can tolerate minor distortion, but the blocks may be displaced if moved too far to remain locked to surrounding units. Once one unit is completely displaced, such revetments have little reserve strength and generally continue to lose units (unravel) until complete failure occurs. Rigid structures may be damaged and fail completely if subjected to differential settlement or loss of support by underlying soil. Grout-filled mattresses of synthetic fabric and reinforced concrete slabs are examples of rigid structures.

2.2.1 Stone Revetments

2.2.1.1 Rubble Revetments

Rubble revetments are constructed of one or more layers of stone or concrete pieces derived from the demolition of sidewalks, streets, and buildings (Figures 2.3). Stone revetments are constructed of either two layers of uniform-sized pieces (quarry stone) or a gradation of sizes between upper or lower limits (riprap). Riprap revetments are somewhat more difficult to design and inspect because of the required close control of allowable gradations and their tendency to be less stable under large waves. They are, however, acceptable for the majority of low cost shore protection applications. In either case**,** stone revetments are time tested, highly durable, and often the most economical where stone is locally available.

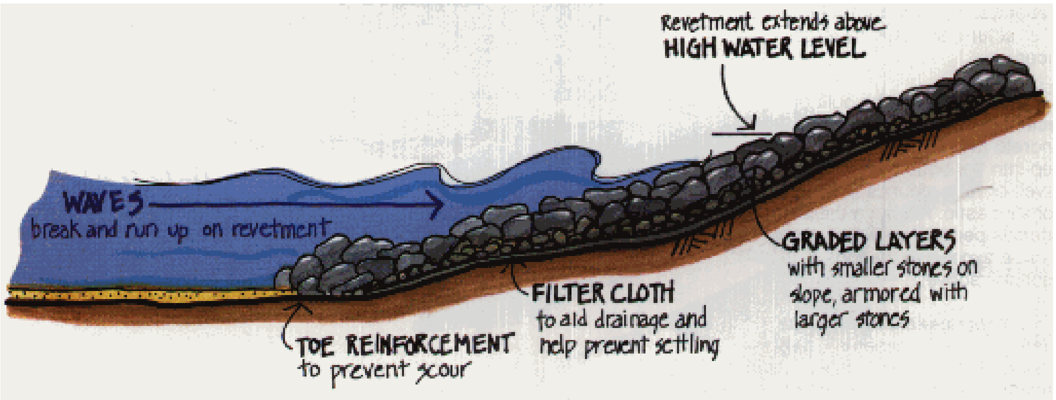

Figure 2.3: Typical section of stone revetment

The primary advantage of a rubble revetment is its flexibility, which allows it to settle into underlying soil or experience minor damage and still continue to function. Because of its rough surface, a rubble revetment experiences less wave runup and overtopping than a smooth-faced structure. The primary disadvantage is that placement of the stone or concrete armor material generally requires heavy equipment.

Prior to construction, the existing ground should be stabilized by grading to an appropriate slope. In most cases, the steepest recommended slope would be 1 vertical on 2 horizontal (1on 2). Fill material should be added as needed to achieve uniformity, but it should be free of large stones and firmly compacted before revetment construction proceeds. Properly sized filter layers should be included to prevent the loss of slope material through voids in the revetment stone. When using filter cloth, an intermediate layer of smaller stone below the armor stone may help distribute the load and prevent rupture of the cloth. The revetment toe should be located about one design wave height (but at least three feet) below the existing bottom to prevent undercutting. In lieu of deep burial, a substantial sacrificial berm of additional rubble (with filtering) should be provided at the toe.

2.2.1.2 Quarrystone

Stone revetments, a proven method of shoreline protection, are durable and can be relatively inexpensive where there is a local source of suitable armor stone. Such stone should be clean, hard, dense, durable, and free of cracks and cleavages. If graded stone filter material is used, it generally will be much finer than the armor stone. An intermediate layer of stone between the armor and filter, one-tenth as heavy as the armor units, may provide the necessary transition to the filter material.

2.2.1.3 Concrete Revetments

A concrete rubble revetment utilizes a waste product otherwise difficult to dispose of in an environmentally acceptable manner. The concrete should have the strength to resist abrasion by water-borne debris and ice pressure. In addition, all protruding reinforcing bars should be burned off prior to placement. Numerous concrete rubble revetments have failed in the past, usually because of neglect of filter requirements.

2.2.1.4 Concrete Block Revetments

Concrete blocks, many of them patented, have various intermeshing or interlocking features (Figure 2.4), and the advantage of a neat, uniform appearance. Many units are light enough to be installed by hand once the slope has been prepared. Their disadvantage is that interlocking between units must be maintained. Once one block is lost, other units can dislodge, and complete failure may result. A good, stable foundation is required since settlement of the toe or subgrade can cause displacement of units and ultimate failure. Also, some concrete bl6ck revetments have smooth faces that can lead to significantly higher wave runup and overtopping.

For maximum effectiveness, concrete block systems should only be placed on a stable slope with the toe buried at least three feet below the existing ground line. Fill materials beneath the revetment should be uniform and well compacted and an adequate filter system, preferably with a properly sized woven filter cloth, should be provided. All concrete must be high quality; standard building blocks will probably deteriorate too quickly. Finally, blocks should not be used where they may be stolen or damaged by waveborne cobbles, ice, or debris.

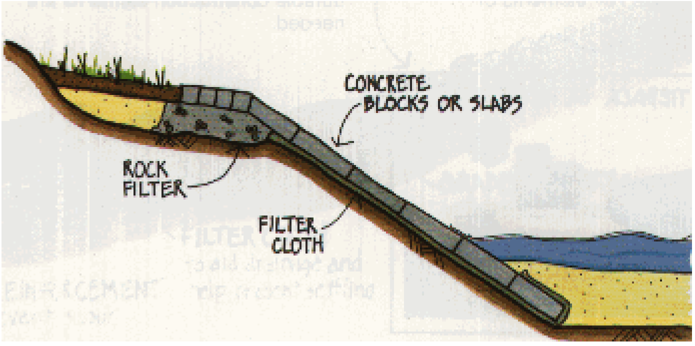

Figure 2.4: Typical section of concrete block revetment

2.2.1.5 Concrete Masonry Blocks

Standard construction masonry blocks should be hand-placed on a filter cloth with their long axes perpendicular to the shoreline and the hollows vertical. Their general availability is a primary advantage, but their wide use also makes them susceptible to theft. They form a deep, tightly fitting section which is stable provided the toe and flanks are adequately protected. Their primary disadvantage is that standard concrete for building construction is not sufficiently durable to provide more than a few years service in a marine environment.

2.2.2 Stacked Bag or Mat Revetments

Several manufacturers produce bags and mats, in various sizes and fabrics that are commonly filled with either sand or lean concrete for use in revetments. While no special equipment is required to fill bags with sand, a mixer and possibly a pump are needed for concrete-filled units. Bags should be filled and stacked against a prepared slope with their long axes parallel to the shoreline and joints offset as in brick work (Figure 2.5). Grout-filled bags can be further stabilized with steel rods driven down through the bags.

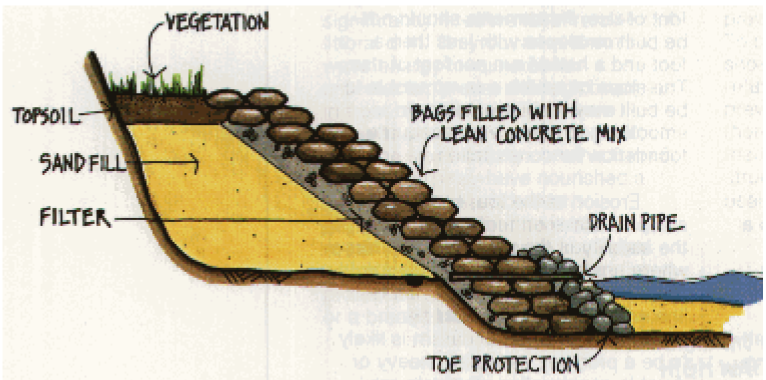

Figure 2.5: Typical section stacked bag revetment

The advantage of a bag revetment is its moderate cost. Sand filled bags are relatively flexible and can be repaired if some are dislodged. They are particularly suited to temporary emergency protection measures. Among their disadvantages are limitation to low energy areas, a relatively short service life compared to other revetments, and their generally unattractive appearance. Since concrete-filled structures are rigid, any movement or distortion from differential settlement of the subgrade can cause a major failure that would be hard to repair. Sand-filled bags are highly susceptible to damage and possible failure from vandalism, impact by water-borne debris, and deterioration of material and seams by sunlight. The smooth, rounded contours of bags also present an interlocking problem and they should be kept flatter and underfilled for stability.

Bags should form a large mass of pillow-like concrete sections with regularly spaced filter meshes for the passage of water. Bags or mattresses should only be placed on a stable slope. While a stacked bag revetment can be placed on a steeper slope than a mattress, it should not exceed 1 vertical on 1.5 horizontal. Fill materials beneath the revetment should be uniform and well compacted and an adequate filter system should be provided. Some form of toe protection is usually required, or the toe should be buried well below the anticipated scour depth. A stacked bag revetment should be at least 2 bags thick, preferably outside layer concrete-filled, but the inner layer sand-filled. When sand is used as filler material, the fabric and its seams must be non-degradable (ultraviolet resistant). However, where vandalism or water-borne debris is likely, only concrete-filled units should be used. As with concrete block revetments, the structure’s integrity depends on the stability of the individual units. Once units are lost or damaged, or settle unevenly, the structure loses its strength.

2.2.3 Riprap Revetments

Riprap revetments are a very effective and popular method of controlling streambank erosion. A revetment is a facing of stone or other armoring material to protect a streambank or shoreline. A riprap revetment consists of layered, various-sized rocks placed on a sloping bank (Figure 2.6). The most commonly used material for riprap is broken limestone, dolomite or quartzite. The type of stone used usually determined by what is locally available. The variance in size and the rough angular surfaces of the rock allow the revetment to absorb the impact of the flowing water instead deflecting the flow which could cause erosion to an adjacent streambank area. The rough angular surfaces of the broken rocks also allows to fit together to form a dense layer of protection over the eroding bank.

Stones that appear to have smooth and rounded surface should be avoided if possible. The surface of these stones does not allow the rocks to interlock which decreases resistance to movement. Broken asphalt should not be used because it has a low density and contains toxic chemicals which can leach out into the water.

Some of the advantages of riprap as an erosion treatment are that it is designed for high velocities and provides high degree of protection. Riprap is also relatively ease of installation and needs a minimal maintenance. In addition; riprap revetment system provides immediate long-term protection.

Riprap revetment also has it own disadvantages. If materially used is not locally and readily available and easily transported to the site, costs can be prohibitive. Furthermore, it may pose hazard to people who must access the revetment and aesthetically pleasing to some people.

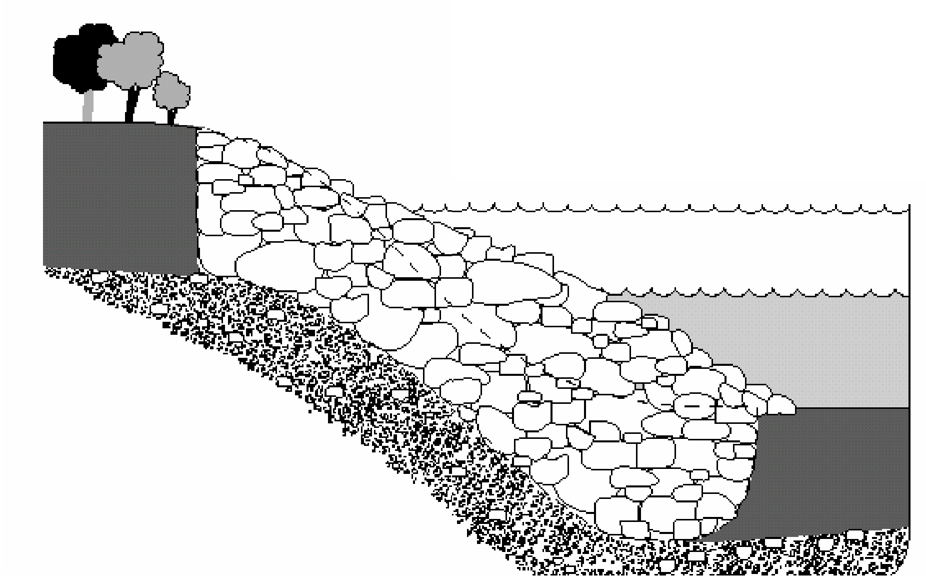

Figure 2.6: Cross section of a riprap revetment

2.3 GABIONS

Gabions are rectangular baskets or mattresses made of galvanized, and sometimes PVC-coated, steel wire in a hexagonal mesh (Figure 2.7). Subdivided into cells of approximately equal size, standard gabion baskets are 3 feet wide and are available in lengths of 6, 9, and 12 feet and heights (thicknesses) of 1, 1.5, and 3 feet. Mattresses are either 9 or 12 inches thick. At the job site, the baskets are unfolded and assembled by lacing the edges together with steel wire. The individual baskets are then wired together and filled with 4- to 8-indh diameter stones. The use of interior liners or sand bags for small size material is not recommended. The lids are finally closed and laced to the baskets, forming a large, heavy mass.

The chief advantage of a gabion revetment is that construction may be accomplished without heavy equipment. The structure is flexible and maintains functional integrity even if the foundation settles. Gabions can be repaired by opening the baskets, refilling them, and then wiring them shut again. Depending on the local supply of stone, a gabion revetment can be a low cost option.

The disadvantage of a gabion structure is that the baskets may open under heavy wave action. They should not be used where action by water-borne debris or cobbles is present. Also, since structural performance depends on the wire mesh, abrasion and damage to the PVC coating can lead to rapid corrosion of the wire and failure of the basket. For that reason, the baskets should be tightly packed and periodically refilled to minimize movement of the stone and subsequent damage to the wire. Rusted and broken wire baskets also pose a safety hazard where traffic across them is required. Gabion structures require periodic inspections so that repairs are made before serious damage occurs.

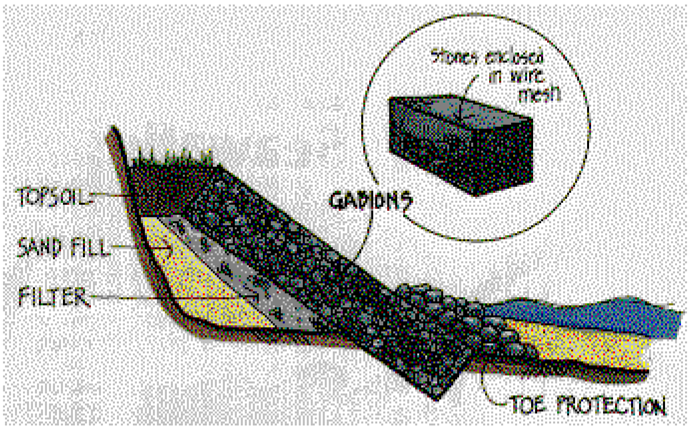

Figure 2.7: Typical section of gabions

2.4 GROINS

Groins are constructed perpendicular to shore and extend out into the water. Used singly or in groups known as groin fields, they trap sand or retard its longshore movement along beaches. Sand accumulates in fillets on the updrift side of the groin, and the shoreline rotates to align itself with the crests of incoming waves. As the adjustment proceeds, the angle between the shoreline and the waves decreases and with it, the longshore transport rate. Sand fillets act as protective barriers, which waves can attack and erode without damaging the previously unprotected upland areas. A groin, without a sand fillet, cannot protect a shoreline from direct wave attack. A prime consideration with groin system design is evaluation of the net direction and amount of longshore sediment transport. Successful performance requires an adequate net longshore transport rate to form an updrift fillet. If the gross transport rate is high but nearly equally divided in both directions (small net transport), groins do not generally function well or successfully form large updrift fillets.

When first built, the sand trapped on a groin's updrift side is no longer available to replenish downdrift beaches, resulting in erosion. When a groin fills to capacity, material passes around or over it to the downdrift shore, but at a slower rate than before the groin was built. If downdrift erosion is unacceptable (it usually is), an alternative is to build more than one groin and fill the area between with sand (Figure 2.8 and 2.9). This will minimize downdrift damages and limit scour at the groin's shoreward end.

Groins are generally effective only when littoral materials are coarser than fine sand. Silts and clays tend to move in suspension and are not retained.

Figure 2.9: Groins

Figure 2.9: Fill the area between groins with sand

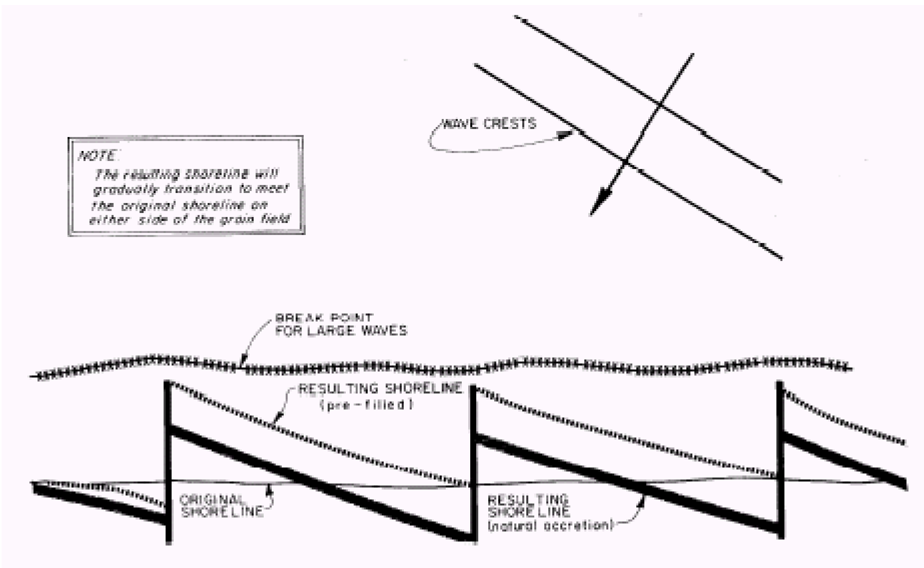

Figure 2.10: Effects of Groins

Important design considerations for groins include their height, length, and spacing (for a groin field). Height determines how much sand can pass over the structure; groins can be built either high or low with respect to the existing beach profile. Low groins, which essentially follow a foot or two above the natural beach profile, are widely used because they stabilize the beach but do not trap excessive amounts of sand and cause downdrift damages. High groins effectively block the supply of sand to downdrift beaches provided sand doesn't pass through them.

A groin's length must be sufficient to create the desired beach profile while allowing adequate passage of sand around its outer end. If a groin extends seaward past where waves break (breaker zone), sediment moving around the structure may be forced too far offshore to be returned to the downdrift beach. Therefore, the groin should not extend past the breaker zone, but it can be shorter provided that it traps a sufficient quantity of sand. All groins should be extended sufficiently landward to prevent their detachment from shore (flanking) if severe erosion occurs.

The correct spacing of groins depends on their length and the desired final shoreline shape. If groins are too far apart, excessive erosion can occur between them. If spaced too closely, they may not function properly, which is particularly critical for high, long groins where sand can only pass around the ends in a curved path back to the beach. If the groins are too close together, the sand is unable to reach the shore again before being forced seaward by the next downdrift groin. A common rule is to provide spacing equal to two or three times the groin length, measured from the water's edge.

Structurally, a groin must resist wave action, currents, the impact of floating debris, and earth pressures created by the difference in sand levels on the two sides. As other structures, a groin must also resist the scour created by waves breaking on the structure and by currents adjacent to it.

2.5 BEACH FILLS

Beach fills are quantities of sand placed on the shoreline by mechanical means, such as dredging and pumping from offshore deposits, or overland hauling and dumping by trucks. The resulting beach provides some protection to the area behind it and also serves as a valuable recreational resource.

The beach fill functions as an eroding buffer zone. As large waves strike it, sand is carried offshore and deposited in a bar. As the bar grows, it causes these large incoming waves to break farther offshore. The useful life of a fill, which depends on how quickly it erodes, can be completely eliminated in a short period of time by a rapid succession of severe storms. The owner must expect, therefore, to periodically add more fill as erosion continues. Beach fills generally have a relatively low initial cost but a regular maintenance cost of adding new fill (periodic renourishment).

The rate at which new fill must be added depends on the relative coarseness of the fill material in relation to the native beach material. Ideally, fill and native beach materials should be perfectly matched, but this is virtually impossible. Generally speaking, if the fill material is coarser than the native material, the fill will erode more slowly and if it is finer, it will erode more quickly.

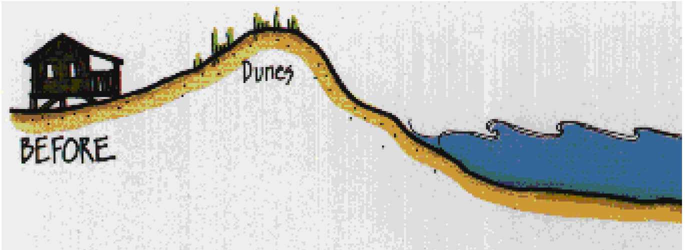

Figure 2.11: Shoreline before fill

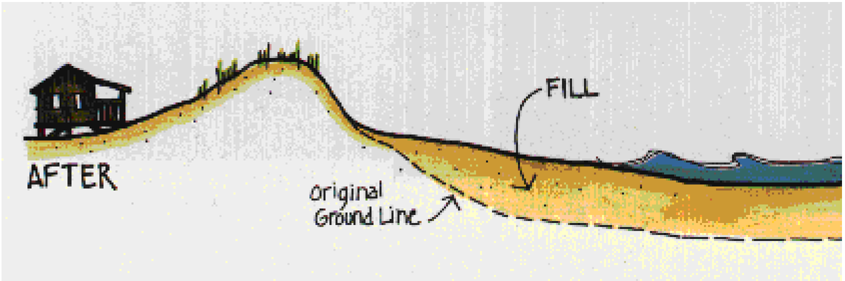

Figure 2.12: Shoreline after fill

The final factor is the beach slope, which should parallel the existing profile and slope on the theory that the existing beach is in equilibrium with the wave forces, and the new beach will eventually assume a similar shape. Equipment can shape the beach fill profile as it is placed, or the fill can be reshaped by waves. The final equilibrium slope depends on the relative coarseness of the fill material because coarser sand results in a steeper beach slope.

If fill is placed over a short length of shoreline, it creates a projection that is subjected to increased wave action. Therefore, it is generally preferable to make the transition to the existing shoreline over a longer distance, which may require cooperation from other landowners. If this is impractical, protective structures, such as groins, may be required to retain the fill.

2.6 VEGETATION

A planting program to establish desired species of vegetation is an inexpensive approach to shoreline protection. Depending on where stabilization is desired, species from two general groups should be selected to insure adequate growth.

Found on parts of shorelines, flooded periodically by brackish water, species of grasses, sedges and rushes occur in marshes of moderate to low energy shorelines. Once extensive and widely distributed, marsh areas were viewed in the past as useless and were subjected to filling and diking. However their destruction has lessened as their importance in the ecosystem and to shoreline protection has been realized.

Upland species (shrubs and trees but particularly grasses) are especially adapted to the low-nutrient, low-moisture environment of the higher beach elevations, where they are subject to abrasion by wind-blown sand particles. Used to trap sand and stabilize the beach, upland vegetation also improves the beauty of a shoreline, prevents erosion by intercepting raindrops, diminishes the velocity of overland flow, increases the soils infiltration rate, and provides a habitat for wildlife.

Even though vegetation provides significant help in stabilizing slopes and preventing erosion, vegetation alone cannot prevent erosion from heavy wave action, nor prevent movement of shoreline bluffs activated by groundwater action. In these instances, structural devices augmented with vegetation are recommended.

The effectiveness of vegetation is also limited by characteristics of the site. For instance, the site requirements which determine the effectiveness of a tidal marsh planting include elevation and tidal regime, which determine the degree, duration, and timing of plant submergence; slope of the site; exposure to wave action; type of soil; salinity regime; and oxygen-aeration times. Plants which are specially adapted for higher beach elevations must tolerate rapid sand accumulation, flooding, salt spray, abrasion by wind-borne sand particles, wind and water erosion, wide temperature fluctuations, drought and low nutrient. Appropriate species also vary with geographical location, climate, and distance from the water (vegetative zone).

2.7 INFILTRATION AND DRAINAGE CONTROLS

Infiltration and drainage controls are often needed for stability along high bluff shorelines. Although many factors lead to slope stability problems, groundwater is one of the most important. The majority of slope failures and landslides occur during or after periods of heavy rainfall or increased groundwater elevations. Infiltration controls prevent water from entering the ground, while drainage controls remove water already present in the soil or on the surface.

Since water entering surface cracks can lead to further instability, these should be filled with compacted soil (preferably clay) as they develop. Surface runoff should also be diverted from critical areas of the bluff by either drainage ditches or swales.

The treatment of subsurface drainage problems is complex. Where such problems exist, a geotechnical engineer should be consulted.

2.8 SLOPE FLATTENING

A bluff slope may be flattened to enhance its stability when adequate room exists at the top, and it does not interfere with the desired land use. Freshly excavated slopes should be planted to prevent erosion due to surface runoff. It may also be necessary to build a revetment or bulkhead at the toe of the slope to protect against wave action.

2.9 PERCHED BEACH

Perched beaches (Figure 2.13), which combine a low breakwater or sill and a fill, are beaches elevated (perched) above the normal level. They are suitable where offshore slopes are gradual enough to permit sill construction in reasonably shallow water at a distance from shore. The perched beach provides a broad buffer zone against wave action, while offering a potentially excellent recreational site.

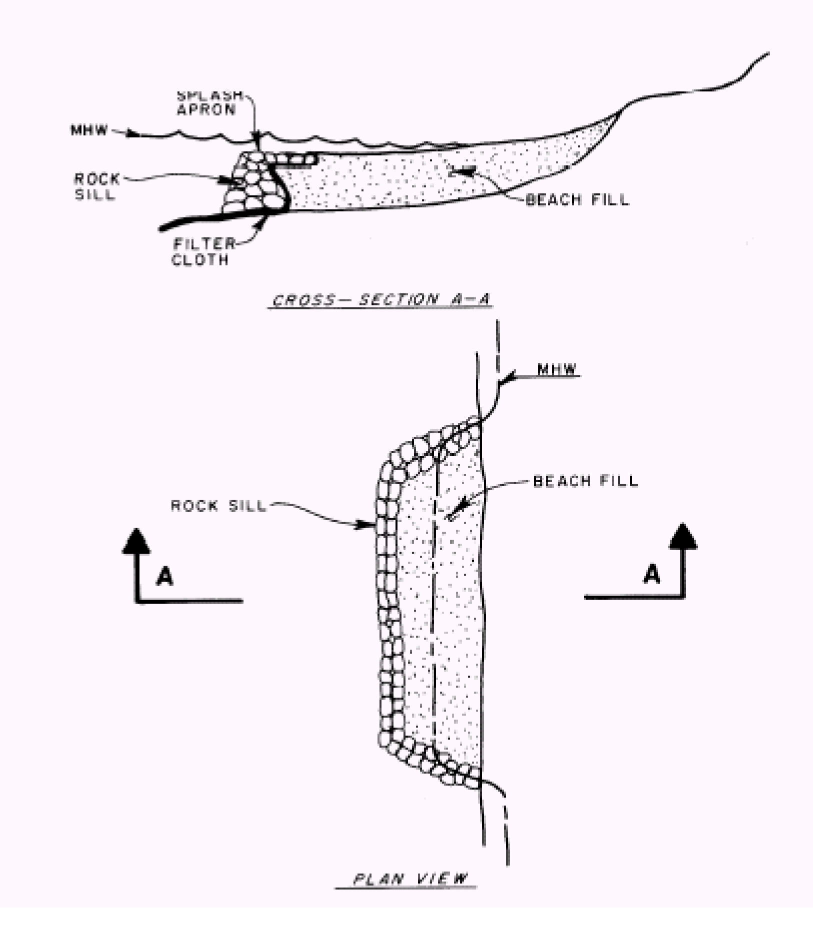

Figure 2.13: Perched (elevated) beach

Perched beach sills can be built using most of the materials described for fixed breakwaters. They must be made sand-tight to retain the beach fill, however. Proper filtering should be provided beneath and behind the sill to prevent settlement and loss of the retained fill.

This alternative provides broad buffer against wave action while offering a potentially excellent recreational site. The sill can be constructed of various materials, but it must be impermeable to the passage of the retained beach sand by using, for instance, a filter cloth behind and beneath the structure. The cloth prevents the fill form escaping through any large voids in the sill and also stabilizes the structure against settlement. While a graded stone core could also be used in rock sill in place of filter cloth, the limited height of such sill generally precludes use of multi-layered structures of this kind. This figure also shows a splash apron which is provided to prevent scour and erosion of the beach fill from overtopping waves.

2.10 STRUCTURES AND FILLS

In addition to perched beaches, fills can also be incorporated in groin systems and with breakwaters. In fact, auxiliary fills are almost mandatory in most cases, because if such structures fill by natural accretion, serious erosion problems almost surely occur at downdrift.

2.11 STRUCTURES AND VEGETATIONS

While vegetation is one means of controlling shoreline erosion, its most serious deficiency is its restriction to areas of limited fetch because it cannot establish itself in heavy wave environments. By placing it in the shelter of a structure such as a breakwater, however, vegetation can be used in areas experiencing considerably heavier wave activity. The use of a temporary structure is particularly appealing because it protects the plants while they become established and it can be removed when the plant mature.

2.12 CEOTEXTILES IN COASTAL PROTECTION

Geotextiles are defined as permeable textiles used in conjunctions with soils or rocks, as integral part of a man-made project. The traditional method of controlling erosion is to shield the soil particles from the moving water with a flexible protective structure. There are many different forms of structure. There are many different forms of structure that can be utilized such as rip-rap (broken rocks) or heavy armour stones, concrete blocks, articulated concrete mattresses and gabion mattresses.

The weight of these protective structures also helps the soil particles in the bank to resist the effect of seepage into the waterway. However, the protective material is usually required to be permeable, in order to prevent the build up of hydrostatic pressure. The drainage openings, which can be very large in the case riprap, would expose the underlying soil to erosion, if other protective measures were not taken. A graded aggregate filter is therefore traditionally used between the natural soil and the protective material could make use of as many as five layers of differently graded aggregates necessary in such a granular filter. It can prove difficult and expensive to install a multi layer erosion protection and filtration system, especially if this has to be placed under moving water.

2.12.1 Geotextile filters

The use of geotextile filter can simplify construction of the erosion control measures, as illustrated in Figure 2.14, where it is replaces several layers of granular filter beneath riprap armour. A geotextile filter can also be used in a similar manner, beneath gabion mattresses or articulated concrete mattresses. The geotextile is placed in contact with and down the gradient of soil to be drained. Water and any particles suspended in the water which are smaller than a given size flow through the geotextile. Those soil particles larger than the size are stopped and prevented from being carried away. The geotextile should be sized to prevent soil particle movement. The geotextile substitute for and serve the same function as the traditional granular filter

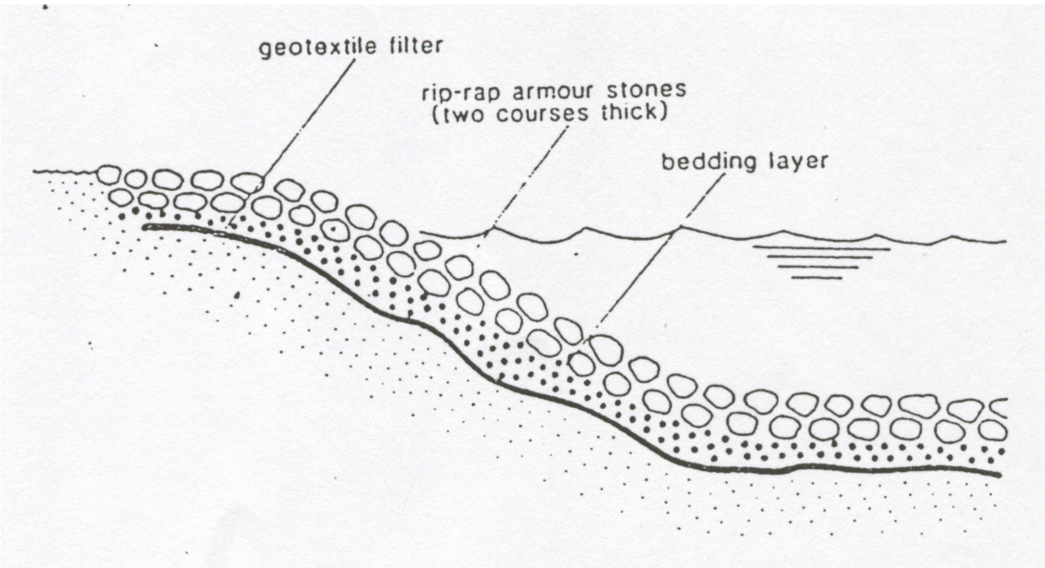

Figure 2.14: Geotextile filter replacing a multi-layer granular filter in bank protection scheme

2.12.2 Riprap Protection

Figure 2.15 shows the most widely used system of protection, which is bonded riprap laid over a layer of thick composite geotextile. This is intended for use on the banks of waterways which have a side slope of between 1 in 4 and 1 in 3. The geotextile filter is laid on the prepared soil surface and is covered by the bonded riprap, without the use of any intermediate blanket layer

Bonded riprap is less susceptible to wave induced movement than loose riprap, and smaller sized stones may therefore be utilized. This usually gives bonded riprap an economic advantage over loose riprap. Small quantities of either cement or hot bitumen are used as the bonding compound and are mixed with riprap stones before placing. When cement is used, care must be taken not to use too high a water content or too large a quantity of cement as this could smear the surface of the geotextile.

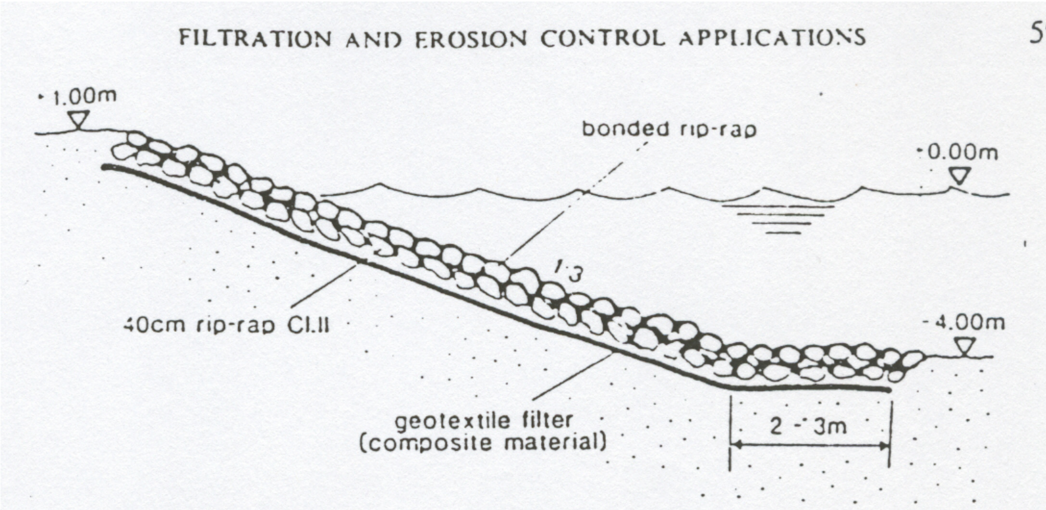

Figure 2.15: The bonded riprap protection systems for canal sides

In some soils, there is a tendency for the particle to become loosened in the wave zone, even when protected by the geotextile and the riprap. Without precautions, the loosened soil particles may move down the slope, to produce a bulge and a depression in the protection works, as shown in Figure 2.16. This problem can be avoided by using a composite geotextile in which a thick rough geotextile is bonded to a thinner geotextile filter layer.

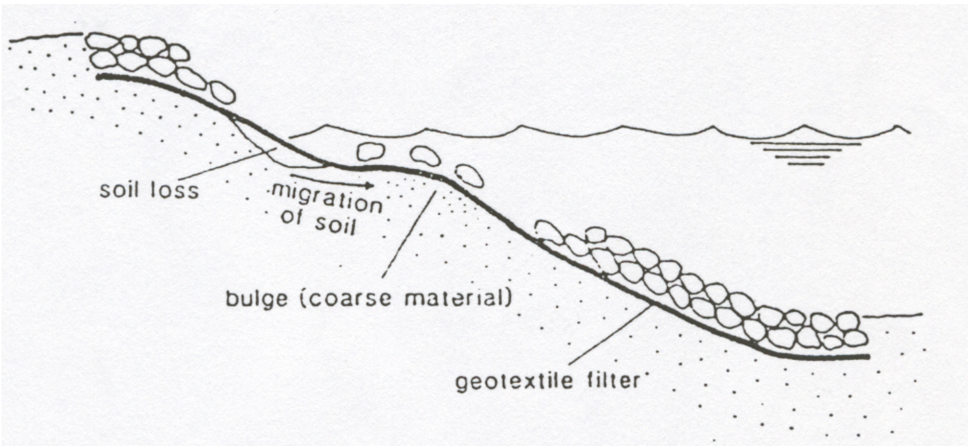

Figure 2.16: Bulging due to soil migration beneath the geotextile

The weakest part in bank protection design is probably the toe of the revetment. If this is subjected to scour it will undermined and will initially bridge over the scour zone, due to the bonding material in the riprap. Without contact between riprap and the soil, the erosion rate increases until riprap collapses into the void. The toe of riprap is then so damaged that pieces break away in clumps. When the erosion protection is installed in dry conditions, the simplest form enhanced toe protection is to extend the riprap revetment into the bed of waterway to a depth exceeding the anticipated scour, as shown in Figure 2.17. An alternative method is to fold over and sew the end of the geotextile sheet (Figure 2.18), to produce a cylinder about 0.4m high and 0.6m wide, which is filled with clay. The riprap on the canal bed is left unbounded, ensuring that this section is fully flexible and is able to deform with the onset of scour, thereby maintaining the protection and limiting the rate of damage.

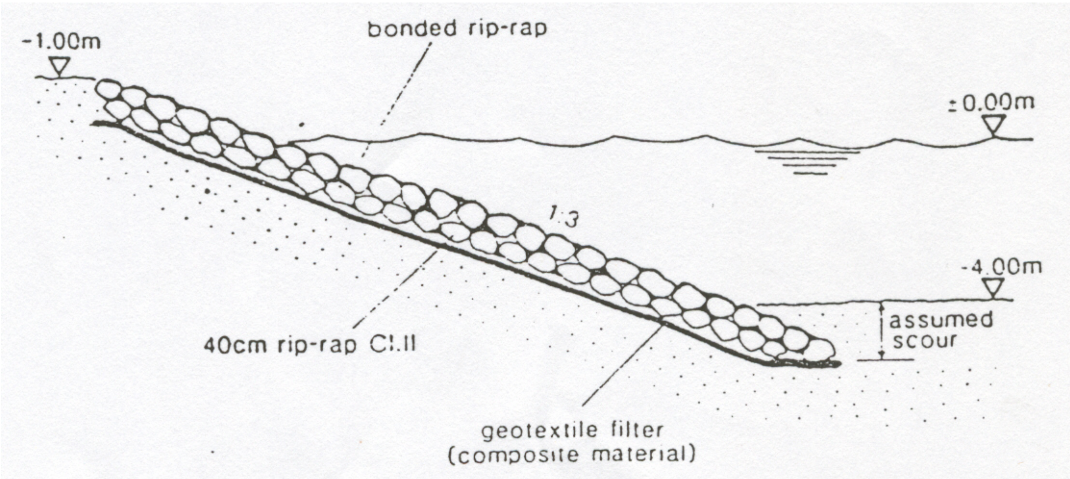

Figure 2.17: Extended toe to deal with scour

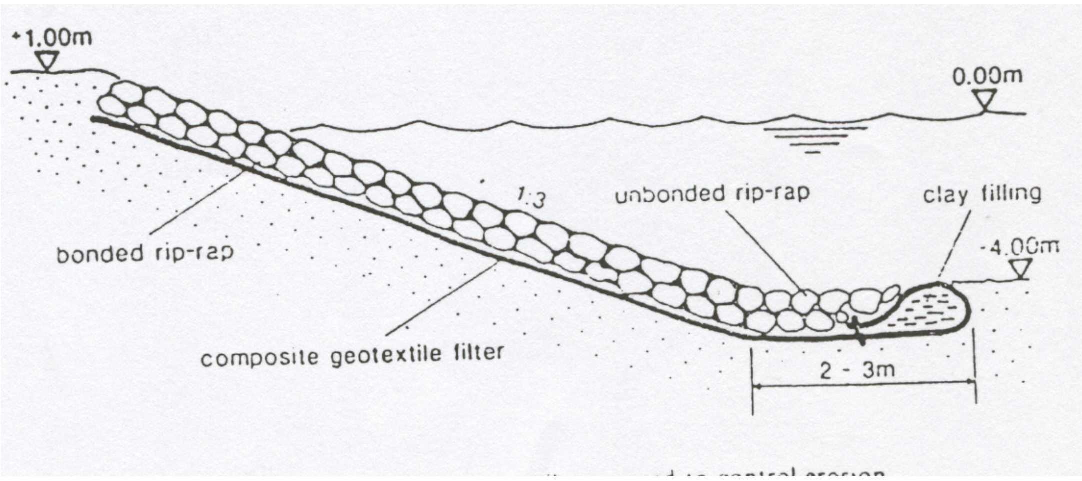

Figure 2.18: Clay filled geotextile toe used to control erosion

2.12.3 Concrete Block Protection

Although it is possible to use loose, closely fitting, concrete blocks for bank protection, these do not have any significant advantages over riprap protection systems.

However, if some means of linking the concrete blocks together is utilized, than this additional integrity enables a lighter weight of riprap protection. These flexibly jointed block systems are collectively known as articulated block revetments. The two most common forms of linking the concrete blocks together are; cables thread through the blocks (Figure 2.19) and individual connections to a continuous geotextile sheet (Figure 2.20).

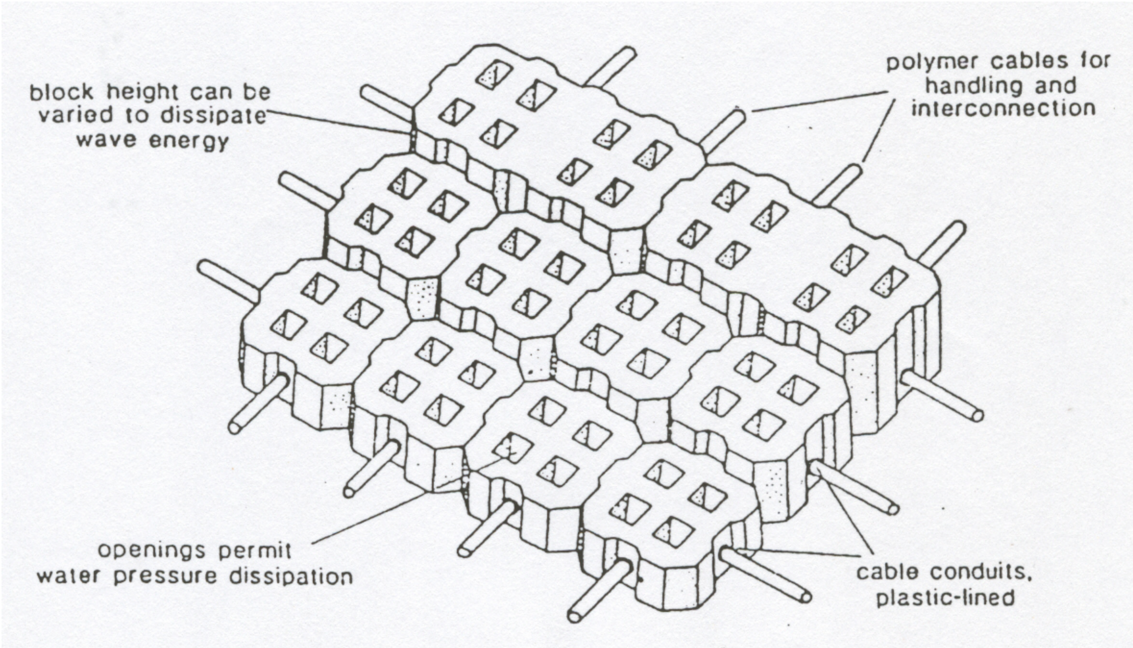

Figure 2.19: An element of an articulated block revetment system

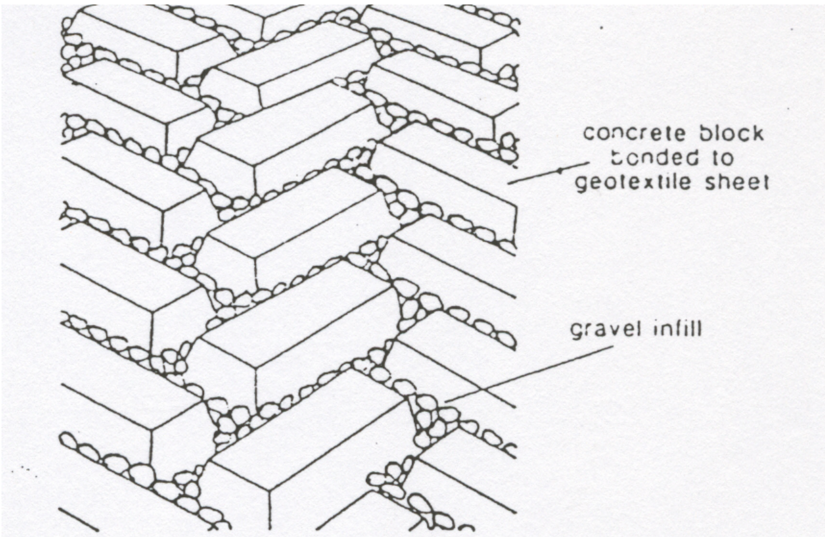

Figure 2.20: Prefabricated sheet of concrete blocks bonded to a geotextile

2.14.4 Gabion Mattress Protection

The most common form of gabion used in bank protection is a rectangular gabion mattress. The outer mesh of gabion is usually formed from geotextile nets or steel mesh protected by zinc or PVC coating.

2.13 SELECTION AMONG AVAILABLE OPTIONS

2.13.1 Shoreform Compatibility

Certain approaches are better suited to particular shoreline configurations than others. It is important to choose a method appropriate to the dominant shoreform at the site.

2.13.1.1Bluff Shorelines

The *no action* alternative can be appropriate since it does not disrupt the natural shoreline processes and requires no investment for protective structures. The property, however, may eventually be totally destroyed by erosion. While relocation also does not disrupt shoreline processes and permanently eliminates any threat to buildings if done properly, it also requires special equipment and skills and can cost as much as or more than a protective structure.

Bulkheads are ideally suited either for full-height retention of low bluffs or as toe protection for high bluffs. They can be constructed of readily available materials, are easily repaired if damaged, and are particularly useful with steep offshore slopes. They can, however, induce toe scour and loss of remaining beach material from the force of reflected waves. They also have high initial costs and some require special pile driving equipment which may have difficulty reaching the work site. Revetments are sometimes effective in bluff situations. Low bluffs that can be re-graded to a stable slope may be effectively protected by revetments.

Revetments can protect the toes of high bluffs, either alone or in conjunction with another device. *Breakwaters* reduce wave energy reaching the bluff but do not provide positive protection to the toe. They may build or maintain a sand beach, which provides some protection against normal waves but would be ineffective against storm waves. They require an adequate sand supply and gentle offshore slopes. *Groins* provide only a buffer by building or holding a beach. Since they require a natural sand supply, they would not work in a clay or silt bluff area unless sand were imported. *Beach fills* only dissipate normal wave action and would not be effective during severe storms. Vegetation provides little protection until well established and, even then, does not positively protect against large storm waves.

Drainage controls are mandatory if groundwater and infiltration adversely affect slope stability. They provide no toe protection against wave action and can be expensive. Also, they are difficult to properly design, and may require the efforts of a qualified professional engineer. *Slope flattening* provides a permanent solution for slope stability problems but does not provide protection against continued wave action. It also requires adequate setback room at the top of the bluff for the slope. *Perched beaches* would protect the bluff from normal wave action but would not provide positive toe protection during storms. A *combination approach* can be the best solution. For instance, drainage controls should be used as needed, possibly with slope flattening as well. Toe protection could be provided with a revetment along with a fronting sand beach for additional protection (provided offshore slopes are mild). Vegetation planted on the re-graded slope would prevent erosion from runoff and also help to stabilize a beach fill.

2.13.1.2Sand Beaches or Low Plains

The no action and relocation alternatives are applicable. *Bulkheads* are generally inappropriate unless an elevated feature is needed, such as a promenade or parking lot. Vertical bulkheads induce toe scour and wave reflections, and could cause a total loss of beach. Revetments are suited for protecting features directly behind the beach since they absorb wave energy and are flexible if settlement occurs. However, they have an adverse aesthetic effect on the beach and can limit use or access to the shore. Their use by a single landowner is generally a problem because they are subject to flanking. Breakwaters are also well suited because they trap and hold sand moving both alongshore and on or offshore. However, they can cause extensive downdrift erosion damages and they are expensive to build.

Groins can effectively build beaches on their updrift sides but can also cause accelerated downdrift erosion. Their functional behavior is complex and difficult to predict. *Beach fill retain* the natural form and character of the beach and enhance its recreational potential. Local sources of suitable sand are not always available, however, and fills require periodic renourishment. Vegetation, effective in low wave energy situations, has low initial costs and enhances natural appearance. Unfortunately, foot and vehicular traffic damage plantings. Drain*age controls* and *slope flattening* are not applicable to beach shorelines. *Perched breaches a*re ideally suited as they increase the available beach area. *Combination methods* are often excellent, such as a perched beach that is further stabilized with vegetation.

2.13.2 Effects on Coastal Processes and Adjacent Properties

Table 2.1 lists the effects of various options on shoreline processes.

Table 2.1: The effects of various options on shoreline processes

Option	**Effect**

No Action	Eroding shoreline will continue to supply material for transport to adjacent shores.
Relocation	Eroding shoreline will continue to supply material for transport to adjacent shores.
Bulkheads	Protect eroding shorelines that may have been supplying material to downdrift areas, which may then experience accelerated erosion. The fronting beach may experience increased erosion due to wave reflections.
Revetments	Protect eroding shorelines that may have been supplying material to downdrift areas, which may then experience accelerated erosion.
Breakwaters	Diminished wave energy behind such structures induces deposition. If the amount of sediment accumulation is significant, the downdrift shore may experience accelerated erosion. If wave energy is significantly reduced, the area behind the breakwater may not have sufficient circulation to maintain water quality.
Groins	Impede longshore transport and induce sedimentation. The downdrift shoreline may experience accelerated erosion due to lack of material supply.
Beach Fills	Provide a new supply of material to the littoral transport system. Increased suspended sediment loads could shoal adjacent navigation channels.
Vegetation	If some material is retained that previously was transported alongshore, it is possible that the downdrift shoreline may experience minimal erosion damages.
Drainage Controls	No significant effects.
Slope Flattening	No significant effects.
Perched Beaches	See "Beach Fills".
Structures and Fills	By filling the structures to near capacity with sand, the longshore transport may pass around the structure and continue to supply the downdrift shore.
Structures and Vegetation	The vegetation help the structure more effectively retain sand, which could cause increased downdrift erosion.

2.13.3 Effects on Shoreline Uses

Table 2.2 lists the significant effects of the various options on shoreline uses.

Table 2.2: Significant effects of the various options on shoreline uses

Option	**Advantages**	**Disadvantages**

No Action	• No consistent positive or negative effects on shoreline uses.	
Relocation	• No consistent or negative effects on shoreline uses.	
Bulkheads	• When used for wharves, provide direct boat access to the shore. • Unless placed high on a beach, they may binder swimming, jogging, walking or fishing.	• Can limit access to beach so that stairs may be required • Tend to cause erosion of existing fronting beach and possibly adjacent shores.
Revetments	• Unless fronted by a sufficient beach width, they may binder swimming, jogging, walking or fishing.	• Certain designs (e.g., gabions or quarrystone) limit access to the beach so that stairs may be needed. • If the fronting beach is normally submerged at hight tide, partially submerged revetments may pose a hazard to swimmers.
Groins	• Rubble structures may provide a habitat for aquatic life. • Can provide access to deeper water for fishing • Trapped sand fillets increase the available • beach area • Do not obstruct access to and from the beach	• May limit travel along the beach • Low groins, when submerged, can be a hazard to boats if not appropriately marked • Rip currents may be induced along groins which may be hazardous to bathers.
Beach Fills	• Provide more beach area for swimming, jogging, walking and fishing • Do not restrict access to and from or along the beach.	• Increased turbidity during placement may cause temporary impairment of fishing.
Vegetation	• Provide a habitat for aquatic life • Creates an opportunity for nature study	• Restricts beach use because plantings cannot be subjected to traffic.
Drainage Control	• When a fronting beach already exists, the potential for swimming, boating, jogging, etc., is unaffected.	• No impairment of shoreline uses.
Slope Flattening	• When a fronting beach already exists, the potential for swimming, boating, jogging, etc., is unaffected. • Access to the beach is enhanced.	• Reduce the land available at the top of the slope.
Perched Beaches	• Provide more beach to be used for swimming, jogging, walking, and fishing. • Do not restrict access to and from or along the beach.	• The sudden drop off the end of the beach at the sill can pose a hazard to bathers. • If not properly marked when submerged, the sill can be a hazard to boaters.
Structures and Fills	• Provide more beach area for swimming, jogging, walking and fishing	• Increased turbidity during fill placement may cause temporary impairment of fishing.
Structures and Vegetation	• Marsh plants provide a habitat for aquatic life.	• Plantings may restrict beach use because they cannot be subjected to traffic.

2.13.4 Effects on the Environment

The environmental effects of low cost devices, because of their limited extent, will generally be minimal. Such impacts as may occur are summarized in Table 2.3.

Table 2.3: impacts of shore protection structures to environment

Option	Effect
No Action	No consistent positive or negative effects.
Relocation	No consistent positive or negative effects.
Bulkheads	Impacts will be almost nonexistent. Bulkheads that stabilize eroding clay bluffs may decrease turbidity and enhance water quality. Construction operations may temporarily increase suspended sediment loads.
Revetments	Same as for bulkheads. In addition, stone structures with submerged lower portions may provide an improved habitat for certain fin and shellfish species.
Breakwaters	Stone and similar materials will improve habitat to an even greater degree than revetments. Decreased wave action and current strengths behind breakwaters may inhibit circulation and exchange and could impair water quality.
Groins	Similar to breakwaters except that water circulation problems are not likely to be as troublesome.
Beach Fills	Increased quantity of sand will result in greater turbidity, especially during initial placement. Sand lost from the fill may deposit elsewhere such as in shellfish beds, etc.
Vegetation	Vegetation provides a superior habitat for many important species in the food web. Well-established stands of vegetation also filter the water and decrease the amounts of suspended sediment and pollutants. Marshes enhance the ecological value of almost any shoreline.
Drainage Controls	No significant impacts except where they stabilize clay bluffs. In those cases, turbidity will be decreased locally.
Slope Flattening	Same as "Drainage Controls".
Perched Beaches	Same as "Breakwaters" and "Beach Fills".
Structures and Fills	Same as "Beach Fills"; also depends on the structure.
Structures and Vegetation	Same as “Vegetation”; also depends on the structure.

2.13.5 Implications for Coastal Zone Management

As can be seen from the information already presented, the selection of proper alternatives for protecting shorelines requires trade-offs among many advantages and disadvantages. No single alternative will apply in every case and each has to be considered on its own merits.

Consistency in the planning or cost shore protection systems requires an encompassing set of guidelines or goals that should be established by each local jurisdiction. The desire is to satisfy a community's development plans without risking property or life, while simultaneously protecting its ecological resources. Each community has its own set of attitudes, social goals, and political styles which will determine the policies it develops.

The purpose of proper shoreline management is to look beyond each individual site to the whole community. Uncontrolled development may adversely affect the shoreline in a number of ways. Management policies should, therefore, be concerned with minimizing changes in patterns of drainage and runoff, preserving ecologically valuable areas such as dunes and wetlands, preserving natural protective forms such as dunes and beaches, avoiding adverse alternation of coastal configurations, protecting coastal waters from pollution, and restoring damaged areas to former conditions. These policies would be applied to the earlier identified shoreforms in relation to low cost shore protection as follows.

2.13.5.1Bluff Shorelines

Adverse uses of lands adjacent to the tops of banks or bluffs should be avoided. Clearing of trees and undergrowth, constructing buildings, or plowing could all destabilize existing slopes by increasing seepage and surface erosion or by adding extra weight (surcharge) which the bluff must supported. Surcharges, in particular, should be avoided. Changes in surface drainage patterns should be planned to divert the flow away from the bluff face.

Zoning regulations should be instituted to restrict development to areas landward of setback lines. These should be established based on projected shoreline recession amounts over a specified future time period. In fact, in localities threatened with erosion, these setback lines are required for endangered structures to qualify for insurance under the National Flood Insurance Program.

Activities should be discouraged that will alter or disturb the bluff face or toe. Stable bluffs should not be stripped of vegetation, nor should they be unnecessarily excavated, as this could lead to slides and slope failures. This does not eliminate slope flattening or drainage controls as alternatives, because these are used when the bluffs are inherently unstable and must be treated to restore stability. Plantings and other uses of vegetation should be encouraged on all excavated or natural slopes to increase stability and reduce erosion.

Toe protection should be provided in all cases where wave attack undermines the bluff. Any appropriate device outlined within this report, subject to other engineering, shoreline use, or environmental criteria, would be acceptable.

2.13.5.2Beach Shorelines

Activities should be discouraged that remove sand from the active beach zone, whether for fill at other areas, or for placement elsewhere on the beach profile. This would include dredging for beach fills, as a source of concrete aggregates, or as fill for bag structures.

New development should be located inland from the active beach to preclude the need for future shore protection. Setback lines should be established and observed. In most states, public domain is maintained as the area up to the mean high water line (MHHW on the west coast). In some states (e.g. Texas) public domain extends higher, to the point of permanent vegetation. In addition, local Governments in some areas have zoned additional setbacks of 30 feet or more from the MHW line, with the area designated for community recreational purposes.

Actions that adversely affect the littoral system should be discouraged. Accretion devices such as breakwaters and groins interfere with sand transport and may cause downdrift erosion. Sand permanently trapped behind such structures is also unavailable during transport reversals and could cause updrift erosion damages. Supplemental fill (preferably from inland sources) should be placed in the shadow zone of all such structures to minimize adjacent property damages.

Restoration of eroded beaches should be encouraged as part of any shore protection plan. Vegetation may be useful in some locations to further assist stability.

Excavation and removal of dunes should be discouraged because dunes serve as the natural front line of defense for the shore. Control should be exercised through local zoning ordinances or building codes that require special permits for excavation in dune areas. All new development should be located landward of dunes.

Existing vegetation, particularly on dunes, should be protected, primarily by restricting pedestrian or vehicular traffic. Special roads or walkways may be required in some cases.

Dunes should be restored and stabilized whenever possible as part of a comprehensive shore protection plan. Vegetation and snow fencing are principal means of accomplishing this.

2.13.5.3 Wetlands and Marshes

Alterations to the surface of marshes by excavating, filling, clearing, paving or grading should generally be prohibited. The value of marshes for shore protection and as ecological resources has been stressed. In cases of essential development, marsh areas that are destroyed may be replaced by newly developed marshes elsewhere as compensation and as a requirement for development. As an alternative, adjacent damaged marshes may be restored to their full function to replace those taken by development.

Diking or draining of marshes is generally harmful and should not be permitted. Permanent structures that would impair marsh functions should be discouraged. Placement of buildings on piles is often an acceptable alternative, as are elevated walkways, shelters, footbridges, and boat houses.

Discharge of pollutants should be restrained. Marshes serve a valuable water purification function but their ability to absorb pollutants is finite and limited.

Marshes should be restored to full function as part of any comprehensive shore protection plan.

2.14 GLOSSARY

Accretion - Accumulation of sand or other beach material at a point due to natural action of waves, currents and wind. A build-up of the beach.

Alongshore - Parallel to and near the shoreline; same as LONGSHORE.

Backhoe - Excavator similar to a power shovel except that the bucket faces the operator and is pulled toward him.

Bar - Fully or partly submerged mound of sand, gravel, or other unconsolidated material built on the bottom in shallow water by waves and currents.

Beach - Zone of sand or gravel extending from the low water line to a point landward where either the topography abruptly changes or permanent vegetation first appears.

Beach Fill - Sand or gravel placed on a beach by mechanical methods.

Beach, Perched - See PERCHED BEACH.

Bluff - High, steep bank at the water's edge. In common usage, a bank composed primarily of soil. See CLIFF.

Boulders - Large stones with diameters over 10 inches. Larger than COBBLES.

Breaker - A wave as it spills, plunges or collapses on a shore, natural obstruction, or man-made structure.

Breaker Zone - Area offshore where waves break.
Breaking Depth - Stillwater depth where waves break.

Breakwater - Structure aligned parallel to shore, sometimes shore connected, that provides protection from waves.

Bulkhead - A structure that retains or prevents sliding of land or protects the land from wave damage.

Clay - Extremely fine-grained soil with individual particles less than 0.00015 inches in diameter.

Cliff - High steep bank at the water's edge. In common usage, a bank composed primarily of rock. See BLUFF.

Cobbles - Rounded stones with diameters ranging from 3 to 10 inches. Cobbles are intermediate between GRAVEL and BOULDERS.

Crest - Upper edge or limit of a shore protection structure.

Cross Section - View of a structure or beach as if it were sliced by a vertical plane. The cross section should display structure, ground surface, and underlying material.

Culm - Single stem of grass.

Current - Flow of water in a given direction.

Current, Longshore - Current in the breaker zone moving essentially parallel to shore and usually caused by waves breaking at an angle to shore. Also called alongshore current.

Deep Water - Area where surf ace waves are not influenced by the bottom. Generally, a point where the depth is greater than one-half the surface wavelength.

Diffraction- Progressive reduction in wave height when a wave spreads into the shadow zone behind a barrier after the wave has passed its end.

Diurnal - Period or cycle lasting approximately one day. A diurnal tide has one high and one low in each cycle.

Downdrift - Direction of alongshore movement of littoral materials.

Dune - Hill, bank, bluff, ridge, or mound of loose, wind-blown material, usually sand.

Duration - Length of time the wind blows in nearly the same direction across a FETCH (generating area).
Ebb Tide - Part of the tidal cycle between high water and the next low. The falling tide.

Equilibrium - State of balance or equality of opposing forces.

Erosion - Wearing away of land by action of natural forces.

Fetch - Area where waves are generated by wind, which has steady direction and speed. Sometimes called FETCH LENGTH.

Fetch Length - Horizontal direction (in the wind direction) over which a wind generates waves. In sheltered waters, often the maximum distance that wind can blow across water.

Filter Cloth - Synthetic textile with openings for water to escape,
but which prevents passage of soil particles.

Flood Tide - Part of the tidal cycle between low water and the next high. The rising tide.

Glacial Till - Unstratified glacial drift consisting of unsorted clay, sand, gravel, and boulders, intermingled.

Longshore - Parallel to and near the shoreline: same as ALONGSHORE.

Longshore Transport Rate - Rate of transport of littoral material parallel to shore. Usually expressed in cubic yards per year.

Low Tide - Minimum elevation reached by each falling tide.

Low Water Datum (LWD) - The elevation of each of the Great Lakes to which are referenced the depths shown on navigation charts and the authorized depths of navigation projects.

Marsh - Area of soft, wet, or periodically inundated land, generally treeless, and usually characterized by grasses and other low growth.

Mean Higher High Water (MHHW) - Average height of the daily higher high waters over a 19-year period. Only the higher high water of each pair of high waters of a tidal day is included in the mean.

Mean High Water (MHW) - Average height of the daily high waters over a 19-year period. For semidiurnal or mixed tides, the two high waters of each tidal day are included in the mean. For diurnal tides, the single daily high water is used to compute the mean.

Mean Lower Low Water (MLLW) - Average height of the daily lower-low waters of a 19-year period. Only the lower low water of each pair of low waters of a tidal day is included in the mean. Long used as the datum for Pacific coast navigation charts, it is now gradually being adopted for use across the United States.

Mean Low Water (MLW) - Average height of the low waters over a 19-year period. For semidiurnal and mixed tides, the two low waters of each tidal day are included in the mean. For a diurnal tide, the one low water of each tidal day is used in the mean. Mean Low Water has been used as datum for many navigation charts published by the National Ocean Survey, but it is being phased out in favor of Mean Lower Low Water for all areas of the United States.

Mean Sea Level - Average height of the sea surface over a 19-year period. Not necessarily equal to MEAN TIDE LEVEL.

Mean Tide Level - Plane midway between MEAN HIGH WATER and MEAN LOW WATER. Not necessarily equal to MEAN SEA LEVEL. Also called half-tide level.

Mixed Tide - A tide in which there is a distinct difference in height between successive high and successive low waters. For mixed tides there are generally two high and two low waters each tidal day. Mixed tides may be described as intermediate between semidiurnal and diurnal tides.

Module - A structural component, a number of which are joined to make a whole.

Neap Tides - Tides with decreased ranges that occur when the moon is at first or last-quarter- ;4nl in opposition to each other. The neap range is smaller than the mean range for semidiurnal and mixed tides.

Nearshore - In beach terminology, an indefinite zone extending seaward from the shoreline well beyond the breaker zone

Nourishment - Process of replenishing a beach either naturally by longshore transport or artificially by delivery of materials dredged or excavated elsewhere.

Offshore - (1) (Noun) In beach terminology, comparatively flat zone of variable width extending from the breaker zone to the seaward edge of the Continental Shelf. (2) (Adjective) Direction seaward from the shore.

Overtopping - Passing of water over a structure from wave runup or surge action.

Peat - Residual product produced by partial decomposition of organic matter in marshes and bogs.

Peat Pot (vegetation) - Pot formed from compressed peat and filled either with soil or peat moss in which a plant or plants, grown from seed, are transplanted without being removed from the pot.

Perched Beach - Beach or fillet of sand retained above the otherwise normal profile level by a submerged dike or sill.

Permeable - Having openings large enough to permit free passage of appreciable quantities of (1) sand or (2) water.

Pile - Long, heavy section of timber, concrete or metal driven or jetted into the earth or seabed as support or protection.

Pile, Sheet - Pile with a generally slender, flat cross section driven into the ground or seabed and meshed or interlocked with like members to form a diaphragm, wall, or bulkhead.

Piling - Group of piles.

Plug - Core containing both plants and underlying soil, usually cut with a cylindrical coring device and transplanted to a hole cut by the same device.

Polyvinyl Chloride (PVC) - Plastic material (usually black) that forms a resilient coating suitable for protecting metal from corrosion.

Profile, Beach - Intersection of the ground surface with a vertical plane that may extend from the top of the dune line to the seaward limit of sand movement.

PVC - See POLYVINYL CHLORIDE.

Ravelling - Progressive deterioration of a revetment under wave action.

Refraction (of water waves) - (1) Process by which direction of a wave moving in shallow water at an angle to the contours is changed. Part of the wave advancing in shallower water moves more slowly than the part still advancing in deeper water, causing the wave crest to bend toward alignment with the underwater contours. (2) Bending of wave crests by currents.

Revetment - Facing of stone, concrete, etc., built to protect a scarp, embankment, or shore structure against erosion by waves or currents.

Rhizome - Underground stem or root stock. New shoots are usually produced from the tip of the rhizome.

Riprap - Layer, facing, or protective mound of stones randomly placed to prevent erosion, scour, or sloughing of a structure or embankment; also, the stone so used.

Rubble - (1) Loose, angular, waterworn stones along a beach. (2) Rough, irregular fragments of broken rock or concrete.

Runup - The rush of water up a structure or beach on breaking of a wave. Amount of runup is the
vertical height above stillwater level that the rush of water reaches.

Sand - Generally, coarse-grained soils having particle diameters between 0.18 and approximately 0.003 inches. Sands are intermediate between SILT and GRAVEL.
Sandbag - Cloth bag filled with sand or grout and used as a module in a shore protection device.

Sand Fillet- Accretion trapped by a groin or other protrusion in the littoral zone.

Scour - Removal of underwater material by waves or currents, especially at the base or toe of a shore structure.

Screw Anchor - Type of metal anchor screwed into the bottom for holding power.

Seawall - Structure separating land and water areas primarily to prevent erosion and other damage by wave action. See also BULKHEAD.

Semidiurnal Tide - Tide with two high and two low waters in a tidal day, each high and each low approximately equal in stage.

Setup, Wind - Vertical rise in the Stillwater level on a body of water caused by piling up of water on the shore due to wind action. Synonymous with wind tide and STORM SURGE. STORM SURGE usually pertains to the ocean and large bodies of water. Wind setup usually pertains to reservoirs and smaller bodies of water.

Shallow Water - Commonly, water of such a depth that surface waves are noticeably affected by bottom topography. It is customary to consider water of depths less than one-twentieth the surface wavelength as shallow water.

Sheet Pile - see PILE, SHEET.

Shoot - Collective term applied to the STEM and leaves, or any growing branch or twig.

Shore - Narrow strip of land in immediate contact with the sea, inc uding the zone between high and low water lines. A shore of unconsolidated material is usually called a beach.

Shoreline - intersection of a specified plane of water with the shore or beach (e.g., the high water shoreline would be the intersection of the plane of mean high water with the shore or beach). Line delineating the shoreline on National Ocean Survey nautical charts and surveys approximates the mean high water line.

Sill - Low offshore barrier structure whose crest is usually submerged, designed to retain sand on its landward side.

Silt - Generally refers to fine-grained soils having particle diameters between 0.003 and 0.00015 inches. Intermediate between CLAY and SAND.

Slope - Degree of inclination to the horizontal. Usually expressed as a ratio, such as 1:25 or 1 on 25, indicating 1-unit vertical rise in 25 units of horizontal distance; or in degrees from horizontal.

Specifications - Detailed description of particulars, such as size of stone , quality of materials, contractor performance, terms, and quality control.
Sprig - Single plant with its roots relatively bare, as pulled apart from a clump and used for transplanting.

Stem - Main axis of a plant, leaf-bearing and flower-bearing, as distinguished from the root-bearing axis.

Stillwater Level - Elevation that the surface of the water would assume if all wave action were absent.

Storm Surge - Rise above normal water level on the open coast due to action of wind on the water surface. Storm surge resulting from a hurricane also includes the rise in level due to atmospheric pressure reduction as well as that due to wind stress. See SETUP, WIND.

Swell - Wind-generated waves traveling out of their generating area. Swell characteristically exhibits a more regular and longer period, and has flatter crests than waves within their fetch.

Tidal Range - Difference in height between consecutive high and low or higher high and lower low) waters. The mean range is the difference in height between mean high water and mean low water. The diurnal range is the difference in height between mean higher high water and mean lower low water. For diurnal tides, the mean and diurnal ranges are identical. For semidiurnal and mixed tides, the spring range is the difference in height between the high and low waters during the time of spring tides.

Tide - Periodic rising and falling of water resulting from gravitational attraction of the moon, sun and other astronomical bodies acting upon the rotating earth. Although the accompanying horizontal movement of the water resulting from the same cause is also sometimes called tide, it is preferable to designate the latter as tidal current, reserving the name TIDE for vertical movement.

Tide Station - Place at which tide observations are being taken. A primary tide station is a location where continuous observations are taken over a number of years to obtain basic tidal data for the locality. A secondary tide station is operated over a short period of time to obtain data for a specific purpose.

Tie Rod - Steel rod used to tie back the top of a bulkhead or seawall. Also, a U-shaped rod used to tie Sandgrabber blocks together, or a straight rod used to tie Nami Rings together.

Tiller - A plant SHOOT which springs from the root or bottom of the original plant stalk.

Topography - Configuration of a surface, including relief, position of streams, roads, buildings, etc.

Transplant - SHOOT or CULM removed from one location and replanted in another.
Trough of Wave - Lowest part of a waveform between successive crests. Also, that part of a wave below Stillwater level.

Updrift - Direction opposite the predominant movement of littoral materials in longshore transport.

Wake (boat) - Waves generated by the motion of a vessel through water.

Wale - Horizontal beam on a bulkhead used to laterally transfer loads against the structure and hold it in a straight alignment.

Waterline - Juncture of land and sea. This line migrates, changing with the tide or other fluctuation in water level. Where waves are present on the beach, this line is

also known as the limit of backrush. (Approximately, the intersection of land with Stillwater level.)
Wave - Ridge, deformation, or undulation of the surface of a liquid.

Wave Climate - Normal seasonal wave regimen along a shoreline.

Wave Crest - Highest part of a wave or that part above Stillwater level.
Wave Diffraction - See DIFFRACTION.
Wave Direction - Direction from which a wave approaches.

Wave Height - Vertical distance between a crest and the preceding trough.

Wavelength - Horizontal distance between similar points on two successive waves measured perpendicular to the crest.

Wave Period - Time in which a wave crest traverses a distance equal to one wavelength. Time for two successive wave crests to pass a fixed point.

Wave Refraction - See REFRACTION (of water waves).

Wave Steepness - Ratio of wave height to wavelength.

Wave Trough - Lowest part of a wave form between successive crests. Also, that part of a wave below that part of a wave below Stillwater level.

Weep Hole - Hole through a solid revetment, bulkhead, or seawall for relieving pore pressure.

Wind Setup - See SETUP, WIND.

Windward - Direction from which wind is blowing.

Wind Waves - (1) Waves being formed and built up by wind. (2) Loosely, any waves generated by wind.

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APPENDIX

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30 June 1995

**US Army Corps
of Engineers**

ENGINEERING AND DESIGN

Design of Coastal Revetments, Seawalls, and Bulkheads

ENGINEER MANUAL

DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
Washington, DC 20314-1000

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CECW-EH-D

Manual
No. 1110-2-1614

30 June 1995

Engineering and Design
DESIGN OF COASTAL REVETMENTS, SEAWALLS, AND BULKHEADS

1. Purpose. This manual provides guidance for the design of coastal revetment, seawalls, and bulkheads.

2. Applicability. This manual applies to HQUSACE elements, major subordinate commands (MSC), districts, laboratories, and field operating activities (FOA) having civil works responsibilities.

3. Discussion. In areas subject to wind-driven waves and surge, structures such as revetments, seawalls, and bulkheads are commonly employed either to combat erosion or to maintain development at an advanced position from the natural shoreline. Proper performance of such structures is predicated on close adherence to established design guidance. This manual presents important design considerations and describes commonly available materials and structural components. All applicable design guidance must be applied to avoid poor performance or failure. Study of all available structural materials can lead, under some conditions, to innovative designs at significant cost savings for civil works projects.

FOR THE COMMANDER:

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Colonel, Corps of Engineers
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Engineering and Design
DESIGN OF COASTAL REVETMENTS, SEAWALLS, AND BULKHEADS

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Chapter 1
Introduction

1-1. Purpose

This manual provides guidance for the design of coastal revetments, seawalls, and bulkheads.

1-2. Applicability

This manual applies to HQUSACE elements, major subordinate commands, districts, laboratories, and field operating activities having civil works responsibilities.

1-3. References

Required and related publications are listed in Appendix A. Bibliographic items are cited in the text by author and year of publication, with full references listed in Appendix A. If any reference item contains information conflicting with this manual, provisions of this manual govern.

1-4. Background

Structures are often needed along either bluff or beach shorelines to provide protection from wave action or to retain *in situ* soil or fill. Vertical structures are classified as either seawalls or bulkheads, according to their function, while protective materials laid on slopes are called revetments.

a. Revetments. Revetments are generally constructed of durable stone or other materials that will provide sufficient armoring for protected slopes. They consist of an armor layer, filter layer(s), and toe protection. The armor layer may be a random mass of stone or concrete rubble or a well-ordered array of structural elements that interlock to form a geometric pattern. The filter assures drainage and retention of the underlying soil. Toe protection is needed to provide stability against undermining at the bottom of the structure.

b. Bulkheads and seawalls. The terms *bulkhead* and *seawall* are often used interchangeably. However, a bulkhead is primarily intended to retain or prevent sliding of the land, while protecting the upland area against wave action is of secondary importance. Seawalls, on the other hand, are more massive structures whose primary purpose is interception of waves. Bulkheads may be either cantilevered or anchored (like sheetpiling) or gravity structures (such as rock-filled timber cribs). Their use is limited to those areas where wave action can be resisted by such materials. In areas of intense wave action, massive concrete seawalls are generally required. These may have either vertical, concave, or stepped seaward faces.

c. Disadvantages. Revetments, bulkheads, and seawalls mainly protect only the upland area behind them. All share the disadvantage of being potential wave reflectors that can erode a beach fronting the structure. This problem is most prevalent for vertical structures that are nearly perfect wave reflectors and is progressively less prevalent for curved, stepped, and rough inclined structures that absorb or dissipate increasing amounts of wave energy.

1-5. Discussion

The designer is responsible for developing a suitable solution which is economical and achieves the project's purpose (see EM 1110-2-3300). Caution should be exercised, however, when using this manual for anything beyond preliminary design in which the primary goal is cost estimating and screening of alternatives. Final design of large projects usually requires verification by hydraulic model studies. The construction costs of large projects offer considerable opportunities for refinements and possible cost savings as a result of model studies. Model studies should be conducted for all but small projects where limited budgets control and the consequences of failure are not serious.

Chapter 2
Functional Design

2-1. Shoreline Use

Some structures are better suited than others for particular shoreline uses. Revetments of randomly placed stone may hinder access to a beach, while smooth revetments built with concrete blocks generally present little difficulty for walkers. Seawalls and bulkheads can also create an access problem that may require the building of stairs. Bulkheads are required, however, where some depth of water is needed directly at the shore, such as for use by boaters.

2-2. Shoreline Form and Composition

a. Bluff shorelines. Bluff shorelines that are composed of cohesive or granular materials may fail because of scour at the toe or because of slope instabilities aggravated by poor drainage conditions, infiltration, and reduction of effective stresses due to seepage forces. Cantilevered or anchored bulkheads can protect against toe scour and, being embedded, can be used under some conditions to prevent sliding along subsurface critical failure planes. The most obvious limiting factor is the height of the bluff, which determines the magnitude of the earth pressures that must be resisted, and, to some extent, the depth of the critical failure surface. Care must be taken in design to ascertain the relative importance of toe scour and other factors leading to slope instability. Gravity bulkheads and seawalls can provide toe protection for bluffs but have limited applicability where other slope stability problems are present. Exceptions occur in cases where full height retention is provided for low bluffs and where the retained soil behind a bulkhead at the toe of a higher bluff can provide sufficient weight to help counterbalance the active thrust of the bluff materials.

b. Beach shorelines. Revetments, seawalls, and bulkheads can all be used to protect backshore developments along beach shorelines. As described in paragraph 1-4*c*, an important consideration is whether wave reflections may erode the fronting beach.

2-3. Seasonal Variations of Shoreline Profiles

Beach recession in winter and growth in summer can be estimated by periodic site inspections and by computed variations in seasonal beach profiles. The extent of winter beach profile lowering will be a contributing factor in determining the type and extent of needed toe protection.

2-4. Design Conditions for Protective Measures

Structures must withstand the greatest conditions for which damage prevention is claimed in the project plan. All elements must perform satisfactorily (no damage exceeding ordinary maintenance) up to this condition, or it must be shown that an appropriate allowance has been made for deterioration (damage prevention adjusted accordingly and rehabilitation costs amortized if indicated). As a minimum, the design must successfully withstand conditions which have a 50 percent probability of being exceeded during the project's economic life. In addition, failure of the project during probable maximum conditions should not result in a catastrophe (i.e., loss of life or inordinate loss of money).

2-5. Design Water Levels

The maximum water level is needed to estimate the maximum breaking wave height at the structure, the amount of runup to be expected, and the required crest elevation of the structure. Minimum expected water levels play an important role in anticipating the amount of toe scour that may occur and the depth to which the armor layer should extend.

a. Astronomical tides. Changes in water level are caused by astronomical tides with an additional possible component due to meteorological factors (wind setup and pressure effects). Predicted tide levels are published annually by the National Oceanic and Atmospheric Administration (NOAA). The statistical characteristics of astronomical tides at various U.S. ports were analyzed in Harris (1981) with probability density functions of water levels summarized in a series of graphs and tables. Similar tables are available for the Atlantic Coast in Ebersole (1982) which also includes estimates of storm surge values.

b. Storm surge. Storm surge can be estimated by statistical analysis of historical records, by methods described in Chapter 3 of the Shore Protection Manual (SPM), or through the use of numerical models. The numerical models are usually justified only for large projects. Some models can be applied to open coast studies, while others can be used for bays and estuaries where the effects of inundation must be considered.

c. Lake levels. Water levels on the Great Lakes are subject to both periodic and nonperiodic changes. Records dating from 1836 reveal seasonal and annual changes due to variations in precipitation. Lake levels (particularly Ontario and Superior) are also partially controlled by regulatory works operated jointly by Canadian and U.S. authorities. These tend to minimize water level variations in those lakes. Six-month forecasts of lake levels are published monthly by the Detroit District (Figure 2-1).

2-6. Design Wave Estimation

Wave heights and periods should be chosen to produce the most critical combination of forces on a structure with due consideration of the economic life, structural integrity, and hazard for events that may exceed the design conditions (see paragraph 2-4). Wave characteristics may be based on an analysis of wave gauge records, visual observations of wave action, published wave hindcasts, wave forecasts, or the maximum breaking wave at the site. Wave characteristics derived from such methods may be for deepwater locations and must be transformed to the structure site using refraction and diffraction techniques as described in the SPM. Wave analyses may have to be performed for extreme high and low design water levels and for one or more intermediate levels to determine the critical design conditions.

2-7. Wave Height and Period Variability and Significant Waves

a. Wave height.

(1) A given wave train contains individual waves of varying height and period. The significant wave height, H_s, is defined as the average height of the highest one-third of all the waves in a wave train. Other wave heights such as H_{10} and H_1 can also be designated, where H_{10} is the average of the highest 10 percent of all waves, and H_1 is the average of the highest 1 percent of all waves. By assuming a Rayleigh distribution, it can be stated that

$$H_{10} \approx 1.27\,H_s \qquad (2\text{-}1)$$

and

$$H_1 \approx 1.67\,H_s \qquad (2\text{-}2)$$

(2) Available wave information is frequently given as the energy-based height of the zeroth moment, H_{mo}. In deep water, H_s and H_{mo} are about equal; however, they may be significantly different in shallow water due to shoaling (Thompson and Vincent 1985). The following equation may be used to equate H_s from energy-based wave parameters (Hughes and Borgman 1987):

$$\frac{H_s}{H_{mo}} = \exp\left[C_0\left(\frac{d}{g\,T_p^2}\right)^{-C_1}\right] \qquad (2\text{-}3)$$

where

C_0, C_1 = regression coefficients given as 0.00089 and 0.834, respectively

d = water depth at point in question (i.e., toe of structure)

g = acceleration of gravity

T_p = period of peak energy density of the wave spectrum

A conservative value of H_s may be obtained by using 0.00136 for C_0, which gives a reasonable upper envelope for the data in Hughes and Borgman. Equation 2-3 should not be used for

$$\frac{d}{g\,T_p^2} < 0.0005 \qquad (2\text{-}4)$$

or where there is substantial wave breaking.

(3) In shallow water, H_s is estimated from deepwater conditions using the irregular wave shoaling and breaking model of Goda (1975, 1985) which is available as part of the Automated Coastal Engineering System (ACES) package (Leenknecht et al. 1989). Goda (1985) recommends for the design of rubble structures that if the depth is less than one-half the deepwater significant wave height, then design should be based on the significant wave height at a depth equal to one-half the significant deepwater wave height.

b. Wave period. Wave period for spectral wave conditions is typically given as period of the peak energy density of the spectrum, T_p. However, it is not uncommon to find references and design formulae based on the average wave period (T_z) or the significant wave period (T_s , average period of the one-third highest waves). Rough guidance on the relationship among these wave periods is given in Table 2.1.

c. Stability considerations. The wave height to be used for stability considerations depends on whether the

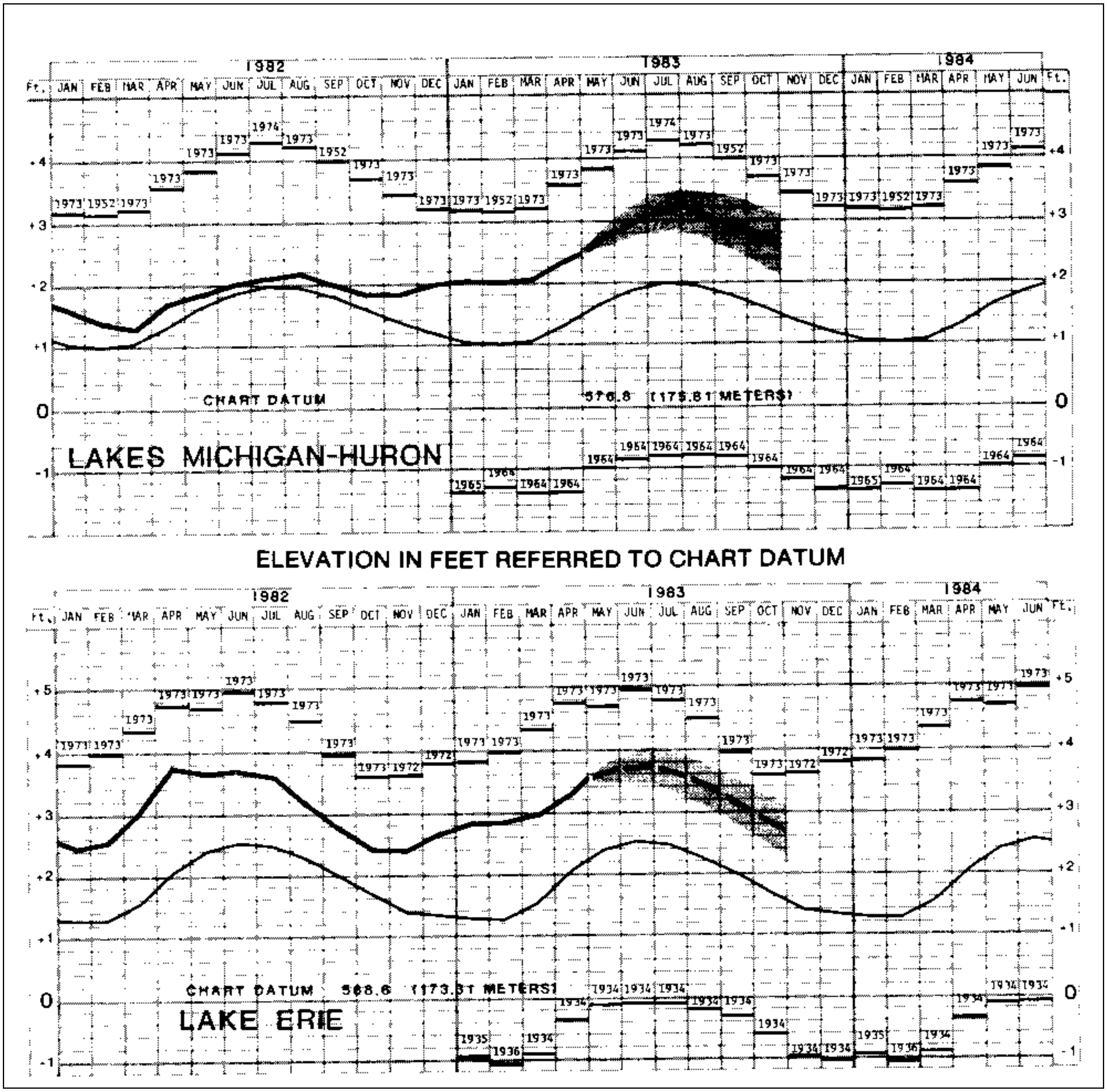


Figure 2-1. Monthly lake level forecast

structure is rigid, semirigid, or flexible. Rigid structures that could fail catastrophically if overstressed may warrant design based on H_1. Semirigid structures may warrant a design wave between H_1 and H_{10}. Flexible structures are usually designed for H_s or H_{10}. Stability coefficients are coupled with these wave heights to develop various degrees of damage, including no damage.

2-8. Wave Gauges and Visual Observations

Available wave data for use by designers is often sparse and limited to specific sites. In addition, existing gauge data are sometimes analog records which have not been analyzed and that are difficult to process. Project funding

Table 2-1
Relationships among T_p, T_s, and T_z

T_z / T_p	T_s / T_p	Comments	γ
0.67	0.80	Severe surf zone conditions[1]	NA
0.74	0.88	Pierson-Moskowitz spectrum[2]	1.0
0.80	0.93	Typical JONSWAP spectrum[2]	3.3
0.87	0.96	Swell from distant storms[2]	10.0

[1] Developed from data in Ahrens (1987).
[2] Developed from Goda (1987).

and time constraints may prohibit the establishment of a viable gauging program that would provide sufficient digital data for reliable study. Visual observations from shoreline points are convenient and inexpensive, but they have questionable accuracy, are often skewed by the omission of extreme events, and are sometimes difficult to extrapolate to other sites along the coast. A visual wave observation program is described in Schneider (1981). Problems with shipboard observations are similar to shore observations.

2-9. Wave Hindcasts

Designers should use the simple hindcasting methods in ACES (Leenknecht et al. 1989) and hindcasts developed by the U.S. Army Engineer Waterways Experiment Station (WES) (Resio and Vincent 1976-1978; Corson et al. 1981) for U.S. coastal waters using numerical models. These later results are presented in a series of tables for each of the U.S. coasts. They give wave heights and periods as a function of season, direction of wave approach, and return period; wave height as a function of return period and seasons combined; and wave period as a function of wave height and approach angle. Several other models exist for either shallow or deep water. Specific applications depend on available wind data as well as bathymetry and topography. Engineers should stay abreast of developments and choose the best method for a given analysis. Contact the Coastal Engineering Research Center (CERC) at WES for guidance in special cases.

2-10. Wave Forecasts

Wave forecasts can be performed using the same methodologies as those for the wave hindcasts. Normally, the Corps hindcasts waves for project design, and the Navy forecasts waves to plan naval operations.

2-11. Breaking Waves

a. Wave heights derived from a hindcast should be checked against the maximum breaking wave that can be supported at the site given the available depth at the design still-water level and the nearshore bottom slope. Figure 2-2 (Weggel 1972) gives the maximum breaker height, H_b, as a function of the depth at the structure, d_s , nearshore bottom slope, m, and wave period, T. Design wave heights, therefore, will be the *smaller* of the maximum breaker height or the hindcast wave height.

b. For the severe conditions commonly used for design, H_{mo} may be limited by breaking wave conditions. A reasonable upper bound for H_{mo} is given by

$$\left(H_{mo}\right)_{\max} = 0.10\, L_p \tanh\left(\frac{2\pi d}{L_p}\right) \tag{2-5}$$

where L_p is wavelength calculated using T_p and d.

2-12. Height of Protection

When selecting the height of protection, one must consider the maximum water level, any anticipated structure settlement, freeboard, and wave runup and overtopping.

2-13. Wave Runup

Runup is the vertical height above the still-water level (swl) to which the uprush from a wave will rise on a structure. Note that it is not the distance measured along the inclined surface.

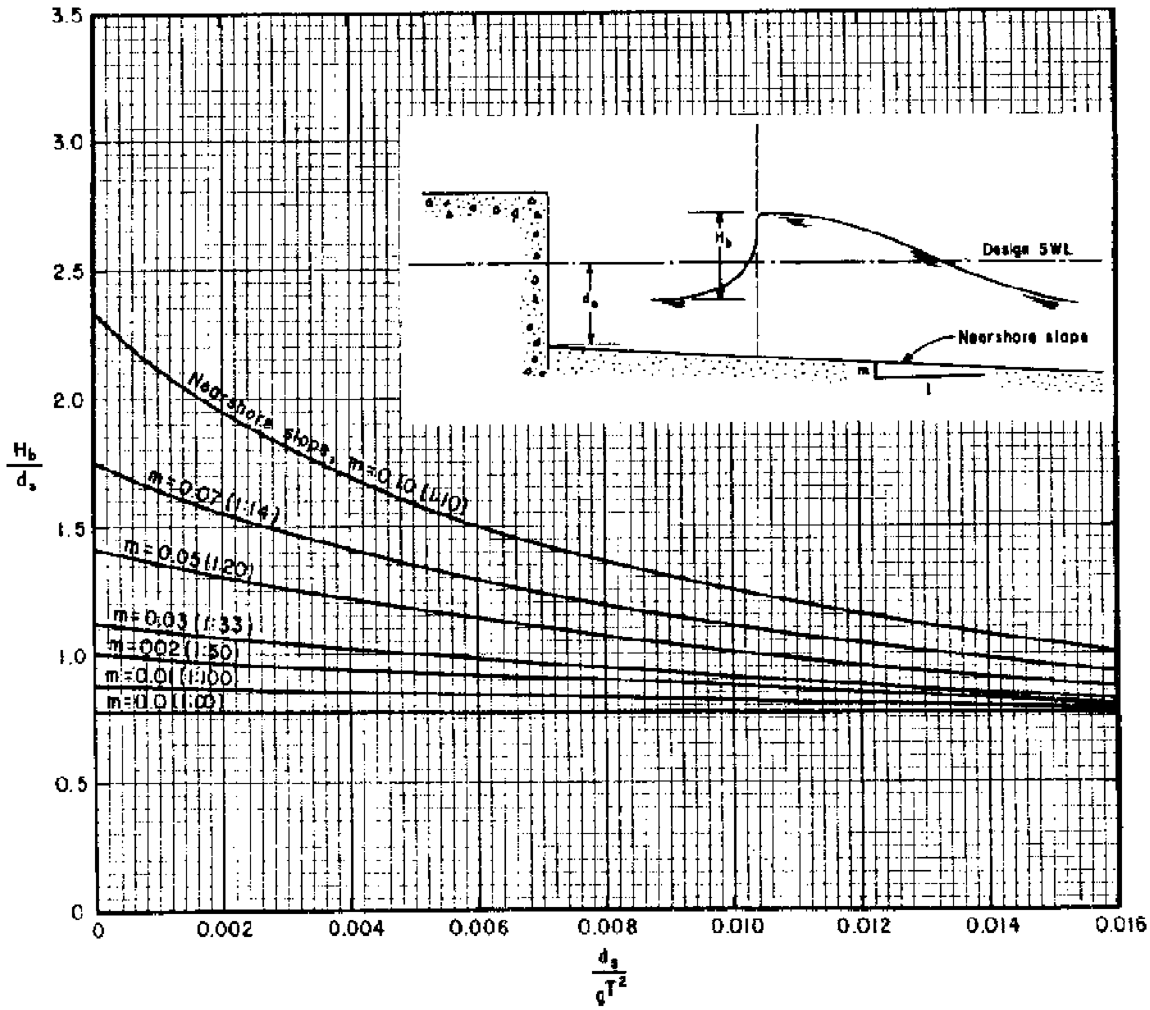


Figure 2-2. Design breaker height

a. Rough slope runup.

(1) Maximum runup by irregular waves on riprap-covered revetments may be estimated by (Ahrens and Heimbaugh 1988)

$$\frac{R_{max}}{H_{mo}} = \frac{a\xi}{1 + b\xi} \tag{2-6}$$

where

R_{max} = maximum vertical height of the runup above the swl

a, b = regression coefficients determined as 1.022 and 0.247, respectively

ξ = surf parameter defined by

$$\xi = \frac{\tan\theta}{\left(\dfrac{2\pi H_{mo}}{gT_p^2}\right)^{1/2}} \tag{2-7}$$

where θ is the angle of the revetment slope with the horizontal. Recalling that the deepwater wavelength may be determined by

$$L_o = \frac{g T_p^2}{2\pi} \tag{2-8}$$

the surf parameter is seen to be the ratio of revetment slope to square root of wave steepness. The surf parameter is useful in defining the type of breaking wave conditions expected on the structure, as shown in Figure 2-3.

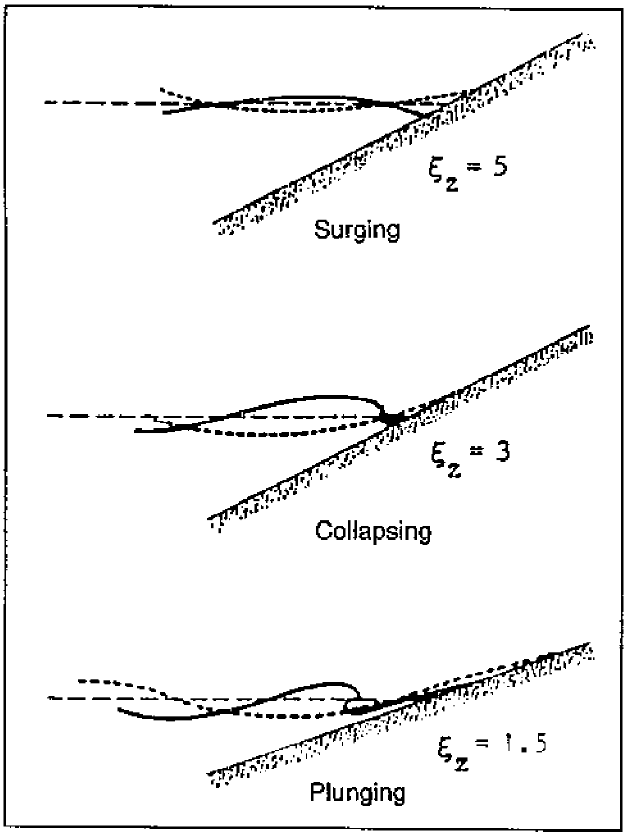


Figure 2-3. Surf parameter and breaking wave types

(2) A more conservative value for R_{max} is obtained by using 1.286 for *a* in Equation 2-6. Maximum runups determined using this more conservative value for *a* provide a reasonable upper limit to the data from which the equation was developed.

(3) Runup estimates for revetments covered with materials other than riprap may be obtained with the rough slope correction factors in Table 2-2. Table 2-2 was developed for earlier estimates of runup based on monochromatic wave data and smooth slopes. To use the correction factors in Table 2-2 with the irregular wave rough slope runup estimates of Equation 2-6, multiply R_{max} in Equation 2-6 by the correction factor listed in Table 2-2, and divide by the correction factor for quarrystone. For example, to estimate R_{max} for a stepped 1:1.5 slope with vertical risers, determine R_{max} by Equation 2-6 and multiply by (correction factor for stepped slope/correction factor for quarrystone) (0.75/0.60) = 1.25. R_{max} for the stepped slope is seen to be 25 percent greater than for a riprap slope.

b. Smooth slope runup. Runup values for smooth slopes may be found in design curves in the SPM. However, the smooth slope runup curves in the SPM were based on monochromatic wave tests rather than more realistic irregular wave conditions. Using H_s for wave height with the design curves will yield runup estimates that may be exceeded by as much as 50 percent by waves in the wave train with heights greater than H_s. Maximum runup may be estimated by using Equation 2-6 and converting the estimate to smooth slope by dividing the result by the quarrystone rough slope correction factor in Table 2-2.

c. Runup on walls. Runup determinations for vertical and curved-face walls should be made using the guidance given in the SPM.

2-14. Wave Overtopping

a. It is generally preferable to design shore protection structures to be high enough to preclude overtopping. In some cases, however, prohibitive costs or other considerations may dictate lower structures than ideally needed. In those cases it may be necessary to estimate the volume of water per unit time that may overtop the structure.

b. Wave overtopping of riprap revetments may be estimated from the dimensionless equation (Ward 1992)

$$Q' = C_0 \, e^{C_1 F'} \, e^{C_2 m} \tag{2-9}$$

where Q' is dimensionless overtopping defined as

$$Q' = \frac{Q}{\left(g H_{mo}^3\right)^{1/2}} \tag{2-10}$$

where Q is dimensional overtopping in consistent units, such as cfs/ft. F' in Equation 2-9 is dimensionless freeboard defined as

Table 2-2
Rough Slope Runup Correction Factors (Carstea et al. 1975b)

Armor Type	Slope (cot θ)	Relative Size H / K_r [a,b]	Correction Factor r
Quarrystone	1.5	3 to 4	0.60
Quarrystone	2.5	3 to 4	0.63
Quarrystone	3.5	3 to 4	0.60
Quarrystone	5	3	0.60
Quarrystone	5	4	0.68
Quarrystone	5	5	0.72
Concrete Blocks[c]	Any	6[b]	0.93
Stepped slope with vertical risers	1.5	$1 \le H_o'/K_r$ [d]	0.75
Stepped slope with vertical risers	2.0	$1 \le H_o'/K_r$ [d]	0.75
Stepped slope with vertical risers	3.0	$1 \le H_o'/K_r$ [d]	0.70
Stepped slope with rounded edges	3.0	$1 \le H_o'/K_r$ [d]	0.86
Concrete Armor Units			
Tetrapods random two layers	1.3 to 3.0	-	0.45
Tetrapods uniform two layers	1.3 to 3.0	-	0.51
Tribars random two layers	1.3 to 3.0	-	0.45
Tribars uniform one layer	1.3 to 3.0	-	0.50

[a] K_r is the characteristic height of the armor unit perpendicular to the slope. For quarrystone, it is the nominal diameter; for armor units, the height above the slope.
[b] Use H_o' for $d_s/H_o' > 3$; and the local wave height, H_s for $d_s/H_o' \le 3$.
[c] Perforated surfaces of Gobi Blocks, Monoslaps, and concrete masonry units placed hollows up.
[d] K_r is the riser height.

$$F' = \frac{F}{\left(H_{mo}^2 L_o\right)^{1/3}} \tag{2-11}$$

where F is dimensional freeboard (vertical distance of crest above swl). The remaining terms in Equation 2-9 are m (cotangent of revetment slope) and the regression coefficients C_0, C_1, and C_2 defined as

$$\begin{aligned} C_0 &= 0.4578 \\ C_1 &= -29.45 \\ C_2 &= 0.8464 \end{aligned} \tag{2-12}$$

The coefficients listed above were determined for dimensionless freeboards in the range $0.25 < F' < 0.43$, and revetment slopes of 1:2 and 1:3.5.

c. Overtopping rates for seawalls are complicated by the numerous shapes found on the seawall face plus the variety of fronting berms, revetments, and steps. Information on overtopping rates for a range of configurations is available in Ward and Ahrens (1992). For bulkheads and simple vertical seawalls with no fronting revetment and a small parapet at the crest, the overtopping rate may be calculated from

$$Q' = C_0 \exp\left[C_1 F' + C_2\left(\frac{F}{d_s}\right)\right] \tag{2-13}$$

where Q' is defined in Equation 2-10, F' is defined in Equation 2-11, d_s is depth at structure toe, and the regression coefficients are defined by

$$\begin{aligned} C_0 &= 0.338 \\ C_1 &= -7.385 \\ C_2 &= -2.178 \end{aligned} \tag{2-14}$$

For other configurations of seawalls, Ward and Ahrens (1992) should be consulted, or physical model tests should be performed.

2-15. Stability and Flexibility

Structures can be built by using large monolithic masses that resist wave forces or by using aggregations of smaller units that are placed either in a random or in a well-ordered array. Examples of these are large reinforced concrete seawalls, quarrystone or riprap revetments, and geometric concrete block revetments. The massive monoliths and interlocking blocks often exhibit superior initial strength but, lacking flexibility, may not accommodate small amounts of differential settlement or toe scour that may lead to premature failure. Randomly placed rock or concrete armor units, on the other hand, experience settlement and readjustment under wave attack, and, up to a point, have reserve strength over design conditions. They typically do not fail catastrophically if minor damages are inflicted. The equations in this chapter are suitable for preliminary design for major structures. However, final design will usually require verification of stability and performance by hydraulic model studies. The design guidance herein may be used for final design for small structures where the consequences of failure are minor. For those cases, project funds are usually too limited to permit model studies.

2-16. Armor Unit Stability

a. The most widely used measure of armor unit stability is that developed by Hudson (1961) which is given in Equation 2-15:

$$W = \frac{\gamma_r H^3}{K_D \left(\frac{\gamma_r}{\gamma_w} - 1 \right)^3 \cot \theta} \tag{2-15}$$

where

W = required individual armor unit weight, lb (or W_{50} for graded riprap)

γ_r = specific weight of the armor unit, lb/ft^3

H = monochromatic wave height

K_D = stability coefficient given in Table 2-3

γ_w = specific weight of water at the site (salt or fresh)

θ = is structure slope (from the horizontal)

Stones within the cover layer can range from 0.75 to 1.25 *W* as long as 50 percent weigh at least *W* and the gradation is uniform across the structure's surface. Equation 2-15 can be used for preliminary and final design when *H* is less than 5 ft and there is no major overtopping of the structure. For larger wave heights, model tests are preferable to develop the optimum design. Armor weights determined with Equation 2-15 for monochromatic waves should be verified during model tests using spectral wave conditions.

b. Equation 2-15 is frequently presented as a stability formula with N_s as a stability number. Rewriting Equation 2-15 as

$$N_s = \frac{H}{\left(\frac{W}{\gamma_r} \right)^{1/3} \left(\frac{\gamma_r}{\gamma_w} - 1 \right)} \tag{2-16}$$

it is readily seen that

$$N_s = \left(K_D \cot \theta \right)^{1/3} \tag{2-17}$$

By equating Equations 2-16 and 2-17, *W* is readily obtained.

c. For irregular wave conditions on revetments of dumped riprap, the recommended stability number is

$$N_{sz} = 1.14 \cot^{1/6} \theta \tag{2-18}$$

where N_{sz} is the zero-damage stability number, and the value 1.14 is obtained from Ahrens (1981b), which recommended a value of 1.45 and using H_s with Equation 2-16, then modified based on Broderick (1983), which found using H_{10} (10 percent wave height, or average of highest 10-percent of the waves) in Equation 2-16 provided a better fit to the data. Assuming a Rayleigh wave height distribution, $H_{10} \approx 1.27\ H_s$. Because H_s is more readily available than H_{10}, the stability number in Equation 2-17 was adjusted (1.45/1.27 = 1.14) to allow H_s to be used in the stability equation while providing the more conservative effect of using H_{10} for the design.

d. Stability equations derived from an extensive series of laboratory tests in The Netherlands were presented in van der Meer and Pilarczyk (1987) and van der

Table 2-3
Suggested Values for Use In Determining Armor Weight (Breaking Wave Conditions)

Armor Unit	n[1]	Placement	Slope (cot θ)	K_D
Quarrystone				
Smooth rounded	2	Random	1.5 to 3.0	1.2
Smooth rounded	>3	Random	1.5 to 3.0	1.6
Rough angular	1	Random	1.5 to 3.0	Do Not Use
Rough angular	2	Random	1.5 to 3.0	2.0
Rough angular	>3	Random	1.5 to 3.0	2.2
Rough angular	2	Special[2]	1.5 to 3.0	7.0 to 20.0
Graded riprap[3]	2[4]	Random	2.0 to 6.0	2.2
Concrete Armor Units				
Tetrapod	2	Random	1.5 to 3.0	7.0
Tripod	2	Random	1.5 to 3.0	9.0
Tripod	1	Uniform	1.5 to 3.0	12.0
Dolos	2	Random	2.0 to 3.0[5]	15.0[6]

[1] n equals the number of equivalent spherical diameters corresponding to the median stone weight that would fit within the layer thickness.

[2] Special placement with long axes of stone placed perpendicular to the slope face. Model tests are described in Markle and Davidson (1979).

[3] Graded riprap is not recommended where wave heights exceed 5 ft.

[4] By definition, graded riprap thickness is two times the diameter of the minimum W_{50} size.

[5] Stability of dolosse on slope steeper than 1 on 2 should be verified by model tests.

[6] No damage design (3 to 5 percent of units move). If no rocking of armor (less than 2 percent) is desired, reduce K_D by approximately 50 percent.

Meer (1988a, 1988b). Two stability equations were presented. For plunging waves,

$$N_s = 6.2\, P^{\,0.18} \left(\frac{S}{\sqrt{N}} \right)^{0.2} \xi_z^{0.5} \tag{2-19}$$

and for surging or nonbreaking waves,

$$N_s = 1.0\, P^{\,-0.13} \left(\frac{S}{\sqrt{N}} \right)^{0.2} \sqrt{\cot\theta}\; \xi_z^{P} \tag{2-20}$$

where

P = permeability coefficient

S = damage level

N = number of waves

P varies from P = 0.1 for a riprap revetment over an impermeable slope to P = 0.6 for a mound of armor stone with no core. For the start of damage S = 2 for revetment slopes of 1:2 or 1:3, or S = 3 for revetment slopes of 1:4 to 1:6. The number of waves is difficult to estimate, but Equations 2-19 and 2-20 are valid for N = 1,000 to N = 7,000, so selecting 7,000 waves should provide a conservative estimate for stability. For structures other than riprap revetments, additional values of P and S are presented in van der Meer (1988a, 1988b).

e. Equations 2-19 and 2-20 were developed for deepwater wave conditions and do not include a wave-height truncation due to wave breaking. van der Meer therefore recommends a shallow water correction given as

$$\frac{N_{s\ (shallow\ water)}}{N_{s\ (deep\ water)}} = \frac{1.40\, H_s}{H_2} \tag{2-21}$$

where H_2 is the wave height exceeded by 2 percent of the waves. In deep water, $H_2 \approx 1.40\ H_s$, and there is no correction in Equation 2-21.

2-17. Layer Thickness

a. Armor units. As indicated in the SPM, the thickness of an armor layer can be determined by Equation 2-22:

$$r = n\,k_\Delta \left(\frac{W}{w_r} \right)^{1/3} \tag{2-22}$$

where r is the layer thickness in feet, n is the number of armor units that would fit within the layer thickness (typically n=2), and k_Δ is the layer coefficient given in Table 2-4. For estimating purposes, the number of armor units, N_r, for a given surface area in square feet, A, is

$$N_r = A\,n\,k_\Delta \left(1 - \frac{P}{100} \right) \left(\frac{w_r}{W} \right)^{\frac{2}{3}} \tag{2-23}$$

where P is the average porosity of the cover layer from Table 2-4.

b. Graded riprap. The layer thickness for graded riprap must be at least twice the nominal diameter of the W_{50} stone, where the nominal diameter is the cube root of the stone volume. In addition, $r_{\min}$ should be at least 25 percent greater than the nominal diameter of the largest stone and should always be greater than a minimum layer thickness of 1 ft (Ahrens 1975). Therefore,

$$r_{\min} = \max \left[2.0 \left(\frac{W_{50\,\min}}{\gamma_r} \right)^{1/3} ; \; 1.25 \left(\frac{W_{100}}{\gamma_r} \right)^{1/3} ; \; 1 \; ft \right] \tag{2-24}$$

where $r_{\min}$ is the minimum layer thickness perpendicular to the slope. Greater layer thicknesses will tend to increase the reserve strength of the revetment against waves greater than the design. Gradation (within broad limits) appears to have little effect on stability provided the W_{50} size is used to characterize the layer. The following are suggested guidelines for establishing gradation limits (from EM 1110-2-1601) (see also Ahrens 1981a):

(1) The lower limit of W_{50} stone, $W_{50\,\min}$, should be selected based on stability requirements using Equation 2-15.

(2) The upper limit of the W_{100} stone, $W_{100\,\max}$, should equal the maximum size that can be economically obtained from the quarry but not exceed 4 times $W_{50\,\min}$.

(3) The lower limit of the W_{100} stone, $W_{100\,\min}$, should not be less than twice $W_{50\,\min}$.

(4) The upper limit of the W_{50} stone, $W_{50\,\max}$, should be about 1.5 times $W_{50\,\min}$.

(5) The lower limit of the W_{15} stone, $W_{15\,\min}$, should be about 0.4 times $W_{50\,\min}$.

(6) The upper limit of the W_{15} stone, $W_{15\,\max}$, should be selected based on filter requirements specified in EM 1110-2-1901. It should slightly exceed $W_{50\,\min}$.

(7) The bulk volume of stone lighter than $W_{15\,\min}$ in a gradation should not exceed the volume of voids in the revetment without this lighter stone. In many cases, however, the actual quarry yield available will differ from the gradation limits specified above. In those cases the designer must exercise judgment as to the suitability of the supplied gradation. Primary consideration should be given to the $W_{50\,\min}$ size under those circumstances. For instance, broader than recommended gradations may be suitable if the supplied W_{50} is somewhat heavier than the required $W_{50\,\min}$. Segregation becomes a major problem, however, when the riprap is too broadly graded.

2-18. Reserve Stability

a. General. A well-known quality of randomly placed rubble structures is the ability to adjust and resettle under wave conditions that cause minor damages. This has been called reserve strength or reserve stability. Structures built of regular or uniformly placed units such as concrete blocks commonly have little or no reserve stability and may fail rapidly if submitted to greater than design conditions.

b. Armor units. Values for the stability coefficient, K_D, given in paragraph 2-16 allow up to 5 percent damages under design wave conditions. Table 2-5 contains values of wave heights producing increasing levels of damage. The wave heights are referenced to the zero-damage wave height ($H_{D=0}$) as used in Equation 2-15. Exposure of armor sized for $H_{D=0}$ to these larger wave heights should produce damages in the range given. If the armor stone available at a site is lighter than the stone size calculated using the wave height at the site, the zero-damage wave height for the available stone can be

Table 2-4
Layer Coefficients and Porosity for Various Armor Units

Armor Unit	n	Placement	K_Δ	P (%)
Quarrystone (smooth)	2	Random	1.00	38
Quarrystone (rough)	2	Random	1.00	37
Quarrystone (rough)	≥3	Random	1.00	40
Graded riprap	2[a]	Random	N/A	37
Tetrapod	2	Random	1.04	50
Tribar	2	Random	1.02	54
Tribar	1	Uniform	1.13	47
Dolos	2	Random	0.94	56

[a] By definition, riprap thickness equals two cubic lengths of W_{50} or 1.25 W_{100}.

Table 2-5
$H/H_{D=0}$ for Cover Layer Damage Levels for Various Armor Types ($H/H_{D=0}$ for Damage Level in Percent)

Unit	$0 \le \%_D < 5$	$5 \le \%_D < 10$	$10 \le \%_D < 15$	$15 \le \%_D < 20$	$20 \le \%_D \le 30$
Quarrystone (smooth)	1.00	1.08	1.14	1.20	1.29
Quarrystone (angular)	1.00	1.08	1.19	1.27	1.37
Tetrapods	1.00	1.09	1.17	1.24	1.32
Tribars	1.00	1.11	1.25	1.36	1.50
Dolos	1.00	1.10	1.14	1.17	1.20

calculated, and a ratio with the site's wave height can be used to estimate the damage that can be expected with the available stone. All values in the table are for randomly placed units, n=2, and minor overtopping. The values in Table 2-5 are adapted from Table 7-8 of the SPM. The SPM values are for breakwater design and nonbreaking wave conditions and include damage levels above 30 percent. Due to differences in the form of damage to breakwaters and revetments, revetments may fail before damages reach 30 percent. The values should be used with caution for damage levels from breaking and non-breaking waves.

c. Graded riprap. Information on riprap reserve stability can be found in Ahrens (1981a). Reserve stability appears to be primarily related to the layer thickness although the median stone weight and structure slope are also important.

2-19. Toe Protection

a. General. Toe protection is supplemental armoring of the beach or bottom surface in front of a structure which prevents waves from scouring and under-cutting it. Factors that affect the severity of toe scour include wave breaking (when near the toe), wave runup and backwash, wave reflection, and grain-size distribution of the beach or bottom materials. The revetment toe often requires special consideration because it is subjected to both hydraulic forces and the changing profiles of the beach fronting the revetment. Toe stability is essential because failure of the toe will generally lead to failure throughout the entire structure. Specific guidance for toe design based on either prototype or model results has not been developed. Some empirical suggested guidance is contained in Eckert (1983).

b. Revetments.

(1) Design procedure. Toe protection for revetments is generally governed by hydraulic criteria. Scour can be caused by waves, wave-induced currents, or tidal currents. For most revetments, waves and wave-induced currents will be most important. For submerged toe stone, weights can be predicted based on Equation 2-25:

$$W_{min} = \frac{\gamma_r H^3}{N_s^3 \left(\frac{\gamma_r}{\gamma_w} - 1 \right)^3} \tag{2-25}$$

where N_s is the design stability number for rubble toe protection in front of a vertical wall, as indicated in the SPM (see Figure 2-7). For toe structures exposed to wave action, the designer must select either Equation 2-15 which applies at or near the water surface or Equation 2-25 above. It should be recognized that Equation 2-25 yields a minimum weight and Equation 2-15 yields a median weight. Stone selection should be based on the weight gradations developed from each of the stone weights. The relative importance of these factors depends on the location of the structure and its elevation with respect to low water. When the toe protection is for scour caused by tidal or riverine currents alone, the designer is referred to EM 1110-2-1601. Virtually no data exist on currents acting on toe stone when they are a product of storm waves and tidal or riverine flow. It is assumed that the scour effects are partially additive. In the case of a revetment toe, some conservatism is provided by using the design stability number for toe protection in front of a vertical wall as suggested above.

(2) Suggested toe configurations. Guidance contained in EM 1110-2-1601 which relates to toe design configurations for flood control channels is modified for coastal revetments and presented in Figure 2-4. This is offered solely to illustrate possible toe configurations. Other schemes known to be satisfactory by the designer are also acceptable. Designs I, II, IV, and V are for up to moderate toe scour conditions and construction in the dry. Designs III and VI can be used to reduce excavation when the stone in the toe trench is considered sacrificial and will be replaced after infrequent major events. A thickened toe similar to that in Design III can be used for underwater construction except that the toe stone is placed on the existing bottom rather than in an excavated trench.

c. Seawalls and bulkheads.

(1) General considerations. Design of toe protection for seawalls and bulkheads must consider geotechnical as well as hydraulic factors. Cantilevered, anchored, or gravity walls each depend on the soil in the toe area for their support. For cantilevered and anchored walls, this passive earth pressure zone must be maintained for stability against overturning. Gravity walls resist sliding through the frictional resistance developed between the soil and the base of the structure. Overturning is resisted by the moment of its own weight supported by the zone of bearing beneath the toe of the structure. Possible toe configurations are shown in Figure 2-5.

(2) Seepage forces. The hydraulic gradients of seepage flows beneath vertical walls can significantly increase toe scour. Steep exit gradients reduce the net effective weight of the soil, making sediment movement under waves and currents more likely. This seepage flow may originate from general groundwater conditions, water derived from wave overtopping of the structure, or from precipitation. A quantitative treatment of these factors is presented in Richart and Schmertmann (1958).

(3) Toe apron width. The toe apron width will depend on geotechnical and hydraulic factors. The passive earth pressure zone must be protected for a sheet-pile wall as shown in Figure 2-6. The minimum width, B, from a geotechnical perspective can be derived using the Rankine theory as described in Eckert (1983). In these cases the toe apron should be wider than the product of the effective embedment depth and the coefficient of passive earth pressure for the soil. Using hydraulic considerations, the toe apron should be at least twice the incident wave height for sheet-pile walls and equal to the incident wave height for gravity walls. In addition, the apron should be at least 40 percent of the depth at the structure, d_s. Greatest width predicted by these geotechnical and hydraulic factors should be used for design. In all cases, undercutting and unraveling of the edge of the apron must be minimized.

(4) Toe stone weight. Toe stone weight can be predicted based on Figure 2-7 (from Brebner and Donnelly 1962)). A design wave between H_1 and H_{10} is suggested. To apply the method assume a value of d_t the distance from the still water level to the top of the toe. If the resulting stone size and section geometry are not appropriate, a different d_t should be tried. Using the median stone weight determined by this method, the allowable gradation should be approximately 0.5 to 1.5 W.

2-20. Filters

A filter is a transitional layer of gravel, small stone, or fabric placed between the underlying soil and the structure. The filter prevents the migration of the fine soil particles through voids in the structure, distributes the weight of the armor units to provide more uniform settlement, and permits relief of hydrostatic pressures within the soils. For areas above the waterline, filters also

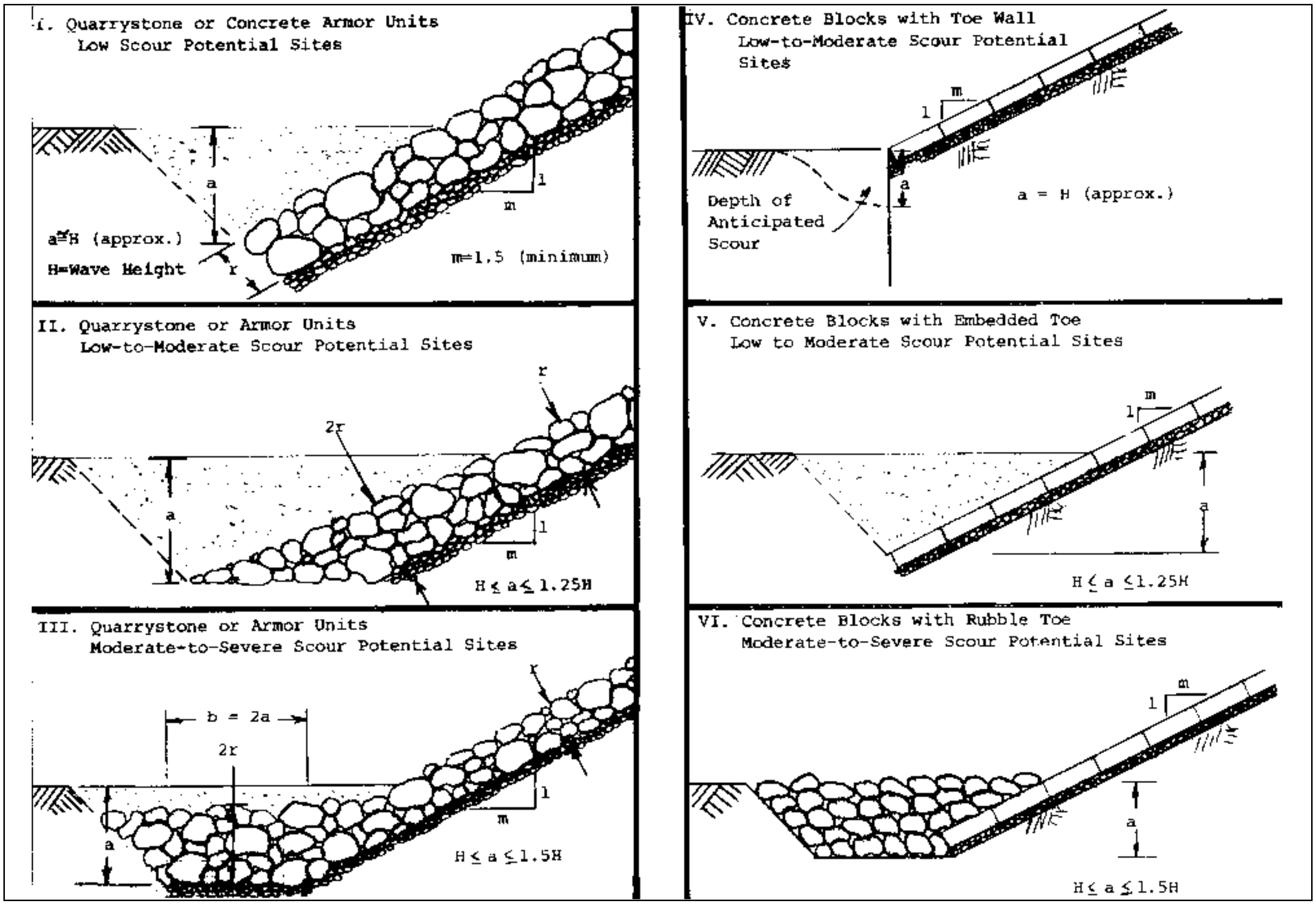


Figure 2-4. Revetment toe protection (Designs I through VI)

prevent surface water from causing erosion (gullies) beneath the riprap. In general form layers have the relation given in Equation 2-26:

$$\frac{d_{15\,upper}}{d_{85\,under}} < 4 \tag{2-26}$$

Specific design guidance for gravel and stone filters is contained in EM 1110-2-1901 and EM 1110-2-2300 (see also Ahrens 1981a), and guidance for cloth filters is contained in CW 02215. The requirements contained in these will be briefly summarized in the following paragraphs.

a. Graded rock filters. The filter criteria can be stated as:

$$\frac{d_{15\,filter}}{d_{85\,soil}} < 4 \textit{ to } 5 < \frac{d_{15\,filter}}{d_{15\,soil}} \tag{2-27}$$

where the left side of Equation 2-27 is intended to prevent piping through the filter and the right side of Equation 2-27 provides for adequate permeability for structural bedding layers. This guidance also applies between successive layers of multilayered structures. Such designs are needed where a large disparity exists between the void size in the armor layer and the particle sizes in the underlying layer.

b. Riprap and armor stone underlayers. Underlayers for riprap revetments should be sized as in Equation 2-28,

$$\frac{d_{15\,armor}}{d_{85\,filter}} < 4 \tag{2-28}$$

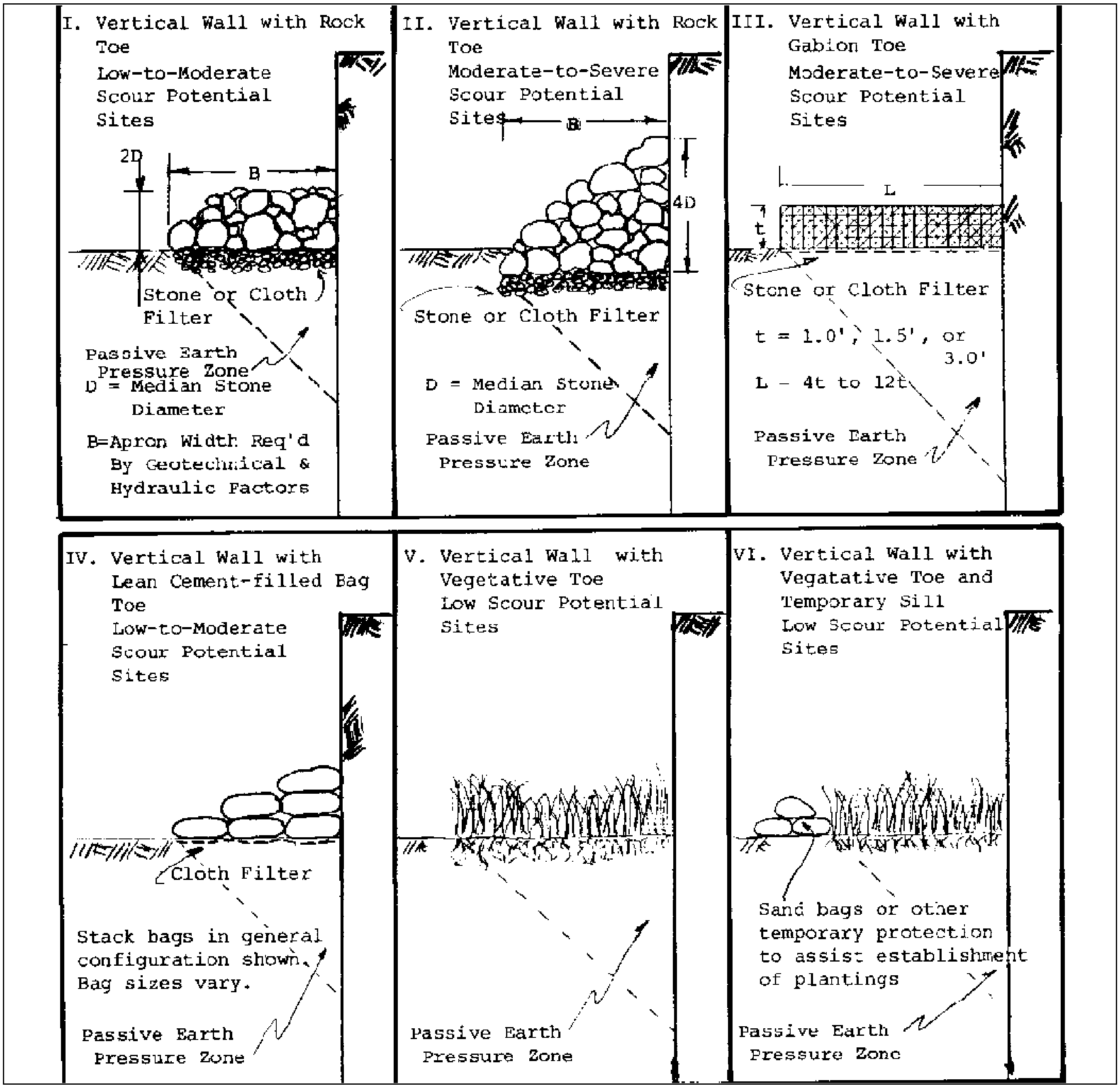


Figure 2-5. Seawall and bulkhead toe protection

where the stone diameter *d* can be related to the stone weight *W* through Equation 2-22 by setting *n* equal to 1.0. This is more restrictive than Equation 2-27 and provides an additional margin against variations in void sizes that may occur as the armor layer shifts under wave action. For large riprap sizes, each underlayer should meet the condition specified in Equation 2-28, and the layer thicknesses should be at least 3 median stone diameters. For armor and underlayers of uniform-sized quarrystone, the first underlayer should be at least 2 stone diameters thick, and the individual units should weigh about one-tenth the units in the armor layer. When concrete armor units with K_D > 12 are used, the underlayer should be quarrystone weighing about one-fifth of the overlying armor units.

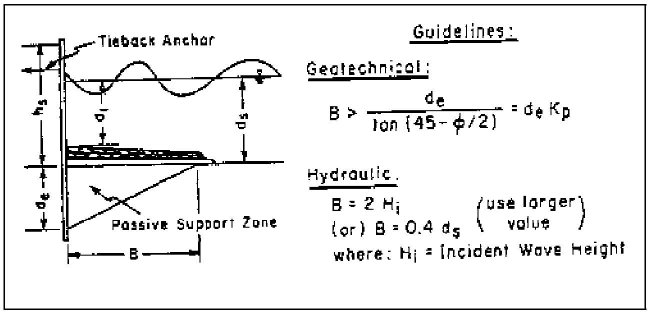


Figure 2-6. Toe aprons for sheet-pile bulkheads

c. Plastic filter fabric selection. Selection of filter cloth is based on the equivalent opening size (EOS), which is the number of the U.S. Standard Sieve having openings closest to the filter fabric openings. Material will first be retained on a sieve whose number is equal to the EOS. For granular soils with less than 50 percent fines (silts and clays) by weight (passing a No. 200 sieve), select the filter fabric by applying Equation 2-29:

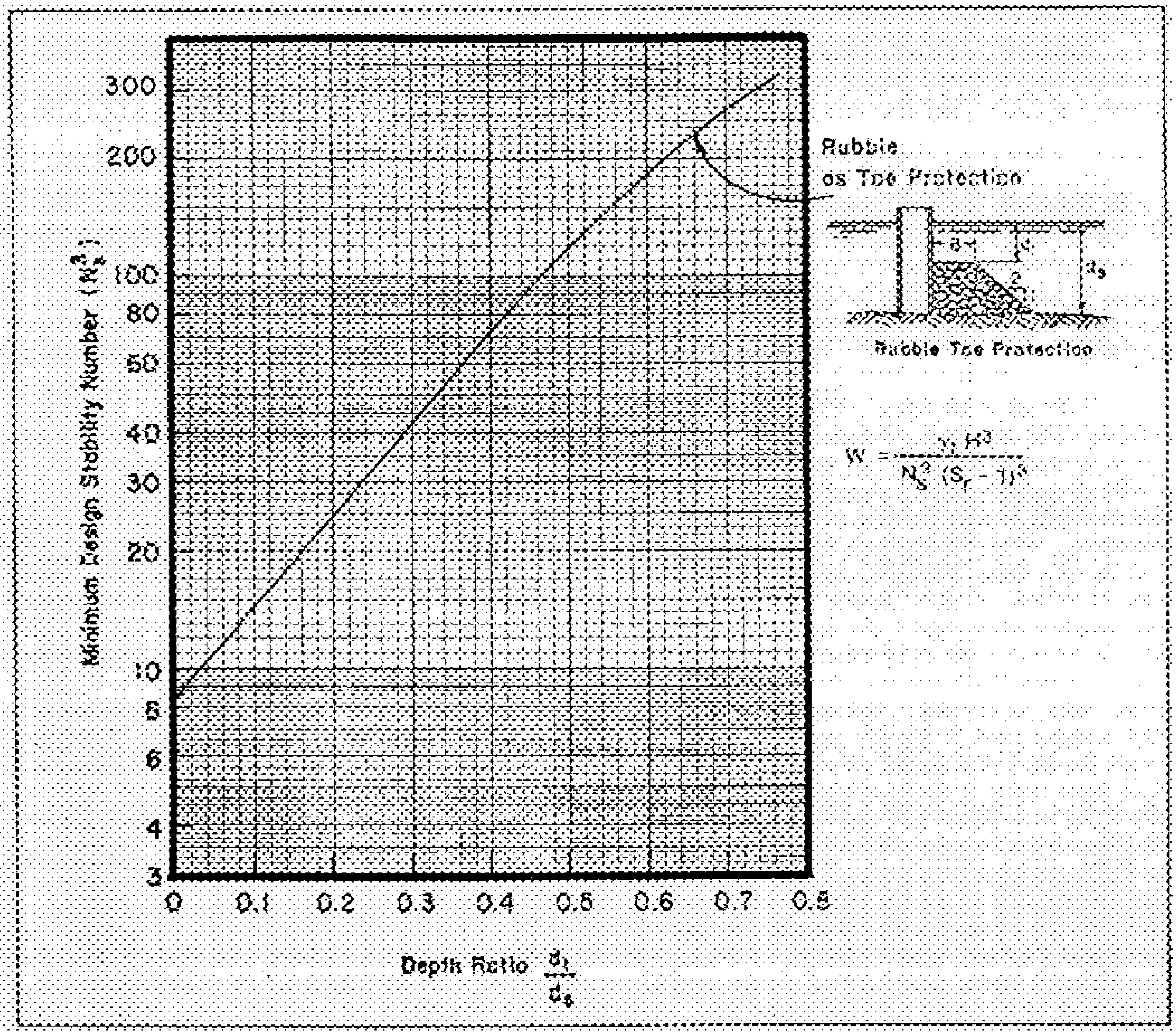


Figure 2-7. Value of N_s, toe protection design for vertical walls (from Brebner and Donnelly 1962)

$$\frac{EOS\ sieve}{d_{85\ soil}} \leq 1 \qquad (2\text{-}29)$$

For other soils, the EOS should be no larger than the openings in a No. 70 sieve. Furthermore, no fabric should be used whose EOS is greater than 100, and none should be used alone when the underlying soil contains more than 85 percent material passing a No. 200 sieve. In those cases, an intermediate sand layer may provide the necessary transition layer between the soil and the fabric. Finally, the gradient ratio of the filter fabric is limited to a maximum value of three. That is, based on a head permeability test, the hydraulic gradient through the fabric and the 1 in. of soil adjacent to the fabric (i_1) divided by the hydraulic gradient of the 2 in. of soil between 1 and 3 in. above the fabric (i_2) is:

$$Gradient\ ratio = \frac{i_1}{i_2} \leq 3 \qquad (2\text{-}30)$$

Studies such as those in Chen et al. (1981) suggest that these filter cloth selection requirements may be somewhat restrictive.

d. Filter fabric placement. Experience indicates that synthetic cloths can retain their strength even after long periods of exposure to both salt and fresh water. To provide good performance, however, a properly selected cloth should be installed with due regard for the following precautions. First, heavy armor units may stretch the cloth as they settle, eventually causing bursting of the fabric in tension. A stone bedding layer beneath armor units weighing more than 1 ton for above-water work (1.5 tons for underwater construction) is suggested (Dunham and Barrett 1974), and multiple underlayers may be needed under primary units weighing more than 10 tons. Filter guidance must be properly applied in these cases. Second, the filter cloth should not extend seaward of the armor layer; rather, it should terminate a few feet landward of the armor layers as shown in Figure 2-8. Third, adequate overlaps between sheets must be provided. For lightweight revetments this can be as little as 12 in. and may increase to 3 ft for larger underwater structures. Fourth, sufficient folds should be included to eliminate tension and stretching under settlement. Securing pins with washers is also advisable at 2-to 5-ft intervals along the midpoint of the overlaps. Last, proper stone placement requires beginning at the toe and proceeding up

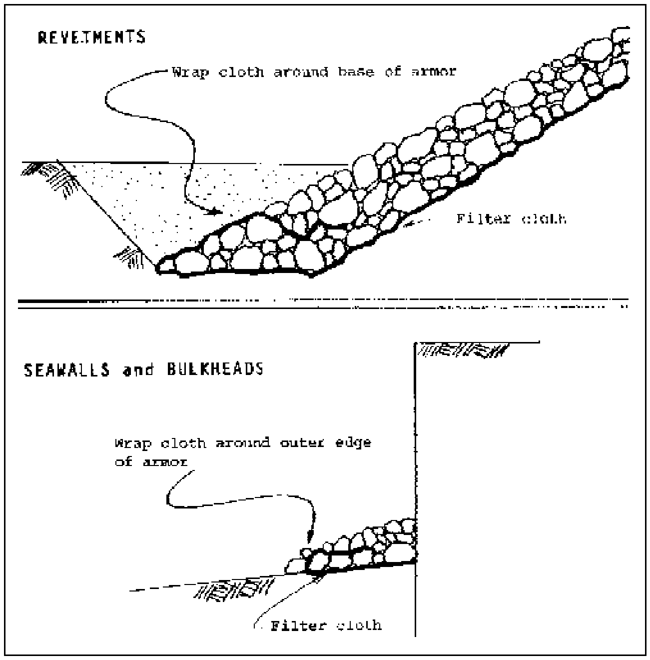


Figure 2-8. Use of filter cloth under revetment and toe protection stone

the slope. Dropping stone can rupture some fabrics even with free falls of only 1 ft, although Dunham and Barrett (1974) suggest that stones weighing up to 250 lb can safely be dropped from 3 ft. Greater drop heights are allowable under water where blocks up to 1 ton can be dropped through water columns of at least 5 ft.

2-21. Flank Protection

Flank protection is needed to limit vulnerability of a structure from the tendency for erosion to continue around its ends. Return sections are generally needed at both ends to prevent this. Sheet-pile structures can often be tied well into existing low banks, but the return sections of other devices such as rock revetments must usually be progressively lengthened as erosion continues. Extension of revetments past the point of active erosion should be considered but is often not feasible. In other cases, a thickened end section, similar to toe protection, can be used when the erosion rate is mild.

2-22. Corrosion

Corrosion is a primary problem with metals in brackish and salt water, particularly in the splash zone where materials are subjected to continuous wet-dry cycles. Mild carbon steel, for instance, will quickly corrode in such conditions. Corrosion-resistant steel marketed under various trade names is useful for some applications. Aluminum sheetpiling can be substituted for steel in some places. Fasteners should be corrosion-resistant materials such as stainless or galvanized steel, wrought iron, or nylon. Various protective coatings such as coal-tar epoxy can be used to treat carbon steel. Care must always be taken to avoid contact of dissimilar metals (galvanic couples). The more active metal of a galvanic couple tends to act as an anode and suffers accelerated corrosion. The galvanic series of common metals in seawater is given in Table 2-6 (Uhlig 1971). This table can be used for estimating the corrosion potential of galvanic couples, but the complexity of corrosion processes makes it useful only as guide. For example, although aluminum and copper are closer together on the table than aluminum and stainless steel, in actual practice polarization effects with stainless steel make it more compatible with aluminum than aluminum copper couples. The Construction Engineering Research Laboratory (CERL) should be contacted when either performance or longevity is a significant requirement.

2-23. Freeze-Thaw Cycles

Concrete should be designed for freeze-thaw resistance (as well as chemical reactions with salt water), as concrete may seriously degrade in the marine environment. Guidance on producing suitable high quality concrete is presented in EM 1110-2-2000 and Mather (1957).

Table 2-6
Galvanic Series in Sea Water

	MATERIAL	MATERIAL (≈ ACTIVITY)
MORE ACTIVE	Magnesium Stainless steel - 316 AS	Stainless steel - 304 AS
	Zinc	Lead
	Tin	
	Aluminum 52S4	
	Aluminum 4S	Magnesium bronze
	Aluminum 3S	Naval brass
	Aluminum 2S	
	Aluminum 53S-T	Nickel AS
	Yellow brass	
	Aluminum bronze	
	Red brass	
	Aluminum 17S-T	Copper, silicon bronze
	Aluminum 24S-T	
	Mild steel	Composition G bronze
	Wrought iron	Composition M bronze
	Cast iron	Nickel PS
LESS ACTIVE	Stainless steel-410 AS	
	Stainless steel-304 PS	
	Stainless steel-316 PS	

AS Active state
PS Passive state

2-24. Marine Borer Activity

Timber used in marine construction must be protected against damage from marine borers through treatment with creosote and creosote coal-tar solutions or with water-borne preservative salts (CCA and ACA). In some cases, a dual treatment using both methods is necessary. Specific guidance is included in EM 1110-2-2906.

2-25. Ultraviolet Light

The ultraviolet component of sunlight quickly degrades untreated synthetic fibers such as those used for some filter cloths and sand-bags. Some fabrics can completely disintegrate in a matter of weeks if heavily exposed. Any fabric used in a shore protection project should be stabilized against ultraviolet light. Carbon black is a common stabilizing additive which gives the finished cloth a characteristic black or dark color in contrast to the white or light gray of unstabilized cloth. Even fabric that is covered by a structure should be stabilized since small cracks or openings can admit enough light to cause deterioration.

2-26. Abrasion

Abrasion occurs where waves move sediments back and forth across the faces of structures. Little can be done to prevent such damages beyond the use of durable rock or concrete as armoring in critical areas such as at the sand line on steel piles.

2-27. Vandalism and Theft

At sites where vandalism or theft may exist, construction materials must be chosen that cannot be easily cut, carried away, dismantled, or damaged. For instance, sand-filled fabric containers can be easily cut, small concrete blocks can be stolen, and wire gabions can be opened with wire cutters and the contents scattered.

2-28. Geotechnical Considerations

The stability of vertical bulkheads, particularly sheet-pile structures, requires consideration of overturning and stabilizing forces. Static forces include active soil and water pressures from the backfill, water and passive soil pressures on the seaward side, and anchor forces (when applicable). Dynamic forces are the result of wave action and seepage flow within the soil. Wave impacts increase soil pressure in the backfill and require larger resisting passive earth pressures and anchor forces to ensure stability. Seepage forces reduce passive pressures at the toe and tend to decrease factors of safety. Toe scour decreases the effective embedment of the sheetpiling and threatens toe stability of the structure. This scouring action is caused by currents along the bottom and by pressure gradients. Both of these are induced by waves on the surface. A quantitative treatment of these geotechnical considerations can be found in Richart and Schmertmann (1958).

2-29. Wave Forces

Wave forces are determined for cases of nonbreaking, breaking, or broken waves. These cases are dependent on the wave height and depth at the structure. Wave forces for a range of possible water levels and wave periods should be computed.

a. Nonbreaking waves. Current design methods apply to vertical walls with perpendicularly approaching wave orthogonals. The Miche-Rundgren method as described in the SPM should be used. Curves are given in Chapter 7 of the SPM for walls with complete or nearly complete reflection. Complex face geometries cannot be handled, but methods are described which can be used in some cases to correct for low wall heights (where overtopping occurs), oblique wave attack on perpendicular structure faces, and walls on rubble bases.

b. Breaking waves. Breaking waves on vertical structures exert high, short-duration impulses that act in the region where the wave hits the structure. The method developed by Minikin as described in the SPM is recommended, particularly, for rigid structures such as sheet-pile structures or concrete gravity-type structures with pile supports. The Minikin method can yield extremely high wave forces compared to nonbreaking waves. This sometimes requires the exercise of proper judgment by the designer. Curves are given in the SPM to correct for low wall heights. For semirigid structures such as gravity-type seawalls on rubble foundations Equation 2-31 is recommended. Equation 2-31 was developed from Technical Standards for Port and Harbour Facilities in Japan (1980).

$$F = \frac{1}{2}\left[d_s \left(P_1 + P_2 \right) + h_c \left(P_1 + P_4 \right) \right] \tag{2-31}$$

The total force, F, per unit length of the structure, includes both the hydrostatic and dynamic force components. Figure 2-9 illustrates the pressure distribution on the face of the structures due to the breaking waves. The key pressure components can be determined by:

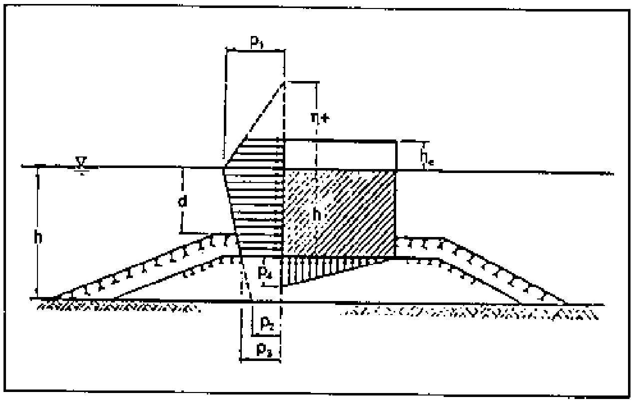


Figure 2-9. Breaking wave pressures on a vertical wall

$$P_1 = (\alpha_1 + \alpha_2)\gamma_w H_b \tag{2-32}$$

$$P_3 = \alpha_3 P_1 \tag{2-33}$$

$$P_4 = \left(1 - \frac{h_c}{1.5\,H_b}\right)P_1 \tag{2-34}$$

where

$$\alpha_1 = 0.6 + \frac{1}{2}\left[\frac{4\pi h/L}{\sinh\,(4\pi h/L)}\right]^2 \tag{2-35}$$

$$\alpha_2 = \min\left[\left(\frac{h_b - d}{3h_b}\right)\left(\frac{H_b}{d}\right)^2 ,\ \frac{2d}{H_b}\right] \tag{2-36}$$

$$\alpha_3 = 1 - \frac{d_s}{h}\left[1 - \frac{1}{\cosh\left(\frac{2\pi h}{L}\right)}\right] \tag{2-37}$$

where

γ_w = specific weight of water

h_c = height of crest of caisson above swl

d = depth at top of rubble mound

d_s = depth at base of caisson

H_b = highest of the random waves breaking at a distance of $5H_s$ seaward of the structure; H_s is the significant wave height of the design sea state

h_b = water depth where H_b is determined

h = water depth at toe of compound breakwater

L = wave length calculated by linear wave theory at the structure for wave period of H_s

As an example, for a vertical wall, 4.3 m (14 ft) high sited in sea water with d_s = 2.5 m (8.2 ft) on a bottom slope of 1:20 (m = 0.05) and experiencing wave crests at an interval of 10 sec, the force on the wall would be determined as follows:

Since there is no rubble-mound base, the water depth d_s = 2.5 m. Using a wave period T = 10 sec and Figure 7-4 of the SPM, the breaking wave height, H_b, is found to be 3.2 m (10.5 ft). Without knowledge of the significant wave height, H_s, the breaking depth, h_b, is determined directly by using SPM Figure 7-2, which yields h_b = 3.07 m (10 ft). The wave breaks at a distance of 11.4 m (37 ft) [(3.07 - 2.5)/0.05] from the wall. Using SPM Appendix C Table C-1, wave length, L, at d_s = 2.5 m is determined to be 48.7 m (160 ft). Then, α_1, α_2, and α_3 are calculated to be 1.036, 0.101, and 0.950, respectively. Crest height, h_c, is less than 1.5 H_b (1.8<4.8) and overtopping exists. The pressure components P_1, P_3, and P_4 are computed from the above equations to be 36.4 kN/m^2 (1,742.8 lb/ft^2), 34.6 kN/m^2 (1656.6 lb/ft^2), and 22.8 kN/m^2 (1,091.7 lb/ft^2), respectively. Equation 3-31 yields a total horizontal force due to the breaking wave of 142 kN/m^2 (6,799 lb/ft^2).

c. Broken waves. Some structures are placed in a position where only broken waves can reach them. In those cases approximate broken wave force, F, per unit length of structure can be estimated (Camfield 1991) by Equation 2-38:

$$F = 0.18\ \gamma\ H_b^2\left(1 - \frac{X_1\ m}{R_A}\right)^2 \tag{2-38}$$

where γ is the specific weight of water and m is the beach slope (m=tan θ). Other variables of Equation 2-38, H_b, X_1, and R_A are defined in Figure 2-10. The adjusted

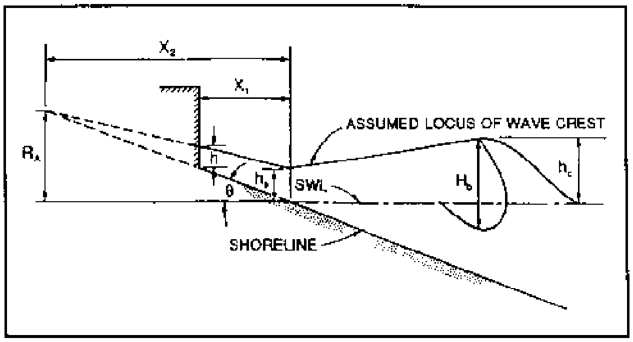


Figure 2-10. Wave pressure from broken waves

wave runup height, R_A, which would occur if the wall was not present can be determined by using Equation 2-6 (rough slopes) or following the methods described in Chapter 2-13 for smooth slopes or slopes covered with rubble other than quarrystone. If accurate force estimates are needed, model tests are required.

For example, deepwater waves are H_{mo} = 0.91 m (3 ft) and T_p = 12 sec. The waves cross 3.05 m (10 ft) of cobble shoreline with a slope of m = 0.10 before impacting on a wall. From Figure 7-3 in SPM (1984), breaking wave height H_b is 2.05 m (6.75 ft). Using Equation 2-7 we find ξ = 1.57, and Equation 2-6 yields R_{max} = 1.36 m (4.48 ft). Use R_{max} for the adjusted runup, R_A, in Equation 2-38 to find the force per unit length of wall is 4.58 kN/m length of wall (317 lb/ft length of wall).

2-30. Impact Forces

Impact forces constitute an important design consideration for shore structures because high winds can propel small pleasure craft, barges, and floating debris and cause great impact forces on a structure. If site or functional conditions require the inclusion of impact forces in the design, other measures should be taken to limit the depth of water against the face of the structure by providing a rubble-mound absorber against the face of the wall or a partly submerged sill seaward of the structure that will ground floating masses and eliminate the potential hazard. In many areas impact hazards may not occur, but where the potential exists (as for harbor structures), impact forces should be evaluated from impulse-momentum considerations.

2-31. Ice Forces

a. General. Ice can affect marine structures in a number of ways. Moving surface ice can cause significant crushing and bending forces as well as large impact loadings. Vertical forces can be caused by the weight of ice on structures at low tide and by buoyant uplift at high tide of ice masses frozen to structural elements. EM 1110-2-1612 should be reviewed before designing any structure subject to ice forces.

b. Damages. Ice formations can cause considerable damage to shoreline at some points, but their net effects are largely beneficial. Spray “freezes” on banks and structures and covers them with a protective layer of ice. Ice piled on shore by wind and wave action does not generally cause serious damage to beaches, bulkheads, or protective riprap, but it provides additional protection against severe winter waves. Some abrasion of timber or concrete structures may be caused, and individual members may be broken or bent by the weight of the ice mass. Piling is sometimes slowly pulled by the repeated lifting effect of ice frozen to the piles or attached members, such as wales, and then it is forced upward by a rise in water stage or wave action. Superstructure damages also sometimes occur due to ice.

2-32. Hydraulic Model Tests

The guidance contained in this manual is suitable for preliminary design of all coastal structures and for final design of minor or inexpensive works where the consequences of failure are not serious. For most cases, however, the final design should be verified through a model testing program. Design deficiencies can be identified with such models, and design economics may be achieved which more than offset the cost of the study. Hudson et al. (1979) contains information on current hydraulic modeling techniques.

2-33. Two-Dimensional Models

Two-dimensional tests are conducted in wave tanks or flumes. Such tests are useful for evaluating toe stone and armor stability, wave runup heights, and overtopping potential. Generated waves may be either monochromatic or irregular depending on the capabilities of the equipment. Monochromatic waves represent the simplest case, and they form the basis for the majority of current design guidance. Irregular waves, on the other hand, are a closer representation of actual prototype conditions. Their use, however, adds to the complexity of a modeling program.

2-34. Three-Dimensional Models

Three-dimensional models are built in large shallow basins where processes such as wave refraction and diffraction are of interest. They can also lead to qualitative results for sediment transport studies. However, these issues are generally unimportant for the design of revetments, seawalls, and bulkheads; therefore, the use of three-dimensional models would be unusual for such structures.

2-35. Previous Tests

WES has conducted a number of two- and three-dimensional model studies of site-specific projects. Details on five of these are given below. Units are given in prototype dimensions.

a. Fort Fisher NC (1982). Important features were (Markle 1982):

Scale	1:24
Waves	Heights of 5.5 to 17.2 ft Periods of 8, 10, and 12 sec
Depths	12, 14.7, 17, and 19 ft
Revetment slope:	1:2

The toe consisted of 8,919-lb StaPods on bedding stone. The sizes of the armor units were 5,900 lb (specially placed) and 8,900 lb (randomly placed). These were stable and undamaged in depths to 14.7 ft. At depths of 17 and 19 ft, considerable damages were experienced, but no failures occurred.

b. El Morro Castle, San Juan, PR (1981). Important features were (Markle 1981):

Scale	1:38.5
Waves	Heights of 10 to 23.3 ft Periods of 15 and 17 sec (north revetment)
	Heights of 2.5 to 10.5 ft Periods of 9, 15, and 17 sec (west revetment)
	18 and 19.9 ft (north revetment)
	13 and 14.9 ft (west revetment)
Revetment slope:	1:3

The toe protection was generally a 10-ft-wide armor stone blanket except in certain areas of the north revetment where a low-crested breakwater was used. Armor stone sizes were 10,300 lb (west revetment), 24,530 lb (north revetment), and 9,360 lb (north revetment behind breakwater). All armor stone was randomly placed.

c. Generalized harbor site for the U.S. Navy (1966). Important features were (USAEWES 1966):

Scale	1:15
Waves	Heights of 5, 10, 15, and 20 ft 10-sec periods
Depths	20 to 40 ft
Revetment slope:	1:5

No toe protection was provided (the toe extended to the flume bottom). Stable rock sizes and values of K_d were reported for several wave conditions.

d. Railroad fills at Ice Harbor and John Day Reservoirs (1962). The tests were conducted for both riprap stability and runup. Important features were (USAEWES 1962):

Scale	1:12
Waves	Height of 2.4 to 2.6 ft Periods of 3, 4, 5, 6, and sec
Depths	20 to 40 ft
Revetment slope:	1:2

No toe protection was provided. The stable W_{50} sizes were

W_{50}	H
300 lb	3.0 to 3.4 ft
500 lb	2.0 to 4.1 ft
700 lb	3.9 to 4.9 ft

e. Levees in Lake Okeechobee, FL (1957). The tests were conducted for both wave runup and overtopping. Important features were (USAEWES 1957):

Scale	1:30 and 1:17
Waves	Heights of 4, 6, 8, and 12 ft Periods of 4.5 to 7 sec
Depths	10, 17.5, and 25 ft

Revetment slope: 1:3, 1:6, and composite slopes

No toe protection was considered. The tests produced a series of runup and overtopping volume curves.

Chapter 3
Revetments

3-1. General

A revetment is a facing of erosion resistant material, such as stone or concrete, that is built to protect a scarp, embankment, or other shoreline feature against erosion. The major components of a revetment are the armor layer, filter, and toe (Figure 3-1). The armor layer provides the basic protection against wave action, while the filter layer supports the armor, provides for the passage of water through the structure, and prevents the underlying soil from being washed through the armor. Toe protection prevents displacement of the seaward edge of the revetment.

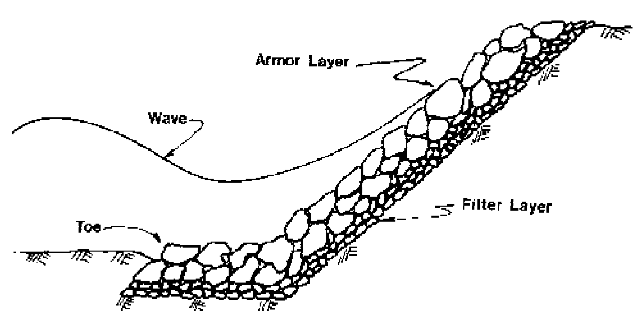


Figure 3-1. Typical revetment section

3-2. Armor Types

Revetment armoring may range from rigid to flexible types. Concrete slabs-on-grade is an example of the former, while riprap and quarrystone are examples of the latter. Rigid armors tend to be more massive but are generally unable to accommodate settlement or adjustments of the underlying materials. Flexible armor is constructed with lighter individual units that can tolerate varying amounts of displacement and shifting. Details of individual armor types are presented in Appendix B. The individual alternatives discussed in Appendix B are summarized in Figure 3-2.

3-3. Design Procedure Checklist

The usual steps needed to design an adequate revetment are:

a. Determine the water level range for the site (paragraph 2-5).

b. Determine the wave heights (paragraphs 2-6 to 2-11).

c. Select suitable armor alternatives to resist the design wave (Appendix B).

d. Select armor unit size (paragraphs 2-15 to 2-18).

e. Determine potential runup to set the crest elevation (paragraphs 2-12 and 2-13).

f. Determine amount of overtopping expected for low structures (paragraph 2-14).

g. Design underdrainage features if they are required.

h. Provide for local surface runoff and overtopping runoff, and make any required provisions for other drainage facilities such as culverts and ditches.

i. Consider end conditions to avoid failure due to flanking (paragraph 2-21.

j. Design toe protection (paragraph 2-19).

k. Design filter and underlayers (paragraph 2-20).

l. Provide for firm compaction of all fill and backfill materials. This requirement should be included on the plans and in the specifications. Also, due allowance for compaction must be made in the cost estimate.

m. Develop cost estimate for each alternative.

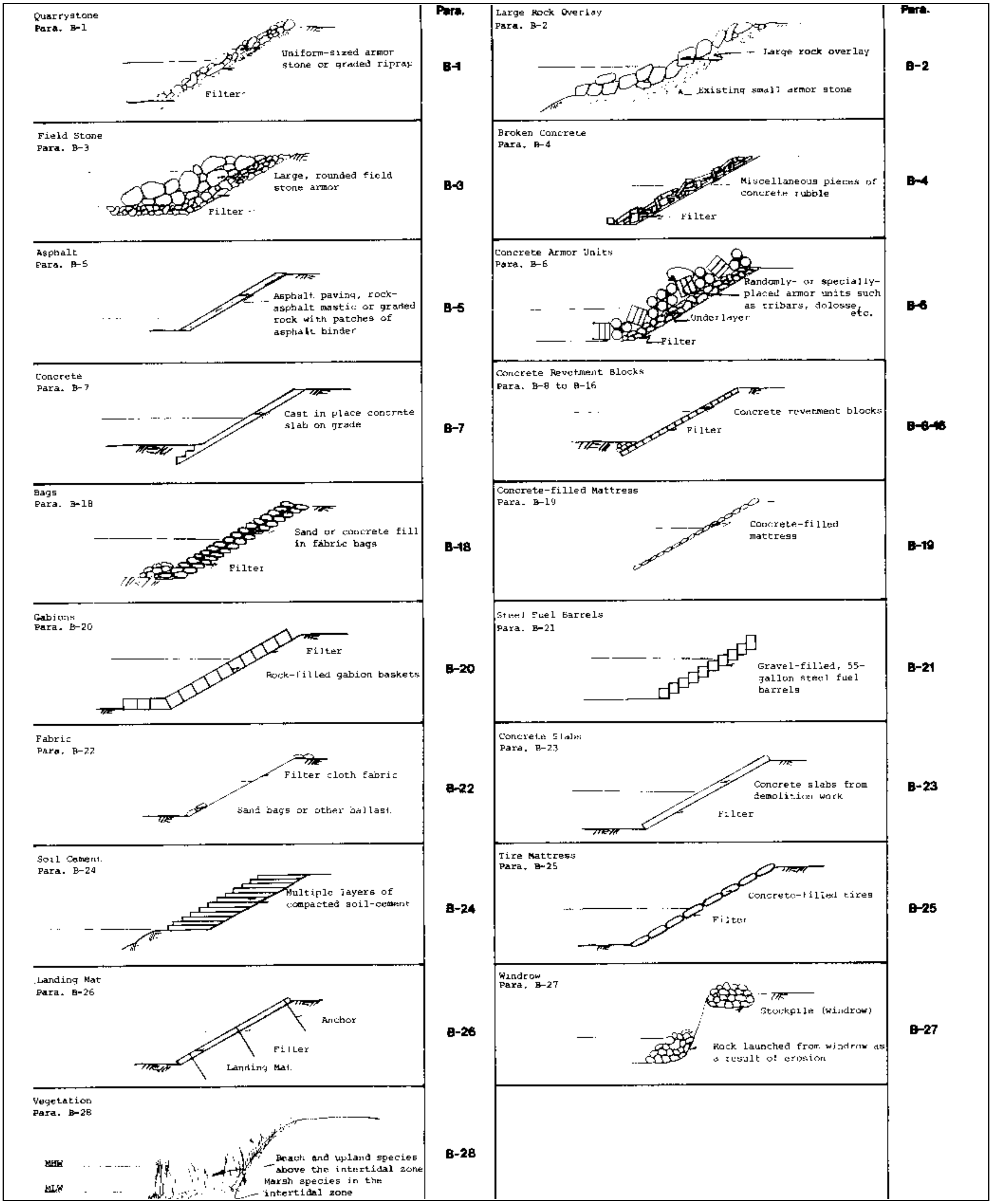


Figure 3-2. Summary of revetment alternatives

Chapter 4
Seawalls

4-1. General

A seawall is a massive structure that is designed primarily to resist wave action along high value coastal property. Seawalls may be either gravity- or pile-supported structures. Common construction materials are either concrete or stone. Seawalls can have a variety of face shapes (Figure 4-1).

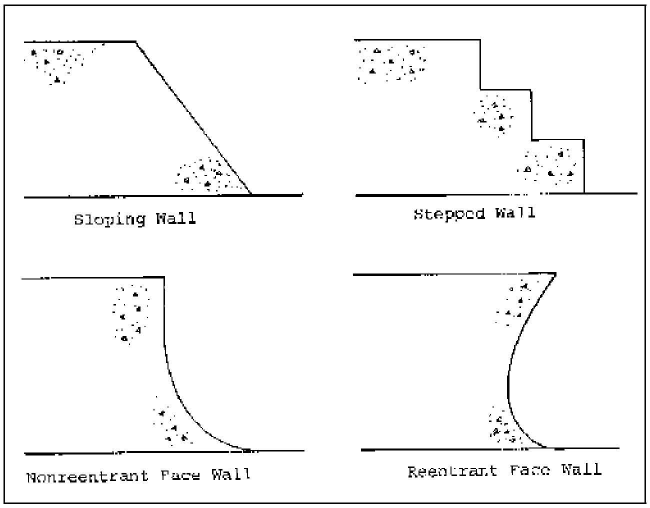


Figure 4-1. Typical concrete seawall sections

4-2. Concrete Seawalls

These structures are often pile-supported with sheetpile cutoff walls at the toe to prevent undermining. Additional rock toe protection may also be used. The seaward face may be stepped, vertical, or recurved. Typical examples are described in Appendix C and shown in Figure 4-2.

4-3. Rubble-Mound Seawalls

These are designed like breakwaters using a rock size that will be stable against the design wave. Stability is determined using the method described in paragraphs 2-15 to 2-18. An example is described in Appendix C and shown in Figure 4-2.

4-4. Design Procedure Checklist

The most critical design elements are a secure foundation to minimize settlement and toe protection to prevent undermining. Both of these are potential causes of failure of such walls. The usual steps needed to develop an adequate seawall design follow.

a. Determine the water level range for the site (paragraph 2-5).

b. Determine the wave heights (paragraphs 2-6 to 2-11).

c. Select suitable seawall configurations (Appendix C).

d. Design pile foundations using EM 1110-2-2906.

e. Select a suitable armor unit type and size (rubble seawalls and toe protection) (paragraphs 2-15 to 2-18).

f. Determine the potential runup to set the crest elevation (paragraphs 2-12 to 2-13).

g. Determine the amount of overtopping expected for low structures (paragraph 2-14).

h. Design underdrainage features if they are required.

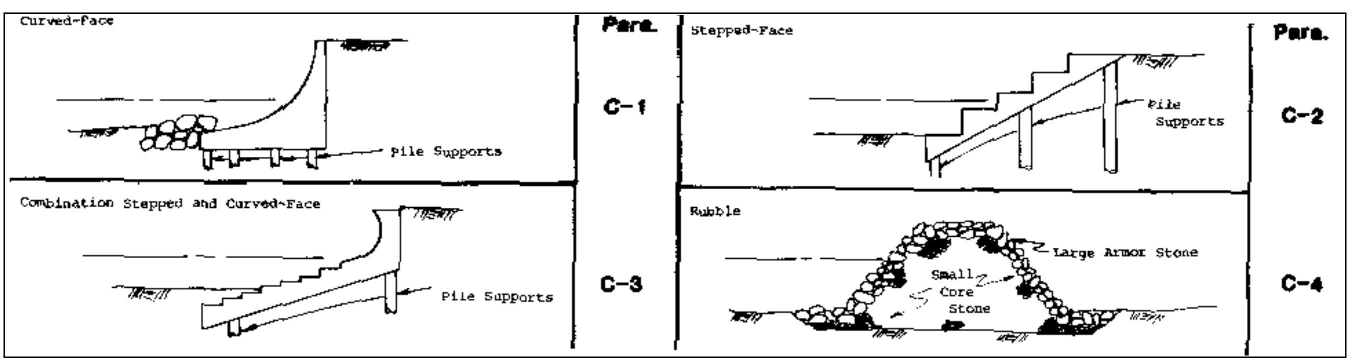


Figure 4.2. Summary of seawall alternatives

i. Provide for local surface runoff and overtopping and runoff, and make any required provisions for other drainage facilities such as culverts and ditches.

j. Consider end conditions to avoid failure due to flanking (paragraph 2-21).

k. Design the toe protection (paragraph 2-19).

l. Design the filter and underlayers (paragraph 2-20).

m. Provide for firm compaction of all fill and backfill materials. This requirement should be included on the plans and in the specifications, and due allowance for compaction must be made in the cost estimate.

n. Develop cost estimate for each alternative.

Chapter 5
Bulkheads

5-1. General

Bulkheads are retaining walls whose primary purpose is to hold or prevent the backfill from sliding while providing protection against light-to-moderate wave action. They are used to protect eroding bluffs by retaining soil at the toe, thereby increasing stability, or by protecting the toe from erosion and undercutting. They are also used for reclamation projects, where a fill is needed seaward of the existing shore, and for marinas and other structures where deep water is needed directly at the shore.

5-2. Structural Forms

Bulkheads are either cantilevered or anchored sheetpiling or gravity structures such as rock-filled timber cribbing. Cantilevers require adequate embedment for stability and are usually suitable where wall heights are low. Toe scour reduces their effective embedment and can lead to failure. Anchored bulkheads are usually used where greater heights are necessary. Such bulkheads also require adequate embedment for stability but are less susceptible to failure due to toe scour. Gravity structures eliminate the expense of pile driving and can often be used where subsurface conditions hinder pile driving. These structures require strong foundation soils to adequately support their weight, and they normally do not sufficiently penetrate the soil to develop reliable passive resisting forces on the offshore side. Therefore, gravity structures depend primarily on shearing resistance along the base of the structure to support the applied loads. Gravity bulkheads also cannot prevent rotational slides in materials where the failure surface passes beneath the structure. Details of typical bulkheads are presented in Appendix D and are summarized in Figure 5-1.

5-3. Design Procedure Checklist

The bulkhead design procedure is similar to that presented for seawalls in paragraph 4-4, except that Appendix D is used for examples of typical bulkheads. In addition, toe protection should be designed using geotechnical and hydraulic conditions, including wave action and current scour.

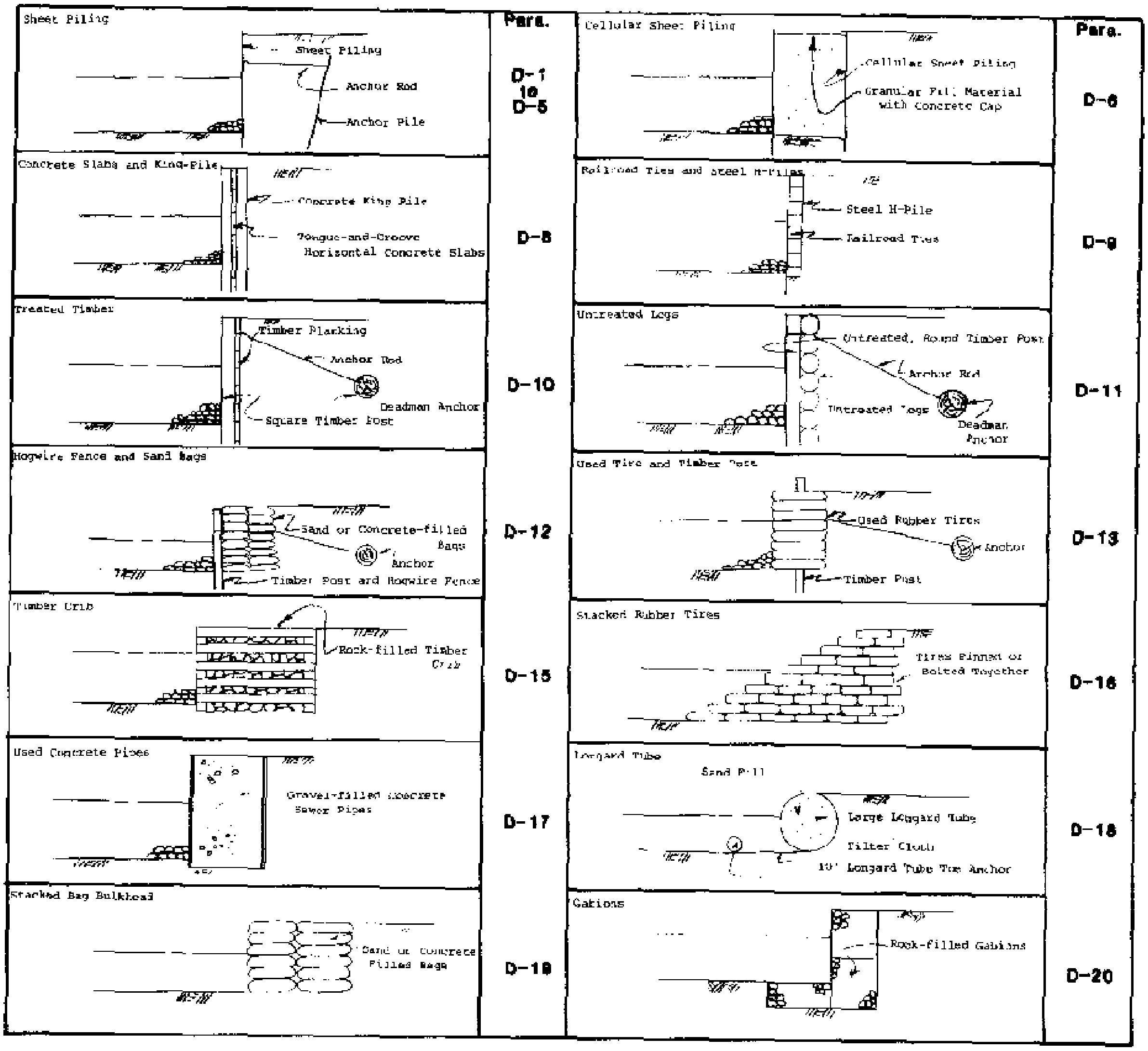


Figure 5-1. Summary of bulkhead alternatives

Chapter 6
Environmental Impacts

6-1. General

Coastal shore protection structures are intended to improve stability by reducing the rate of change in a dynamic coastal system. The environmental impacts may be short-term during construction operations or long-term because of the presence of the structures. The potential environmental impacts, which are similar for each of the coastal shore protection structures featured in this manual, are briefly discussed below. More detailed information may be found in Barnard (1978), Carstea et al. (1975a; 1975b), Ford et al. (1983), Hurme (1979), Johnson and DeWitt (1978), and Mulvihille et al. (1980).

6-2. Physical Impacts

The littoral system at the site of a structure is always moving toward a state of dynamic equilibrium where the ability of waves, currents, and winds to move sediment is matched by the available supply of littoral materials. When there is a deficiency of material moving within a system, the tendency will be for erosion at some location to supply the required material. Once a structure has been built along a shoreline, the land behind it will no longer be vulnerable to erosion (assuming proper function of the structure), and the contribution of littoral material to the system will be diminished along the affected shoreline. The contribution formerly made by the area must now be supplied by the adjoining areas. This can have mixed environmental impacts. The reduction in sedimentation due to decreased erosion may be viewed as a positive effect in many cases. Erosion that is shifted to other areas may result in a negative impact in those locations. Some vertical structures such as bulkheads may cause increased wave reflection and turbulence with a subsequent loss of fronting beach. This is usually viewed as a negative impact. In all cases, the overall situation and the various impacts that result must be evaluated carefully to identify potential changes in the shore and barrier island processes.

6-3. Water Quality Impacts

Impacts of coastal shore protection structures on water quality can be addressed in two categories:

a. Increased suspended solids during construction.

b. Altered circulation caused by structures.

Construction of shore protection structures can result in increased suspended solid loads within the adjoining water body. Recent research results indicate that the traditional fears of water quality degradation caused from suspended solids during in-water construction activities are for the most part unfounded. It has been demonstrated that the increased concentration of suspended solids is generally confined to the immediate vicinity of the construction activity and dissipates rapidly at the completion of the operation. Although these are generally short-term impacts, construction activities should be designed to minimize generation of suspended solids. The dispersion of near-surface suspended solids can be controlled, to a certain extent, by placing a silt curtain around the construction activity. Under quiescent current conditions (less than 0.1 knot) the suspended solids level in the water column outside the curtain can be reduced by as much as 80 to 90 percent. Silt curtains are not recommended where currents exceed 1 knot. Steps must be taken also to avoid the introduction of toxic or other harmful substances resulting from construction materials, equipment leaks, spills, and other accidents. Project specifications should contain provisions that address these concerns. Structures may influence water quality by altering circulation patterns. Modification in circulation may result in changes in the spatial distribution of water quality constituents, differences in the flushing rates of potential contaminants, and changes in the scour patterns and deposition of sediments. Environmental assessment of the effects on circulation should initially emphasize the physical parameters such as salinity, temperature, and velocity. If minimal changes occur in these parameters, then it can be assumed that the chemical characteristics of the system will not be significantly modified. Prediction of changes in circulation and its effect on the physical parameters can be achieved through comparison with existing projects, physical model studies, and numerical simulation.

6-4. Biological Impacts

A wide variety of living resources is present in coastal shore protection project areas and includes species of commercial, recreational, and aesthetic importance. Because shore protection projects exist in arctic, temperate, and tropical climates, biological impacts will generally be highly site-specific and depend upon the nature and setting of the project. The environmental impacts on the benthic communities resulting from suspended solids in the water around shore protection construction are for the most part minor. This is particularly true in the surf zone on open coast beaches where rapid natural changes and disturbances are normal and where survival of the benthic community requires great adaptability. Placement of coastal shore protection structures requires an initial disturbance of the benthic substrate, but it results in the formation of a new substrate composed of structural material and stability of the sediments adjacent to the structure. In many locations the placement of these structures provides new habitat not available otherwise.

6-5. Short-term Impacts

Short-term impacts are usually associated with the actual construction phase of the project. The actual time is typically short (measured in days and weeks) and, therefore, can be scheduled to minimize negative impacts. Transportation of material to the site, preparation and construction using heavy equipment, and back filling and grading will cause temporary air and noise pollution close to the site. Nesting, resting, or feeding waterfowl and fish and other wildlife will be disrupted. Projects should be timed, if possible, to avoid waterfowl and turtle nesting periods and fish spawning periods. Temporarily reduced water quality, discussed in paragraph 6-3, may have biological impacts. However, if the bank is severely eroding or is heavily developed these impacts may be minimal by comparison. Siltation of offshore sea grasses or corals as the result of construction, dredging, and filling at the site may be of short or long duration depending on the composition of the sediment, the currents, and circulation patterns at the site and the locations of these specific resources. Construction impacts at sites with a high percentage of fine material and nearby sea grass bed or corals could be high and require special planning and precautions such as silt curtains. Dredging activities may attract opportunistic foraging fish as well as temporarily destroy benthic habitats. Resuspension of bottom sediments may interfere with respiration and feeding, particularly of nonmotile bottom dwellers. Motile organisms will temporarily flee the disturbed area.

6-6. Long-term Impacts

Long-term effects vary considerably depending upon the location, design and material used in the structure. The impact of a vertical steel sheet bulkhead located at mean low water in a freshwater marsh will be considerably different from a rubble-reveted bank in an industrialized harbor. Vertical structures in particular may accelerate erosion of the foreshore and create unsuitable habitat for many bottom species in front of the structure as the result of increased turbulence and scour from reflected wave energy. On the other hand, rubble toe protection or a riprap revetment extending down into the water at a sloping angle will help dissipate wave energy and will provide reef habitat for many desirable species. Bulkheads and revetments can reduce the area of the intertidal zone and eliminate the important beach or marsh habitat between the aquatic and upland environment. This can also result in the loss of spawning, nesting, breeding, feeding, and nursery habitat for some species. However, birds such as pelicans might benefit. A number of design alternatives should be considered to maximize biological benefits and minimize negative impacts. Table 6-1 summarizes design considerations for improving the environmental quality of these structures.

6-7. Socioeconomic and Cultural Impacts

Secondary impacts are often more controversial than the primary impacts on air, water, noise, and the biota. Land use patterns will often change as the result of construction. However, only two elements normally are directly considered in the design of the structure itself. The structure should be sited to avoid known archaeological or other cultural sites. Secondly, the structure should be designed to be aesthetically pleasing. Coastal shore protection structures change the appearance of the coastline. The visual impact of a structure is dependent on how well the structure blends with its surroundings. The importance of visual impacts is related to the number of viewers, their frequency of viewing, and the overall context. For example, the appearance of a structure in a heavily used urban park is more critical than a structure in an industrial area or an isolated setting. Aesthetic impacts can be adverse or beneficial depending on preconstruction conditions and the perception of the individual observer. Coastal shore protection structures offer a visual contrast to the natural coastal environment. However, many observers prefer a structure to erosion damage. Most coastal shore protection structures improve access to the water's edge for recreation and sightseeing.

6-8. Evaluation of Alternatives

Comparison and evaluation of coastal shore protection alternatives involves examination of economic, engineering, and environmental aspects. Alternatives are evaluated according to how well they meet specified project objectives. Examples of environmental objectives include preservation, protection, and enhancement of aesthetic resources, fish and wildlife habitat, and water quality. Evaluation of the short- and long-term impacts of coastal shore protection structures requires comparison of with-project and without-project conditions. Recognizing the dynamic nature of the coastal system, a forecast must be made of future environmental conditions without the project. These predicted conditions are then compared

Table 6-1
Environmental Design Considerations for Revetments, Seawalls, and Bulkheads

Factor	Design Considerations [1]	Environmental Benefit
Location	Site structure above mean high water	Allows intertidal zone to remain Allows shoreline vegetation to remain Does not interfere with littoral drift
	Avoid wetland sites, spawning beds, shore-bird and turtle nesting beaches	Resource conservation
	Avoid nearby coral reef and seagrass beds	Resource conservation
	Avoid archaeological sites	Preserve historical information
Construction Material	Rubble or riprap	Most desirable, natural and durable
	Treated wood and smooth concrete	Intermediate desirability Less surface area
	Steel sheet pile	Least desirable, least colonizable
	Use largest cost-effective armor stone	More stable physical habitat More size diversity of openings
Design Features	Use riprap or stair-step revetments on a slope of 1 to 1 or flatter when structures are partially submerged	Dissipates more energy More habitat for fish and reef fish
	Use toe protection on structures below mean low water	More diverse habitat Reef-like properties Dissipates wave energy on the bottom
	Use sloping structures and avoid vertical structures especially when a structure is partially submerged	Wave energy not reflected
	Use floating or pile-supported structures for access to vessels	Avoids problems of vertical walls
	Use natural shoreline contours and avoid sharp angles	Aesthetically pleasing

[1] Where applicable and possible.

with the expected conditions resulting from each alternative. Environmental features should be integral parts of the project, not additions made late in design or afterward.

CHAPTER 3

SHORE PROTECTION: BREAKWATER

3.0 INRODUCTION

Aim of this paper is to outline breakwaters, considering their:

- Functions
- Types
- Design Guidelines
- Construction

Breakwaters are structures which provide protection to the harbours and structures such as sea intakes against wave action.

Basic principles are the same for revetments which are basically one sided breakwaters, normally with no overtopping.

Whilst breakwaters are basically simple in form and can be defined with a few drawings and short specifications, they can be multi-million dollar structures with very serious repercussions on failure.

3.1 FUNCTIONS

Breakwaters are constructed for the following purposes:

i. To provide a protection against waves for harbour facilities by decreasing the wave powers into the sheltered area.

ii. To maintain navigational depths for ship going into the harbour by protecting from the intrusion of littoral drift (shoaling).

iii. Control the magnitude and direction of currents

iv. Stabilize the location of an entrance

v. Coastal Protection (revetments)

vi. Often have combination of functions.

Different functions dictate different requirements for the breakwater such as:

- Permeability
- Energy absorption (i.e. reflection)
- Plan layout
- Crest level
- Roughness (in relation to flow pattern)
- Crest width

3.2 TYPES OF BREAKWATER

Types of breakwaters are described in table 3.1 below and classified according to structures or devices used:

Table 3.1: Types of breakwater

Structural Types	**Description**	**Advantages**	**Disadvantages**
Rubble Mound (Sloping)	- core of gravel or quarry run - covered by armour layer of rock or concrete units	- durable - flexible and accommodate settlement - adapts to irregular bathymetry - can cope with partial damage	- large material quantity and size - may need mattress on soft sea beds
Upright /Vertical Wall	- caisson - mass concrete block - cellular concrete block - sheet pile cells	- durable - quick construction - less material and size - easy to incorporate wharf	- need strong foundation - cannot tolerate settlement - inflexible structure, damage occur will be severe - waves reflection and scouring - vertical wall can give high impact forces
Composite	- As above but on rubble mound base except sheet pile cells	- moderate material usage	- can suffer impact forces - disastrous failure - reflected waves can damage rubble mound base

Resonant Breakwater	- Series of rectangular basins built at harbour entrance		- Not a common structural form
Rigid Floating Breakwater	- Usually temporary – large floating body such as ship or pontoon anchored in position	- easy installation - can be relocated - independent of depth - little space usage	- ineffective for long period waves - can resonate on some frequencies
Flexible Floating Breakwater	- Temporary flexible buoyant floating devices such as car tyres with some flotation-lashed together	- quick fabrication and inexpensive - independent of depth - easily relocated	- ineffective on long waves
Air Bubble Curtain	- Submerged pipeline discharging air to cause currents in the water and waves to break	- no space - no shipping restriction - quick construction	- high energy usage - only good on very short waves - can be block by sediments

As a general rule:

- Rubble mound breakwaters most durable and best suited to heavy wave attack.

- Vertical wall breakwaters have use where limited space or material available, particularly in deep water.

- Floating breakwater or bubble curtain only affect short period waves and have no effect on sediment movement.

3.3 EXAMPLES OF TYPES OF BREAKWATER

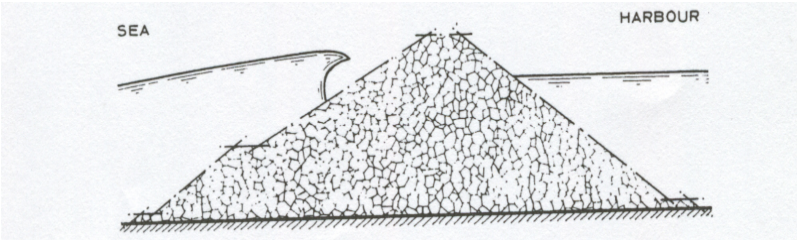


Fig 3.1: Rubble Mound (Sloping face) breakwater

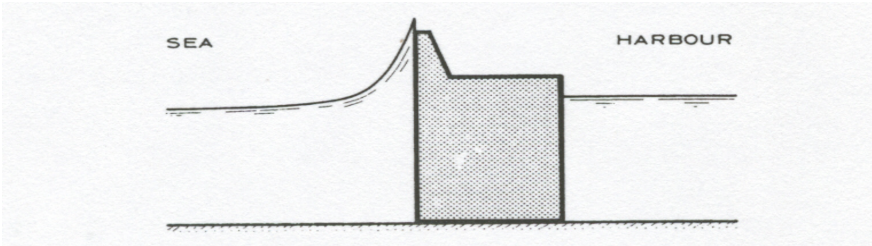


Fig 3.2: Upright / Vertical face breakwater

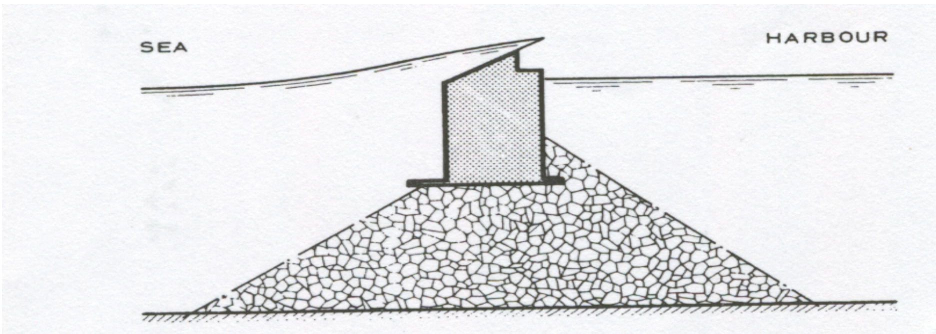


Fig 3.3: Composite face breakwater

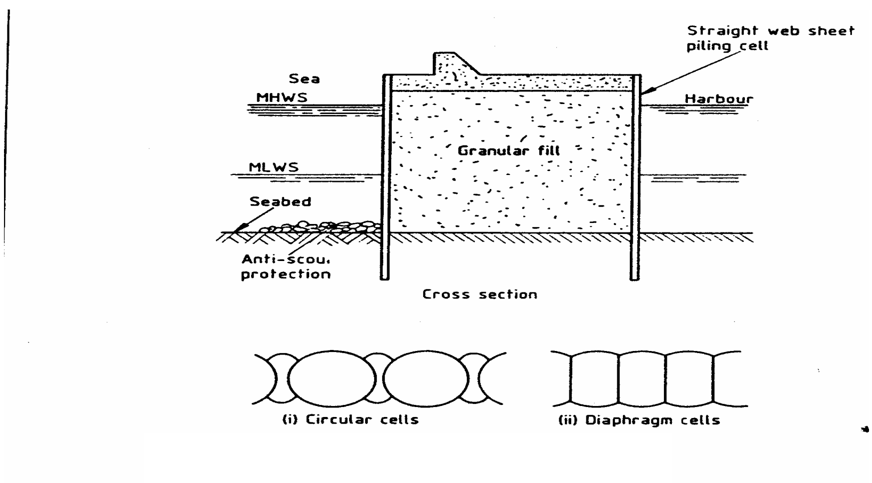


Fig 3.4: Typical Sheet Pile Cell Breakwater

3.4 DESIGN GUIDELINES

3.4.1 Principle of Design

In the design of breakwaters, the following matters shall be examined:

a. Layout of the breakwater
b. Influence of the breakwater on site selection
c. Design conditions and parameters
d. Structural types breakwater
e. Design method
f. Execution method
g. Construction cost

3.4.2 Layout of Breakwater

In determine of the layout of breakwaters, the following matters shall be examined:

a. Environmental conditions
b. Calmness in the harbour
c. Ease of ship manoeuvrability
d. Water quality in the harbour
e. Construction cost and maintenance cost
f. Future plans of the port and harbour

3.4.3 Influence of Breakwaters on Site Selection

The breakwaters may have considerable influence on the site selection because of the relatively high cost of constructing breakwaters. In order to minimize breakwater costs, the following factors should be considered:

The total length of breakwater should be as small as possible.

i. Port site selection and breakwater alignment for narrow wave sector (Fig 3.5)

   On a coast where the waves come from narrow sector only, minimum length is obtained if the port is placed at a headland with an offset of the coastline sufficiently large to allow for the necessary port area, at the same time as one relatively short breakwater can reach sufficient depth, by taking advantage of the curvature of the contour lines.

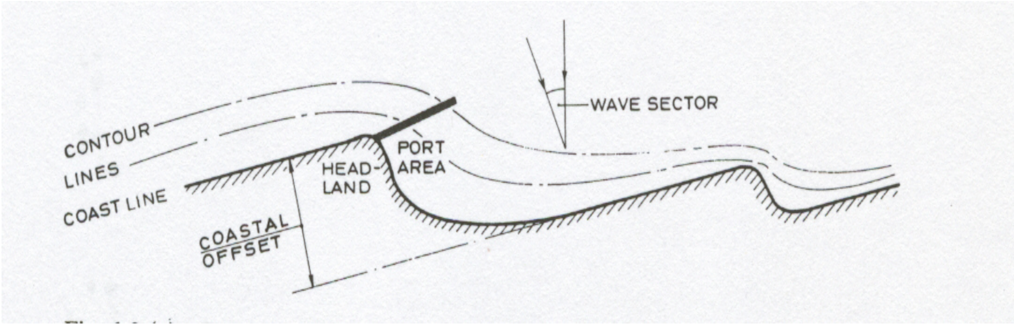


Fig 3.5: Breakwater alignment for narrow wave sector

ii. Port site selection and breakwater alignment for wide wave sector (Fig 3.6)

On the coast where the wave sector is wide the most favourable site would be between two neighbouring headlands particularly if the necessary dredging inside the port area can take place in loose sediments

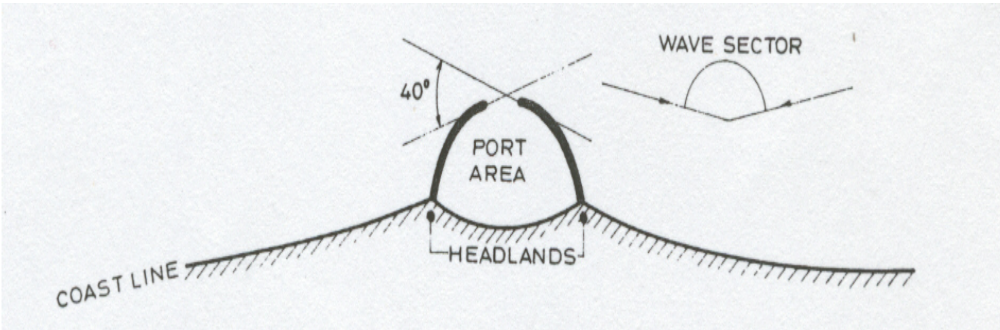


Fig 3.6: Breakwater alignment for wide wave sector

iii. Port site selection and breakwater alignment for rocky shoal and coastal ridge (Fig 3.7)

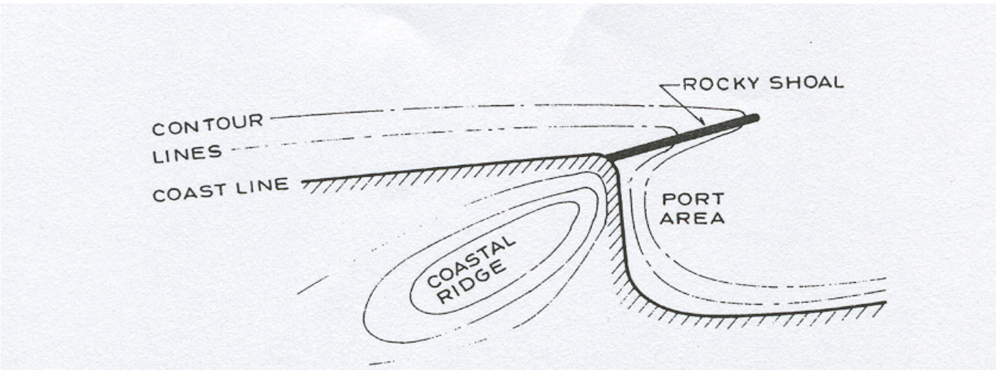


Fig 3.7: Breakwater alignment for rocky shoal and coastal ridge

3.4.4 Design Conditions and Parameters

To determine the design conditions and parameters for the design of a breakwater system, basically there are 3 main areas to be covered:

a. Define performance requirement or functional requirement

- wave levels and associated frequencies that can be tolerated within harbour area
- sizes and manoeuvrability of vessels to use harbour
- whether regular dredging can be tolerated
- whether overtopping and some wave transmission can be tolerated

b. Collect environmental data

- Bathymetry
- Water level variation (tide and storm surge)
- Wave heights and directions
- Winds
- Currents
- Sediment transport and shoreline stability
- Foundation conditions

c. Determine availability of material

3.4.5 Phases (Work Flow)

The number of phases in the design of breakwater depends upon its magnitude. For major breakwaters there are naturally more and more complicated steps in the design than for small traditional breakwaters. The following is a fairly complete list of the steps required for a major breakwater;

a. Establish design organization
b. Site selection
c. Compilation of existing climatic such as;
    - Oceanographic data
    - Hydrographic data
    - Geologic data
d. Preliminary estimate of wave conditions
e. First tentative layout
f. Wave recording during at least one year
g. Recording of long waves during periods where long wave effects may be expected
h. Tide recording for say two months if no tidal data is available
i. Topographic and hydrographic survey
j. Sedimentation study
k. Environmental study

l. Soil investigations
m. Search for construction materials
n. Hydraulic model test for determination of layout
o. Choice of breakwater type
p. Preliminary design of breakwater cross section
q. Model test for optimization of cross section and end of breakwater
r. Final detailed design

3.5 DESIGN OF A RUBBLE MOUND BREAKWATER

Having determined the performance criteria and gathered relevant data for the site, the design for breakwater can proceed:

- Preliminary design
- Final design based on model study

Major aspects of the design are:

- The layout of the breakwater in order to achieve the specified performance of wave reduction, current realignment etc. for the protected area
- The cross section

3.5.1 Alignment

Can use wave diffraction calculation initially, but need to fine tune with a model study – either a physical or numerical model.

3.5.2 Cross Section

No codes available to determine for example the design wave height and loading as with a building code:

- At a particular site a range of structures can be build for a range of costs depending on the design wave height chosen and whether overtopping will be acceptable.

- Classic optimum design curve
    - Curve 1 - construction cost
    - Curve 2 - capitalized maintenance cost
    - Curve 3 - total cost

- Clearly as design wave increases then probability of exceedance drops; but structure is larger with greater construction cost. Likelihood of damage in service is less and thus likelihood and cost of maintenance will decrease.

- Costs due to loss of facility usage must also be considered.

- Best to err on side of safety when designing breakwaters.

i. Assessment of Design Wave

Look at probability of exceedance of wave heights:

- Short Term (Micro) distribution
- Long Term (Macro) distribution

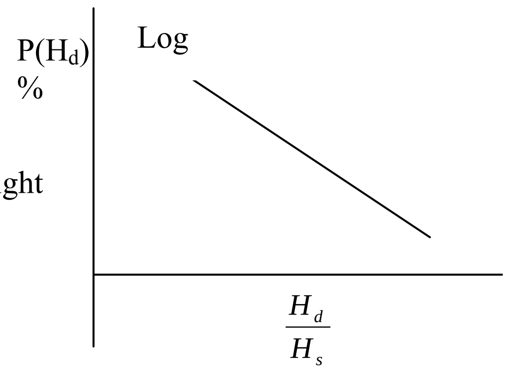
a. Short term distribution

Single storm or record containing N waves can be characterised by Hs (significant wave height) (average of 1/3 highest waves – 13% exceedance). Fortunately the distribution of waves within the storm approximately follows the Rayleigh Distribution (theoretical model).

Thus for an arbitrary design wave height H_d the chance of exceedance by any given wave is: -

$$P(Hd) \quad = e^{-2\left[\frac{H_d}{H_s}\right]^2}$$

Values of $P(H_d)$ vs $\frac{H_d}{H_s}$ plot as a straight line on log normal paper.



The chance that this wave is not exceeded is

$1-P(H_d)$

The chance that this wave is not exceeded in a series of N waves is

$[1-P(H_d)]^N$

The chance that this wave (Hd) is exceeded at least once in a single storm of H waves is

$E_l = 1 - [1- P(H_d)]^N$

The Poisson approximation is

$E_l = 1- e^{-N} P(H_d)$

Handy relationships based on the Rayleigh Distribution are

$H_s = 1.596\ H$ $\quad\quad$ H = average of all waves

$H_s = 1.414$ Hrms

b. Long Term Distribution

It is assumed that significant wave heights obey long term frequency distribution.

Wave height exceedance graph is a plot of significant wave height and probability of exceedance.

The probability of H_d occurring in any single storm characterised by Hsi is

$$E_{2i} = P(H_{si})E_{1i}$$

Overall chance that Hd is exceeded at least once in single storm period is

$$E_3 = \sum_{i=1}^{N} E2i \text{ (N is number of increments of Hs)}$$

P(Hsi) is the probability of occurence of storm represented by significant wave height Hsi .

The chance that design wave height H_d is exceeded at least once during life span of structure

is $P(H_d) = 1- (1-E_3)^{M.L}$

M = number of storms per year
L = structure design life in years.

Can work through for range of values of H_d to get plot of H_d versus $P(H_d)$.

More realistic approach is to look at say maximum wave in 50 year storm as design wave.

- Acceptable level of probability of exceedance for H_d depends to some extent on the type of structure

    - Need lower value when structure can be affected by a single wave - e.g. a light tower or vertical breakwater

    - Can tolerate higher value when need numerous repetitions for major damage as with rubble mound breakwater - with mound breakwaters, wave groups are important and Hs is a useful measure.

For the particular site - determine wave climate and wave height exceedance distribution

- If possible use wave records at site (swell and wind waves) (2 years +).
- Otherwise use refracted/diffracted offshore waves (from wave atlas etc.).
- Also determine local wind waves by hindcasting calculations -graphs in Shore Protection Manual - for total fetch. Wind speed etc.
- Must bear in mind the depth limitation for wave breaking - water levels important (breaking reduces height but increases force).
- Breaking and non-breaking wave $H_b = 0.78d$

Actual design wave chosen depends on

- Expected life of structure
- Availability of construction material
- Initial and maintenance costs

ii. Form of Mound Breakwaters

Cross section

Does not need to cap structure unless have traffic or artificial units to prevent overtopping.

If overtopping prevented can reduce size of lee side armour.

iii. Theoretical Stability Consideration

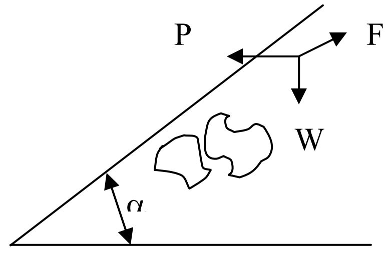
Several formulae developed to determine size of breakwater armour units.

most of form $W = \frac{c\,H^3}{f(\alpha)}$

W - armour stone weight
H - wave height
α - Slope of face
c - coefficient dependent on both mass density of units and shape

IRRIBARREN in 1930's looked at forces acting on an armour stone.

P - wave force
F - friction
W - weight



and developed expression for unit weight

$$W = \frac{N\rho_r H^3}{\Delta^3 (f \cos\alpha + \sin\alpha)^3}$$

where N = coefficient based on stone type
ρ_r = mass density kg/m3
Δ = relative density of stone in water
ρ_w = mass density of water
f = friction coefficient

f not easy to interpret -varies with shape and method placement.

Also had problems that as 450 get ridiculously high values.

HUDSON modified Irribarren to

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

W	=	mass of individual armour unit (kg)
	=	mass density of armour material (kg/m^3)
H	=	design wave height
S_r	=	specific gravity of armour unit relative to water m P r/pw
Θ	=	angle of side slope measured from horizontal in degrees
K_D	=	stability coefficient smooth rounded rock for breaking wave 1.2
		smooth rounded rock for breaking wave 2.4
P_w	=	mass density of (sea) water = 1025 kg/m^3
K_D	=	stability coefficient depending on
	-	armour shape
	-	slope
	-	method of placement
	-	breaking or non-breaking waves.
	-	number of layers
	-	position on structure (head or trunk).
	-	note that Δ = (Sr – 1)

K_D values determined by model tests on various units under various conditions and range tabulated. Depends on way test is undertaken also.

Main weakness is that the only parameter that reflects on wave loading is H.

No consideration of wave period, angle of attack, storm duration. Wave grouping etc.

Interesting to look at significance of change in parameters in Hudson's formula on the resulting armour unit mass.

- 10 per cent increase in wave height gives a 30 per cent increase in armour unit mass.
- 10 per cent increase in stone density reduces mass by 30 per cent.
- Reducing slope from 1 in 1.5 to 1 in 2 reduces I mass by 25 per cent.

Initially K_D values were determined so as the structure was theoretically undamaged if wave height H was app1ied to units of weight H.

Then va1ues of K_D were determined to a11ow for percentage of armour units dislodged i .e. certain damage percent 1evels.

Note that the actual performance of a breakwater cross-section and particularly the armour units is on1y indicated by these formula -must support findings at least with flume tests. Even tests not always reproduceable.

From the value of H for primary armour stone mass there are rules (based on test results) for determination of mass of secondary armour, underlayers, and core.

Search has been for armour units with K_D larger than that for quarry rock.

- Great diversity of shapes - many patented.

Some K_D values quoted are unrealistic.

However these units do have cost advantages where
- Local quarry rock is of poor quality
- Cannot get local supply of large stones
- K_D values required in design are greater than available from quarry rock.

-

iv. Beam Type Cross-Section

v. Optimising Design
   - Considering initial cost and maintenance cost.

Need to assess damage caused by wave conditions greater than represented by H_{so} -significant wave height causing zero damage.

a. Must have long term distribution of significant wave values.

b. Must relate variations in wave conditions (represented by Hs) to performance of structure .Hs is characteristic wave attacking the structure.

This impl1es that the known distribution of wave heights and periods are applied 1n flume tests. This enables us to determine H_{so} -max. wave height that can satisfy no damage criteria. When H_{so} exceeded -get movement of armour from positions which gives % damage.

This is shown on page (10) Hs / H_{so} vs % damage. Collapse taken as 10~ damage (H/H_{so} - 1.45).

Armour unit mass determined from test as

$$W = \frac{2.8(H_{so})^3}{50}$$

c. Must relate construction cost and H_{so}. Use simplified expression

$F1 = f(H_{so}) = 1320\ H_{so} + 8620$ [example only]

d. Must relate
- expected damage
- Offshore conditions
- H_{so}.

Get damage if $Hs>H_{so}$.
Damage expected depends on probability of $Hs > H_{so}$.

$S = \underline{100} \sum \Delta P\ \Delta W$

Where S = capitalized value of total damage
W= amount of damage associated with exceedence of H_{so} by ΔH_s which occurs with probability each year of ΔP

For life of 100 years or more.
δ = interest rate.
Note that this method is only indicative.

Important to stress that formulae give a guide only. Need to test preliminary design of breakwater in flume and also if possible in basin. Need to look not only at single large waves but wave groups or close pairs of large wave than can swamp structure and pluck out armour units.

Engineer should not leave test to laboratory technician -may never have been out of laboratory. Even testing can lead to varying results depending on laboratory technique.

e. Failure of breakwaters

Have been numerous failures of major breakwaters designed by experienced engineers.

Typical failure modes:

1. Plucking out of armour units as a result of wave inertia and drag force plus uplift forces from seepage.

2. Damage to crest as result of overtopping.

3. Excessive loading from sequence of particularly large waves.

4. Failure of secondary armour by sliding.

5. Building up of seepage uplift forces –breakwater with fine core material most susceptible.

6. Structural failure of armour units as a result of settlement or rocking under wave action. (E.g. large Dolosse units -Sines breakwater Portugal).

7. Overstressing during placement of unreinforced concrete units.

8. Abrasion to concrete units due to rocking.

9. Concrete deterioration -chemical attack in salt water - should not be a problem if care taken.

3.6 TYPICAL RUBBLE MOUND BREAKWATERS

The rubble mound breakwater (Fig 3.8) is the oldest breakwater structure, having been used for artificial harbours since Roman Times and a structure composed primarily of rocks dumped or placed upon the sea bed. An outer layer, or layers, of massive rock or precast concrete units provides an armour layer to protect the less massive rock core from wave attack.

Elements and functions of typical rubble mound breakwaters (Table 3.2)

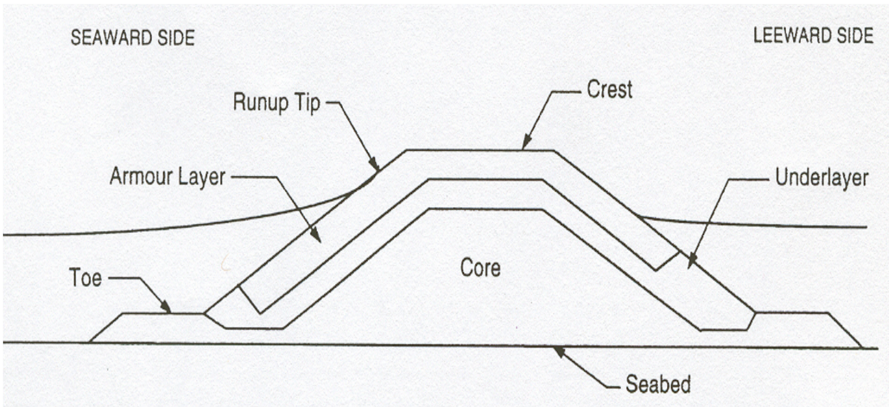


Fig 3.8: Rubble mound breakwater

Table 3.2: Elements and functions of typical rubble mound breakwaters

Element	**Function**
Seabed	Provides embankment stability
Core	Reduce wave transmission
Toe	support the main armour
Under layer	Contains the core and provides foundation for armour
Armour layer	Provides wave protection
Crest	Provide access and reduces overtopping

3.7 CONSTRUCTION

The design of breakwater must be kept simple because much of the breakwater will be constructed underwater

Construction must take into account

- local conditions
- availability of materials
- availability of construction plants

Overseas	-	heavy floating plants available including jack up barges, bottom dump and side dump barges, etc
Australia	-	generally use land based plant unless breakwater is a major structure

- REMEMBER Sea is very powerful.
  Construction method should take this into account.

Designer must have a specific method in mind to ensure that construction is practicable. Specifications and drawings can be developed around specific method but allow alternatives.

Methods of construction:

i. Nice to build in dry but seldom practical to cofferdam area and pump out.

ii. Common method in Australia is to end dump core and place armour with crane or large hydraulic excavator.

   - problems with congestion of plants on long breakwater
   - consider construction of second connection

iii. Stability during construction may dictate that cannot build up layer by layer – problem with scour of finer material by waves and currents

   Also - do not core advance too far ahead of armour

   - design secondary armour to offer reasonable protection

3.8 REFERENCES

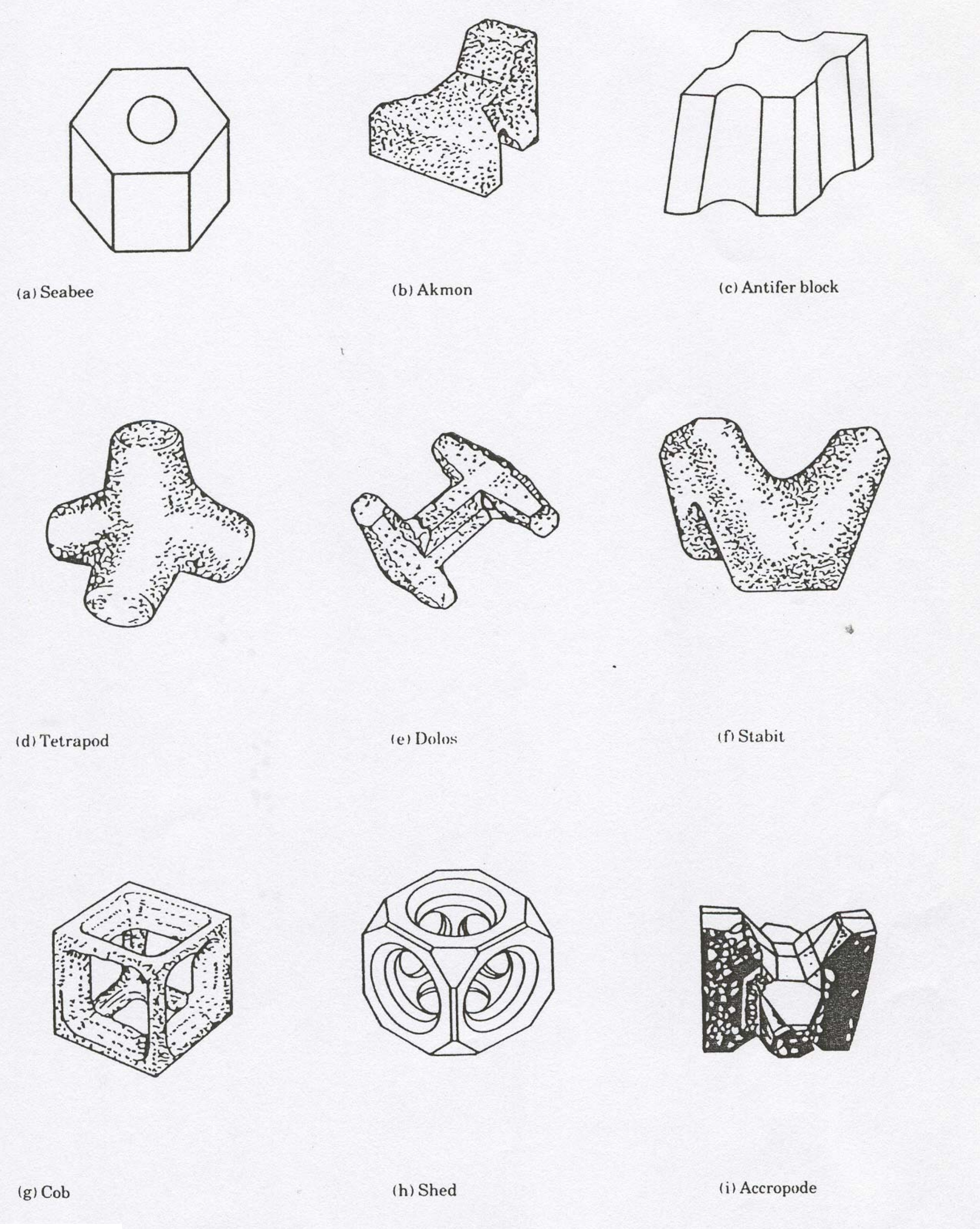
1. BS 6349 PT 7 – Design And Construction Of Breakwater

2. Technical Standards For Port and Harbour Facilities In Japan (OCDI)

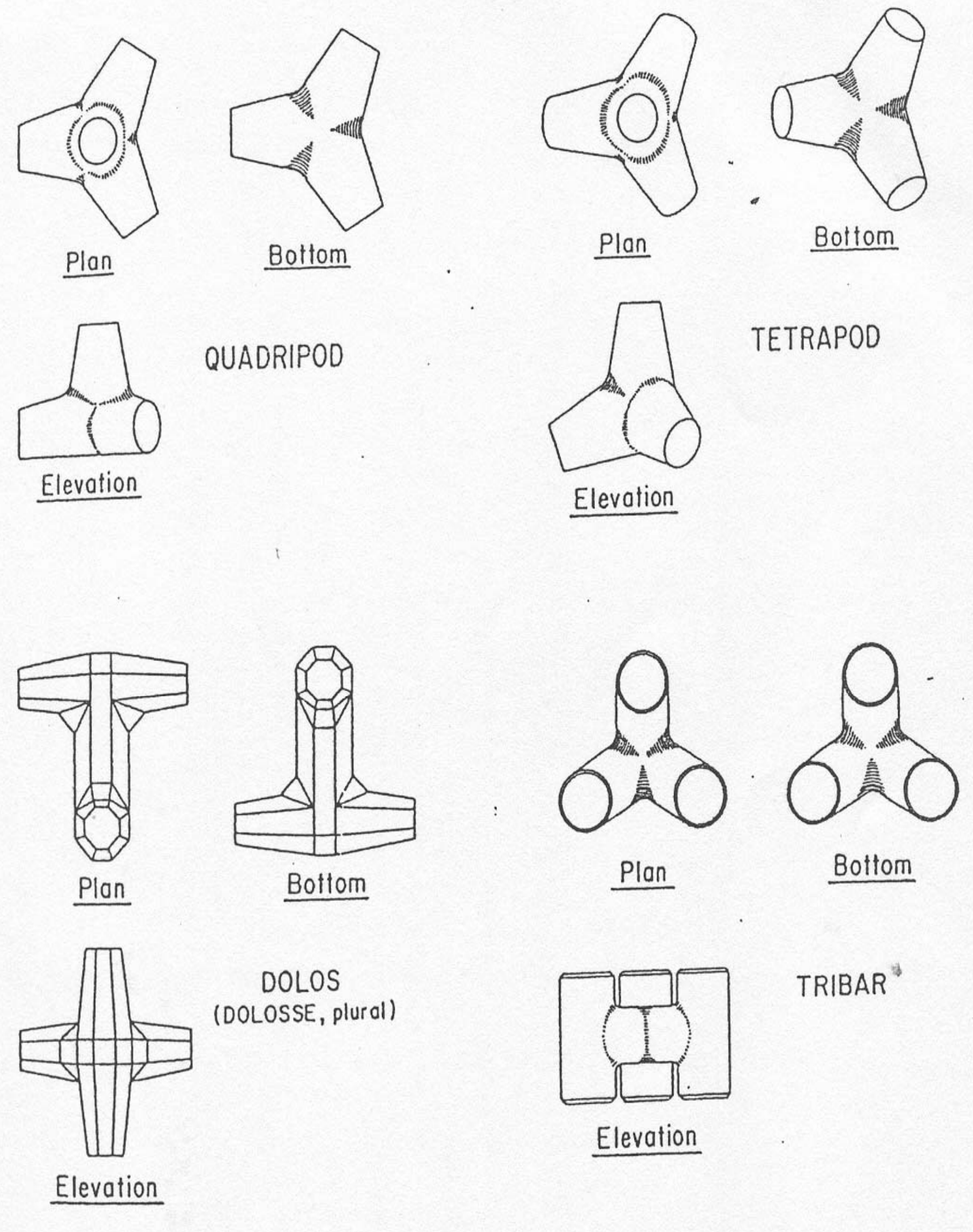
3. Shore Protection Manual (US Army Corps Of Engineers)

APPENDIX

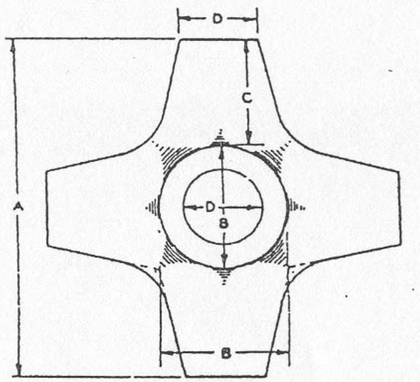
APPENDIX A: TYPE OF ARMOUR



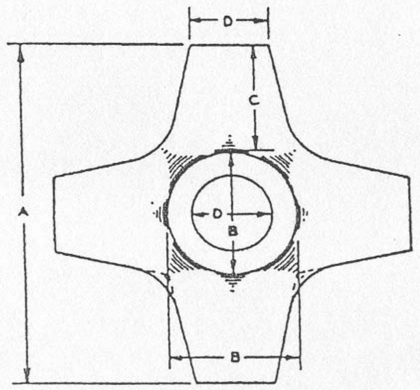
. Examples of concrete armour units



Views of the tetrapod, quadripod, tribar, and dolos armor units.



PLAN



ELEVATION

NOTE: DATA BASED ON HEXAPODS USED IN MODEL TESTS CONDUCTED AT THE WATERWAYS EXPERIMENT STATION.

	VOLUME OF INDIVIDUAL ARMOR UNITS (CU FT)										
	7.14	14.29	28.57	71.43	142.86	214.29	285.71	357.14	428.57	500.00	571.43
UNIT WEIGHT LB/CU FT	WEIGHT OF INDIVIDUAL ARMOR UNITS (TONS)										
140.0	0.50	1.00	2.00	5.00	10.00	15.00	20.00	25.00	30.00	35.00	40.00
149.5	0.53	1.07	2.14	5.34	10.68	16.02	21.36	26.70	32.04	37.38	42.71
156.0	0.56	1.11	2.23	5.57	11.14	16.71	22.29	27.86	33.43	38.00	44.57
162.0	0.58	1.16	2.31	5.79	11.57	17.36	23.14	28.93	34.71	40.50	46.29
	AVERAGE MEASURED THICKNESS OF ONE LAYER PLACED UNIFORMLY (FT)										
	2.48	3.13	3.94	5.35	6.74	7.72	8.50	9.15	9.73	10.24	10.70
	AVERAGE MEASURED THICKNESS OF TWO LAYERS RANDOM PLACED (FT)										
	4.43	5.58	7.03	9.54	12.02	13.76	15.15	16.32	17.34	18.26	19.09
	NUMBER OF ARMOR UNITS PER 1000 SQ FT (ONE LAYER PLACED UNIFORMLY)										
	198.09	124.78	78.60	42.72	26.84	20.51	16.99	14.63	13.01	11.69	10.74
	NUMBER OF ARMOR UNITS PER 1000 SQ FT (TWO LAYERS RANDOM PLACED)										
	328.40	206.86	130.31	70.82	44.49	34.01	28.16	24.26	21.58	19.38	17.80
SYMBOL	DIMENSIONS OF ARMOR UNITS (FT)										
A	3.44	4.33	5.46	7.41	9.34	10.69	11.76	12.67	13.47	14.18	14.82
B	1.23	1.55	1.95	2.65	3.34	3.82	4.21	4.53	4.82	5.07	5.30
C	1.11	1.40	1.76	2.39	3.01	3.45	3.80	4.09	4.35	4.57	4.78
D	0.74	0.93	1.17	1.59	2.01	2.30	2.53	2.73	2.90	3.05	3.19

VOLUME OF INDIVIDUAL ARMOR UNIT $(V) = 0.176A^3$

where

A = overall dimension of unit $\quad C = 0.322A$

$B = 0.357A \quad D = 0.215A$

ARMOR LAYER THICKNESS (2 UNITS RANDOM) $= 1.29A$

NUMBER OF HEXAPODS (TWO LAYERS UNIFORMLY PLACED) PER UNIT AREA $(N_r) = 0.74V^{2/3}$ (see eq. (7-122))

where

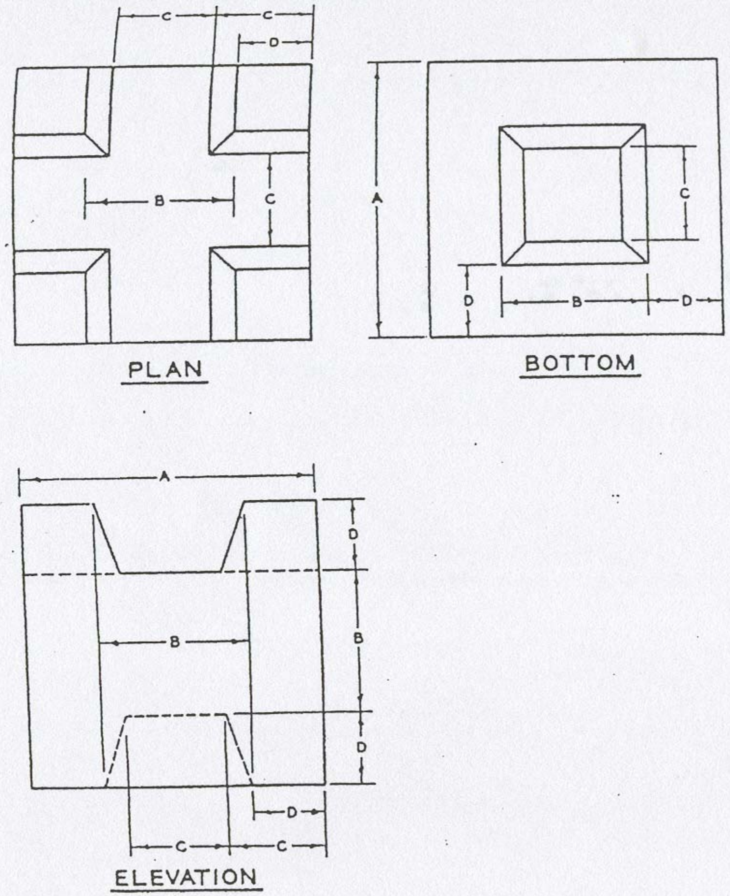
$k_\Delta = 1.29 \quad P = 43$

NUMBER OF HEXAPODS (TWO LAYERS, RANDOM PLACED) PER UNIT AREA $(N_r) = 1.22V^{2/3}$ (see eq. (7-122))

where

$k_\Delta = 1.15 \quad P = 47$

Hexapod specifications.



NOTE: DATA BASED ON MODIFIED CUBES USED IN MODEL TESTS CONDUCTED AT THE WATERWAYS EXPERIMENT STATION.

	VOLUME OF INDIVIDUAL ARMOR UNITS (CU FT)										
	7.14	14.29	28.57	71.43	142.86	214.29	285.71	357.14	428.57	500.00	571.43
UNIT WEIGHT LB/CU FT	WEIGHT OF INDIVIDUAL ARMOR UNITS (TONS)										
140.0	0.50	1.00	2.00	5.00	10.00	15.00	20.00	25.00	30.00	35.00	40.00
149.5	0.53	1.07	2.14	5.34	10.68	16.02	21.36	26.70	32.04	37.38	42.71
156.0	0.56	1.11	2.23	5.57	11.14	16.71	22.29	27.86	33.43	39.00	44.57
162.0	0.58	1.16	2.31	5.79	11.57	17.36	23.14	28.93	34.71	40.50	46.29
	AVERAGE MEASURED THICKNESS OF ONE LAYER PLACED UNIFORMLY (FT)										
	2.16	2.72	3.42	4.65	5.86	6.70	7.38	7.95	8.44	8.89	9.29
	AVERAGE MEASURED THICKNESS OF TWO LAYERS RANDOM PLACED (FT)										
	4.24	5.34	6.73	9.13	11.50	13.16	14.49	15.61	16.59	17.46	18.26
	NUMBER OF ARMOR UNITS PER 1000 SQ FT (ONE LAYER PLACED UNIFORMLY)										
	220.26	138.75	87.40	47.50	29.84	22.81	18.89	16.27	14.47	13.00	11.94
	NUMBER OF ARMOR UNITS PER 1000 SQ FT (TWO LAYERS RANDOM PLACED)										
	314.12	197.87	124.65	67.74	42.56	32.53	26.93	23.20	20.64	18.54	17.02
SYMBOL	DIMENSIONS OF ARMOR UNITS (FT)										
A	2.09	2.63	3.32	4.50	5.67	6.49	7.15	7.70	8.18	8.61	9.01
B	1.05	1.32	1.67	2.26	2.85	3.26	3.59	3.87	4.11	4.33	4.52
C	0.70	0.88	1.11	1.51	1.90	2.18	2.39	2.58	2.74	2.88	3.02
D	0.52	0.66	0.83	1.12	1.41	1.62	1.78	1.92	2.04	2.14	2.24

VOLUME OF INDIVIDUAL ARMOR UNIT (V) = $0.781A^3$

where:

A = width of cube   C = 0.335A

B = 0.502A   D = 0.249A

ARMOR LAYER THICKNESS (2 UNITS, RANDOM) = 1.03A

NUMBER OF MODIFIED CUBES (TWO LAYERS UNIFORMLY PLACED) PER UNIT AREA (N_r) = $0.85V^{-2/3}$ (see eq. (7-122))

where

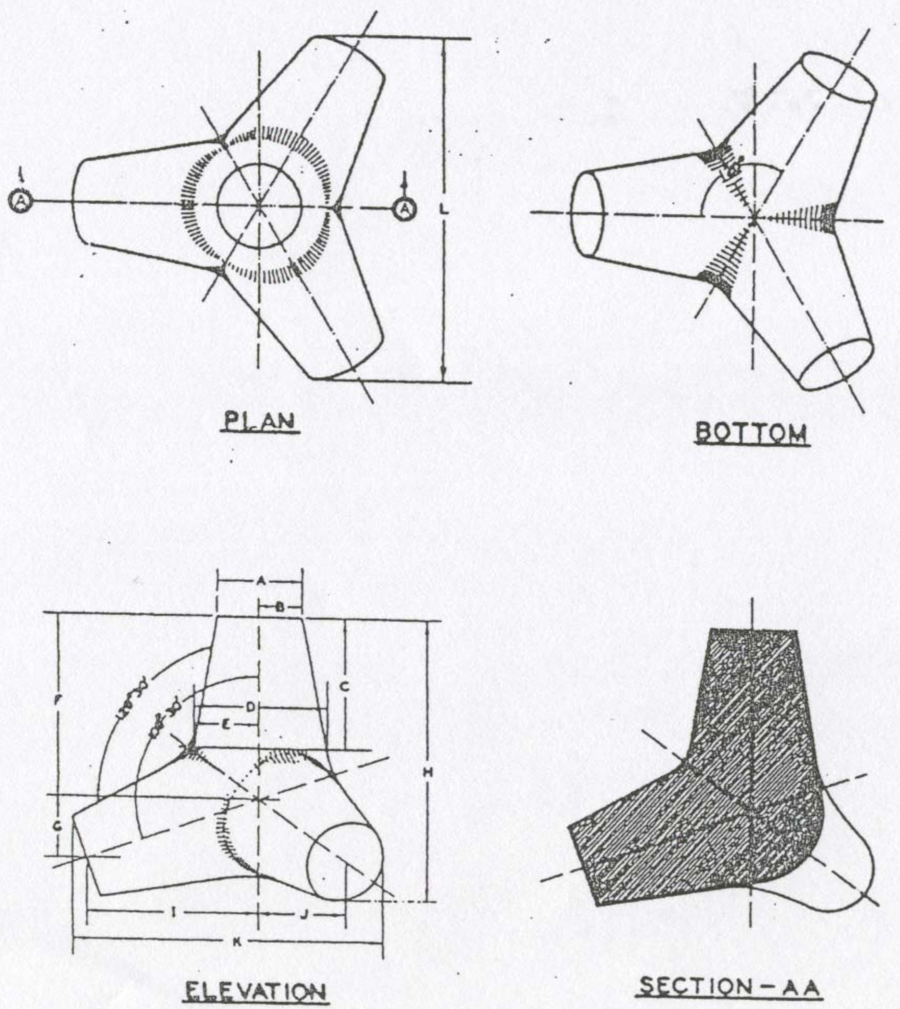
k_Δ = 1.12   P = 25

NUMBER OF MODIFIED CUBES (TWO LAYERS, RANDOM PLACED) PER UNIT AREA (N_r) = $1.17V^{-2/3}$ (see eq. (7-122))

where.

k_Δ = 1.10   P = 47

Modified cube specifications.



NOTE: DATA BASED ON TETRAPODS USED IN MODEL TESTS CONDUCTED AT THE WATERWAYS EXPERIMENT STATION

VOLUME OF INDIVIDUAL ARMOR UNITS (CU FT)

UNIT WEIGHT LB/CU FT	7.14	14.29	28.57	71.43	142.86	214.29	285.71	357.14	428.57	500.00	571.43
	WEIGHT OF INDIVIDUAL ARMOR UNITS (TONS)										
140.0	0.50	1.00	2.00	5.00	10.00	15.00	20.00	25.00	30.00	35.00	40.00
149.5	0.53	1.07	2.14	5.34	10.68	16.02	21.36	26.70	32.04	37.38	42.71
156.0	0.56	1.11	2.23	5.57	11.14	16.71	22.29	27.86	33.43	39.00	44.57
162.0	0.58	1.16	2.31	5.79	11.57	17.36	23.14	28.93	34.71	40.50	46.29
	AVERAGE MEASURED THICKNESS OF TWO LAYERS RANDOM PLACED (FT)										
	4.01	5.05	6.36	8.63	10.87	12.45	13.70	14.76	15.68	16.51	17.26
	NUMBER OF ARMOR UNITS PER 1000 SQ FT (TWO LAYERS RANDOM PLACED)										
	280.18	176.40	111.18	60.42	37.96	29.02	24.02	20.70	18.41	16.64	15.18
SYMBOL	**DIMENSIONS OF ARMOR UNITS (FT)**										
A	0.89	1.12	1.41	1.91	2.41	2.76	3.04	3.27	3.48	3.66	3.83
B	0.44	0.56	0.70	0.95	1.20	1.38	1.52	1.63	1.74	1.83	1.91
C	1.40	1.77	2.23	3.02	3.81	4.36	4.80	5.17	5.50	5.79	6.05
D	1.38	1.74	2.20	2.98	3.76	4.30	4.73	5.10	5.42	5.70	5.96
E	0.69	0.87	1.10	1.49	1.88	2.15	2.37	2.55	2.71	2.85	2.98
F	1.89	2.38	3.00	4.08	5.14	5.88	6.47	6.97	7.41	7.80	8.16
G	0.63	0.79	1.00	1.36	1.71	1.96	2.16	2.32	2.47	2.60	2.72
H	2.94	3.71	4.67	6.34	7.99	9.14	10.07	10.84	11.52	12.13	12.68
I	1.78	2.26	2.83	3.84	4.84	5.54	6.10	6.57	6.98	7.35	7.68
J	0.89	1.12	1.41	1.92	2.42	2.77	3.05	3.28	3.48	3.67	3.84
K	3.21	4.04	5.09	6.91	8.71	9.97	10.97	11.82	12.56	13.23	13.83
L	3.54	4.45	5.61	7.62	9.60	10.98	12.09	13.02	13.84	14.57	15.23

VOLUME OF INDIVIDUAL ARMOR UNIT (V) = $0.280H^3$

where

A = 0.302H	G = 0.215H
B = 0.151H	H = overall dimension of unit
C = 0.477H	I = 0.606H
D = 0.470H	J = 0.303H
E = 0.235H	K = 1.091H
F = 0.644H	L = 1.201H

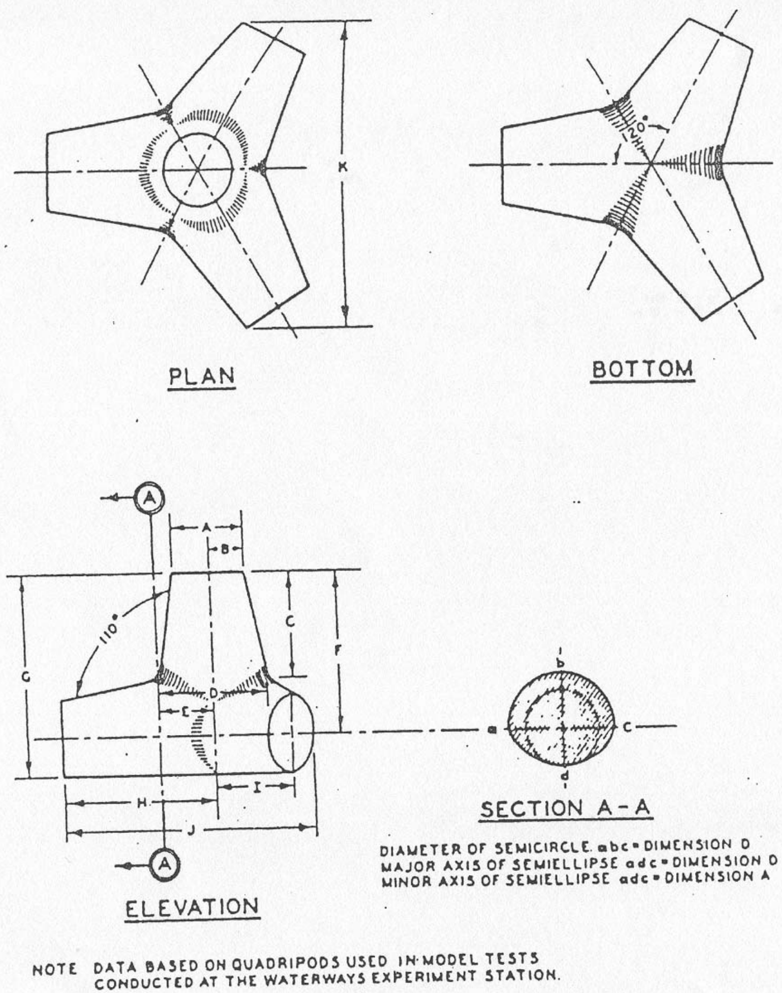
ARMOR LAYER THICKNESS (2 UNITS) = 1.361H

NUMBER OF TETRAPODS (TWO LAYERS, RANDOM PLACED) PER UNIT AREA (N_r) = $1.02^{-2/3}$ (see eq. (7-122))

where

$k_\Delta = 1.02$ $P = 50$

Tetrapod specifications.



	VOLUME OF INDIVIDUAL ARMOR UNITS (CU FT)										
	7.14	14.29	28.57	71.43	142.86	214.29	285.71	357.14	428.57	500.00	571.43
UNIT WEIGHT LB/CU FT	**WEIGHT OF INDIVIDUAL ARMOR UNITS (TONS)**										
140.0	0.50	1.00	2.00	5.00	10.00	15.00	20.00	25.00	30.00	35.00	40.00
149.5	0.53	1.07	2.14	5.34	10.68	16.02	21.36	26.70	32.04	37.38	42.71
156.0	0.56	1.11	2.23	5.57	11.14	16.71	22.29	27.86	33.43	39.00	44.57
162.0	0.58	1.16	2.31	5.79	11.57	17.36	23.14	28.93	34.71	40.50	46.29
	THICKNESS OF TWO LAYERS PLACED PELL-MELL (FT)										
	3.66	4.61	5.81	7.88	9.93	11.37	12.51	13.48	14.33	15.06	15.77
	NUMBER OF ARMOR UNITS PER 1000 SQ FT (TWO LAYERS PLACED PELL-MELL)										
	261.05	164.44	103.59	56.30	35.37	27.04	22.38	19.28	17.15	15.41	14.15
SYMBOL	**DIMENSIONS OF ARMOR UNITS (FT)**										
A	0.93	1.17	1.48	2.00	2.52	2.88	3.17	3.42	3.63	3.82	4.01
B	0.46	0.58	0.74	1.00	1.26	1.44	1.59	1.71	1.82	1.92	2.00
C	1.28	1.61	2.03	2.76	3.48	3.98	4.38	4.72	5.01	5.28	5.52
D	1.38	1.74	2.19	2.97	3.74	4.28	4.71	5.08	5.40	5.68	5.94
E	0.69	0.87	1.09	1.49	1.87	2.14	2.36	2.54	2.70	2.84	2.97
F	1.97	2.48	3.13	4.25	5.35	6.12	6.74	7.26	7.72	8.12	8.49
G	2.43	3.07	3.86	5.24	6.61	7.57	8.33	8.97	9.53	10.03	10.49
H	1.97	2.48	3.13	4.25	5.35	6.12	6.74	7.26	7.72	8.12	8.49
I	0.99	1.24	1.56	2.12	2.67	3.06	3.37	3.63	3.86	4.06	4.25
J	3.36	4.23	5.33	7.23	9.12	10.43	11.48	12.37	13.15	13.83	14.47
K	3.88	4.88	6.15	8.35	10.52	12.05	13.26	14.28	15.18	15.98	16.70

VOLUME OF INDIVIDUAL ARMOR UNIT (V) = $0.495G^3$

where:

$A = 0.382G$	G = overall height of unit
$B = 0.191G$	$H = 0.809G$
$C = 0.526G$	$I = 0.405G$
$D = 0.566G$	$J = 1.379G$
$E = 0.283G$	$K = 1.592G$
$F = 0.809G$	

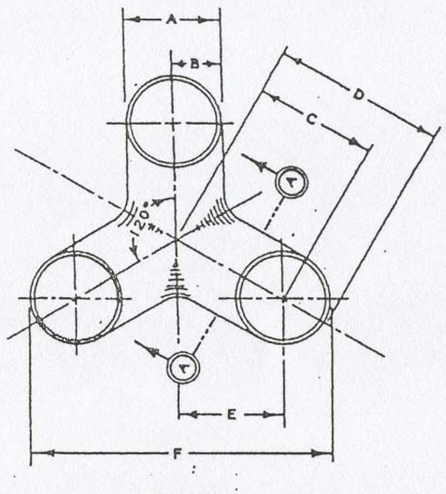
ARMOR LAYER THICKNESS (2 UNITS) = $1.602G$

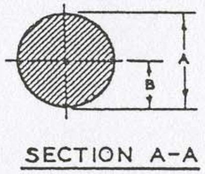
NUMBER OF QUADRIPODS (TWO LAYERS, RANDOM PLACED) PER UNIT AREA $(N_r) = 0.97V^{-2/3}$ (see eq. (7-122))

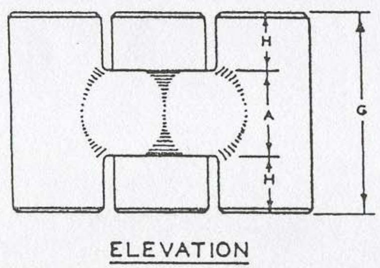
where:

$K_\Delta = 0.95$ $\qquad p = 49$

Quadripod specifications.







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

UNIT WEIGHT LB/CU FT	VOLUME OF INDIVIDUAL ARMOR UNITS (CU FT)										
	7.14	14.29	28.57	71.43	142.86	214.29	285.71	357.14	428.57	500.00	571.43
	WEIGHT OF INDIVIDUAL ARMOR UNITS (TONS)										
140.0	0.50	1.00	2.00	5.00	10.00	15.00	20.00	25.00	30.00	35.00	40.00
149.5	0.53	1.07	2.14	5.34	10.68	16.02	21.36	26.70	32.04	37.38	42.71
156.0	0.56	1.11	2.23	5.57	11.14	16.71	22.29	27.86	33.43	39.00	44.57
162.0	0.58	1.16	2.31	5.79	11.57	17.36	23.14	28.93	34.71	40.50	46.29
	AVERAGE MEASURED THICKNESS OF ONE LAYER PLACED UNIFORMLY (FT)										
	2.18	2.74	3.45	4.69	5.91	6.76	7.44	8.02	8.52	8.97	9.38
	AVERAGE MEASURED THICKNESS OF TWO LAYERS PLACED PELL-MELL (FT)										
	3.85	4.85	6.11	8.30	10.46	11.97	13.17	14.19	15.08	15.87	16.60
	NUMBER OF ARMOR UNITS PER 1000 SQ FT (ONE LAYER PLACED UNIFORMLY)										
	161.34	101.63	64.02	34.80	21.86	16.71	13.83	11.92	10.80	9.52	8.74
	NUMBER OF ARMOR UNITS PER 1000 SQ FT (TWO LAYERS PLACED PELL-MELL)										
	247.88	156.12	98.35	53.45	33.58	25.87	21.25	18.31	16.28	14.63	13.43
SYMBOL	DIMENSIONS OF ARMOR UNITS (FT)										
A	1.05	1.32	1.66	2.25	2.84	3.25	3.58	3.85	4.09	4.31	4.51
B	0.52	0.66	0.83	1.13	1.42	1.62	1.79	1.93	2.05	2.15	2.25
C	1.25	1.58	1.99	2.70	3.41	3.90	4.29	4.62	4.91	5.17	5.41
D	1.78	2.24	2.82	3.83	4.82	5.52	6.08	6.55	6.96	7.33	7.66
E	1.09	1.37	1.72	2.34	2.95	3.38	3.72	4.00	4.25	4.48	4.68
F	3.22	4.06	5.11	6.93	8.74	10.00	11.01	11.86	12.60	13.26	13.87
G	2.09	2.63	3.32	4.51	5.68	6.50	7.15	7.70	8.19	8.62	9.01
H	0.52	0.66	0.83	1.13	1.42	1.62	1.79	1.93	2.05	2.15	2.25

VOLUME OF INDIVIDUAL ARMOR UNIT $(V) = 6.48A^3$

where:

A = diameter of legs $E = 1.08A$

$B = 0.5A$ $F = 3.18A$

$C = 1.25A$ $G = 2A$

$D = 1.75A$ $H = B = 0.5A$

ARMOR LAYER THICKNESS (2 UNITS, RANDOM) = $3.68A$

NUMBER OF TRIBARS (TWO LAYERS, UNIFORMLY PLACED) PER UNIT AREA $(N_r) = 0.80V^{-2/3}$ (see eq. (7-172))

where:

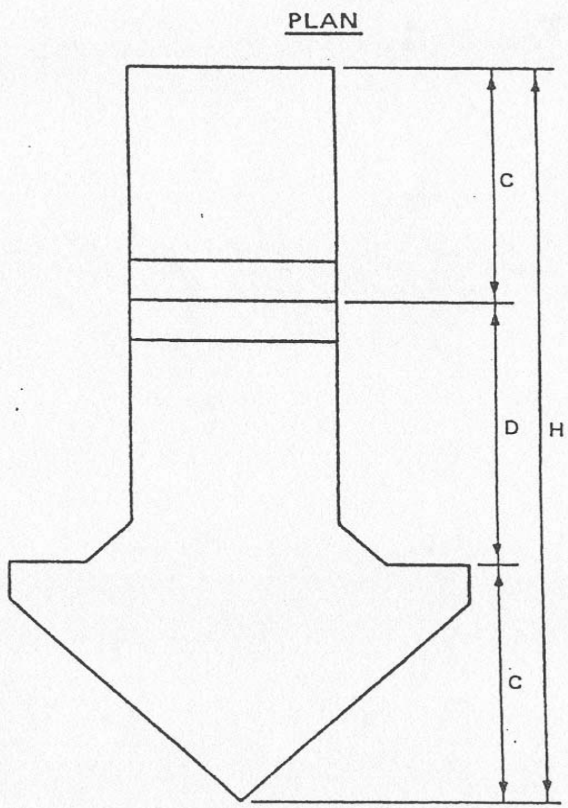
$k_\Delta = 1.02$ $P = 58$

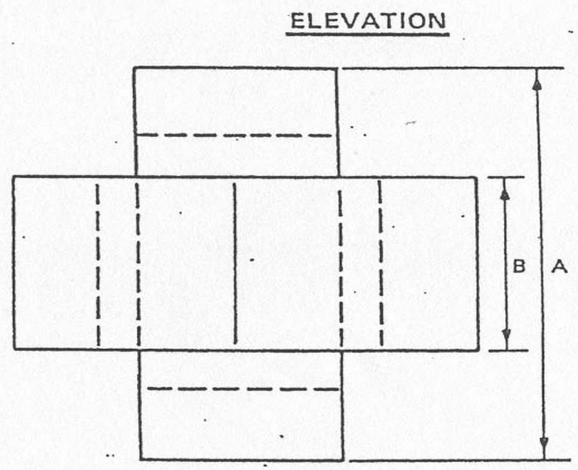
NUMBER OF TRIBARS (TWO LAYERS, RANDOM PLACED) PER UNIT AREA $(N_r) = 0.90V^{-2/3}$ (see eq. (7-172))

where:

$k_\Delta = 1.13$ $P = 47$

Tribar specifications.





VOLUME OF INDIVIDUAL ARMOR UNIT (V) = $0.083H^3$

where:

A = 0.575H   C = 0.345H

B = 0.260H   D = 0.310H

H = overall height of unit

ARMOR LAYER THICKNESS (2 UNITS) = 0.899H

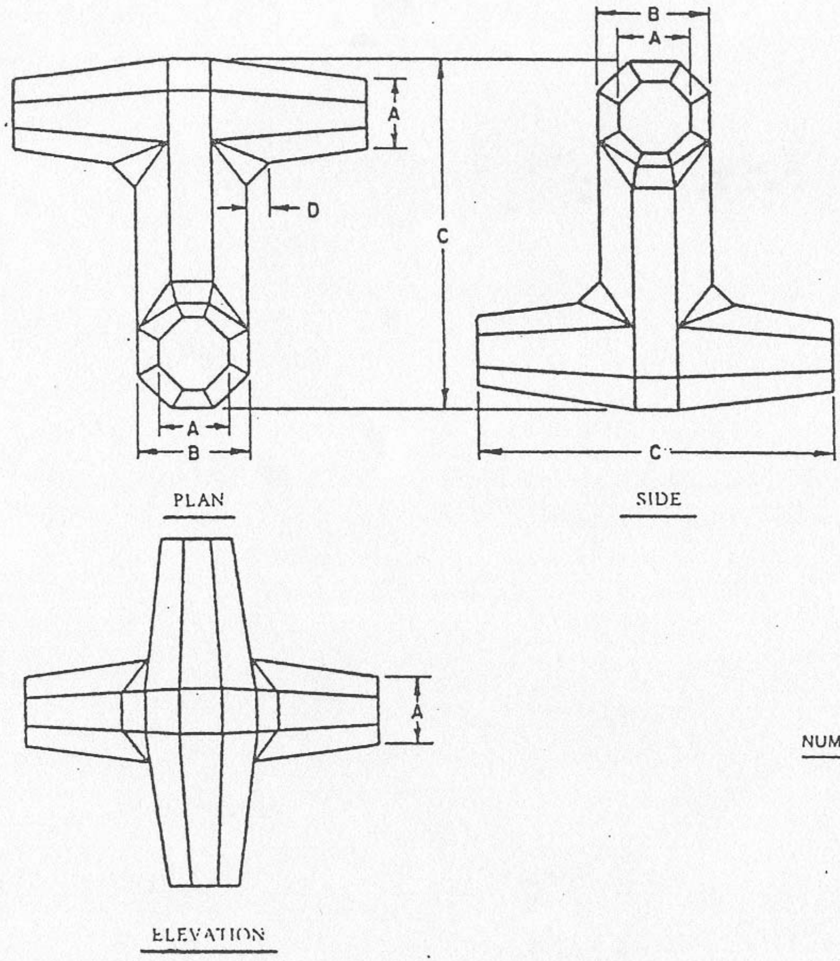
NUMBER OF TOSKANES (TWO LAYERS, RANDOM PLACED)

PER UNIT AREA (N_r) = $0.99V^{-2/3}$ (see eq. (7-122))

where:

k_Δ = 1.03   P = 52

Toskane specifications.



UNIT WEIGHT LB/CU FT	VOLUME OF INDIVIDUAL UNITS (CU FT)										
	7.14	14.29	28.57	71.43	142.86	214.29	285.71	357.14	428.57	500.00	571.43
	WEIGHT OF INDIVIDUAL ARMOR UNITS (TONS)										
140.0	0.50	1.00	2.00	5.00	10.00	15.00	20.00	25.00	30.00	35.00	40.00
149.5	0.53	1.07	2.14	5.34	10.68	16.02	21.36	26.70	32.04	37.38	42.71
156.0	0.56	1.11	2.23	5.57	11.14	16.71	22.29	27.86	32.43	39.00	44.57
162.0	0.58	1.16	2.31	5.79	11.57	17.36	23.14	28.93	34.71	40.50	46.29
	NUMBER OF ARMOR UNITS PER 1,000 SQ FT (TWO LAYERS RANDOM PLACED)										
	223.0	104.5	88.5	48.1	30.3	23.1	19.1	16.4	14.6	13.1	12.0
	AVERAGE MEASURED THICKNESS OF TWO LAYERS RANDOM PLACED (FT)										
	3.6	4.6	5.7	7.8	9.8	11.2	12.4	13.3	14.2	14.9	15.6
SYMBOL	DIMENSIONS OF ARMOR UNITS (FT)										
A	0.71	0.89	1.13	1.53	1.93	2.20	2.43	2.61	2.78	2.92	3.06
B	1.13	1.43	1.80	2.44	3.08	3.52	3.88	4.18	4.44	4.68	4.89
C	3.55	4.47	5.63	7.64	9.63	11.02	12.13	13.08	13.89	14.62	15.29
D	0.20	0.25	0.32	0.43	0.55	0.62	0.69	0.74	0.79	0.83	0.87

NOTE: SHAPE AND DIMENSIONS OF UNIT WERE BASED ON THOSE USED IN MODEL TESTS. AT PRESENT TIME INVENTOR RECOMMENDS B = 0.32 C TO 0.36 C DEPENDING ON THE WEIGHT OF THE INDIVIDUAL UNIT.

VOLUME OF INDIVIDUAL ARMOR UNIT (V) = $0.16C^3$

where:

A = 0.20C   C = overall dimension

B = 0.32C   D = 0.057C

E = 0.364C

ARMOR LAYER THICKNESS (2 UNITS) = 1.020C

NUMBER OF DOLOSSE (TWO LAYERS, RANDOM PLACED) PER UNIT AREA (N_r) = $0.83V^{-2/3}$ (see eq. (7-122))

where:

k_Δ = 0.94   P = 56

Dolos specifications.

APPENDIX B: PHOTOGRAPH

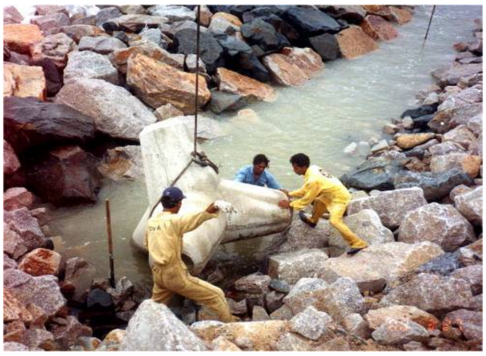
PROJECT : BREAKWATER IN MARANG, TERENGGANU (3.8 T TETRAPODS)

<u>DURING CONSTRUCTION</u>

Tetrapod steel mould

Transporting of breakwater

Toe Berm & Filter layer nearing completion



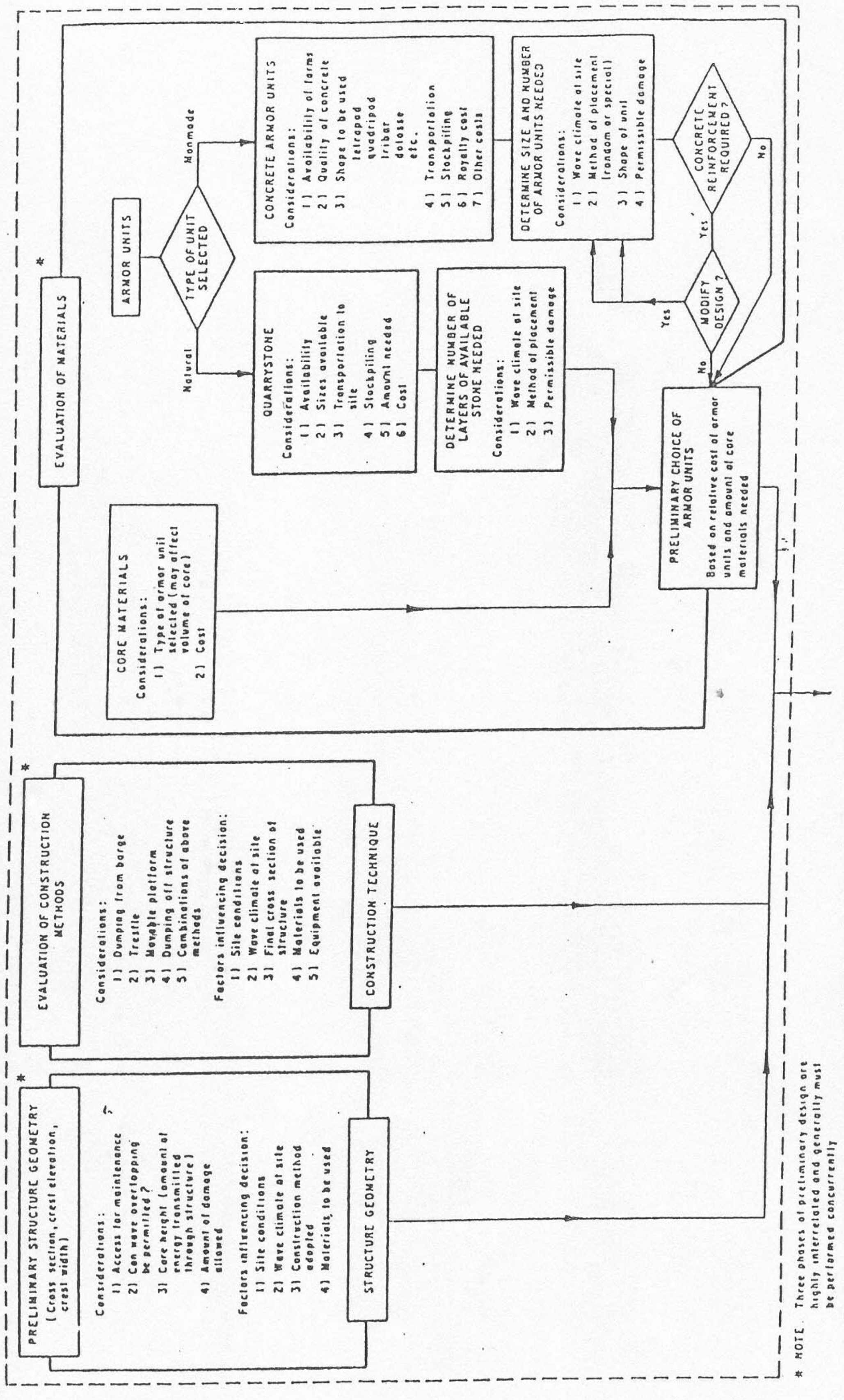
Placing of 3.8 t tetrapod

Placing of first layer of tetrapod

Placing of second layer of 3.8t tetrapod

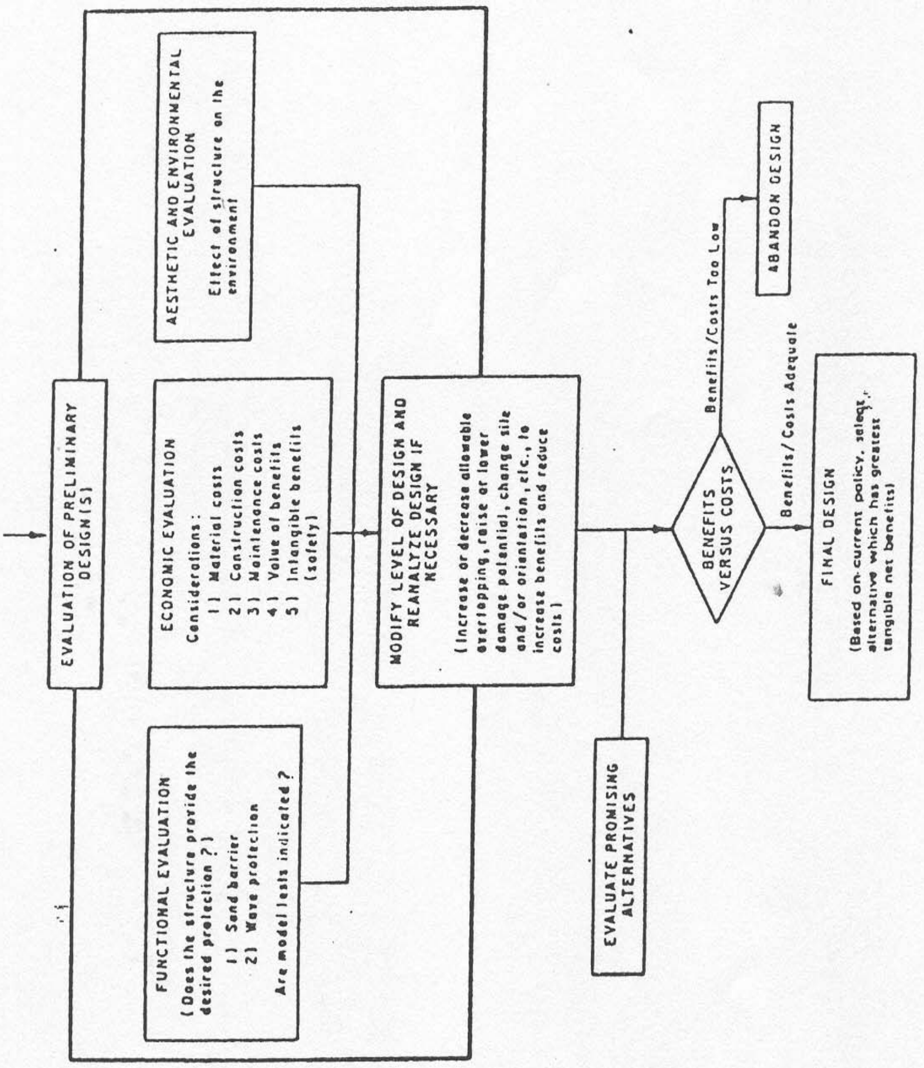
COMPLETION OF 3.8 T TETRAPOD IN MARANG, TERENGGANU

APPENDIX C: SUPPLEMENTARY ATTACHMENTS

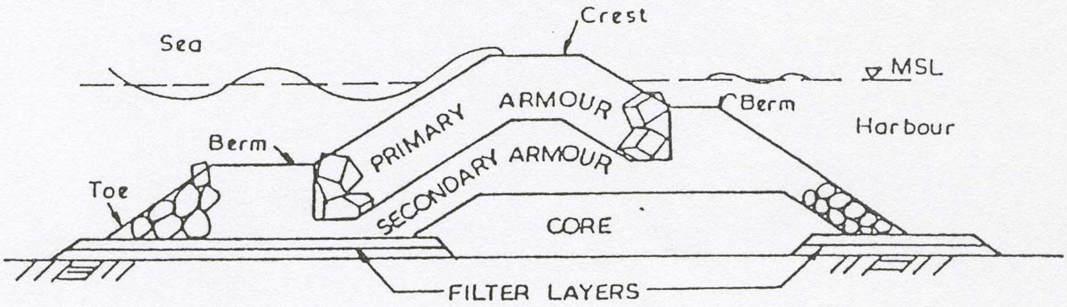


Logic diagram for preliminary design of rubble structure.

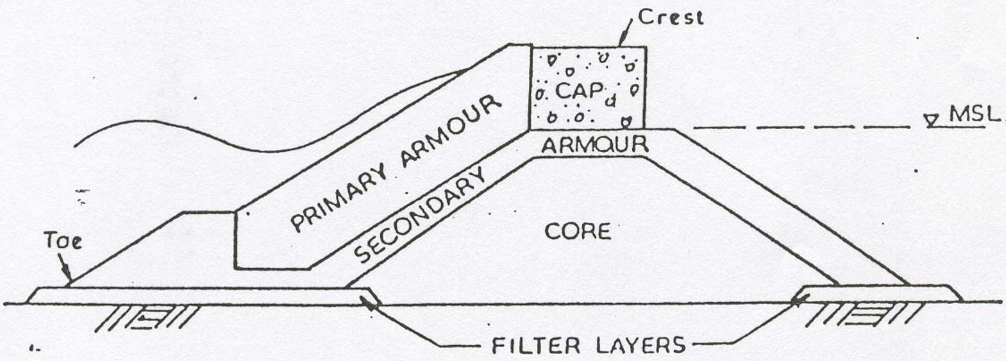
FIGURE 7-118



Logic diagram for evaluation of preliminary design.



TYPICAL SECTION OF AN OVERTOPPED BREAKWATER.



TYPICAL SECTION OF A NEW OVERTOPPED BREAKWATER.

Suggested K_D Values for use in determining armor unit weight[1].

No-Damage Criteria and Minor Overtopping							
			Structure Trunk		Structure Head		
Armor Units	n[3]	Placement	K_D[2]		K_D		Slope
			Breaking Wave	Nonbreaking Wave	Breaking Wave	Nonbreaking Wave	Cot θ
Quarrystone							
Smooth rounded	2	Random	*1.2*	2.4	*1.1*	1.9	1.5 to 3.0 [5]
Smooth rounded	>3	Random	*1.6*	*3.2*	*1.4*	*2.3*	[5]
Rough angular	1	Random [4]	[4]	*2.9*	[4]	*2.3*	
Rough angular	2	Random	2.0	4.0	*1.9*	3.2	1.5
					1.6	2.8	2.0
					1.3	2.3	3.0
Rough angular	>3	Random	*2.2*	*4.5*	*2.1*	*4.2*	[5]
Rough angular	2	Special [6]	5.8	7.0	*5.3*	6.4	[5]
Parallelepiped[7]	2	Special [1]	7.0 – 20.0	*8.5 – 24.0*	—	—	
Tetrapod and Quadripod	2	Random	7.0	8.0	*5.0*	6.0	1.5
					4.5	5.5	2.0
					3.5	4.0	3.0
Tribar	2	Random	*9.0*	10.0	*8.3*	9.0	1.5
					7.8	8.5	2.0
					6.0	6.5	3.0
Dolos	2	Random	15.8[8]	31.8[8]	*8.0*	*16.0*	2.0[9]
					7.0	*14.0*	3.0
Modified cube	2	Random	*6.5*	7.5	—	*5.0*	5
Hexapod	2	Random	*8.0*	9.5	*5.0*	*7.0*	5
Toskane	2	Random	*11.0*	22.0	—	—	5
Tribar	1	Uniform	12.0	15.0	*7.5*	*9.5*	5
Quarrystone (K_{RR}) Graded angular	–	Random	2.2	2.5	—	—	

1 CAUTION: Those K_D values shown in *italics* are unsupported by test results and are only provided for preliminary design purposes.

2 Applicable to slopes ranging from 1 on 1.5 to 1 on 5.

3 n is the number of units comprising the thickness of the armor layer.

4 The use of single layer of quarrystone armor units is not recommended for structures subject to breaking waves, and only under special conditions for structures subject to nonbreaking waves. When it is used, the stone should be carefully placed.

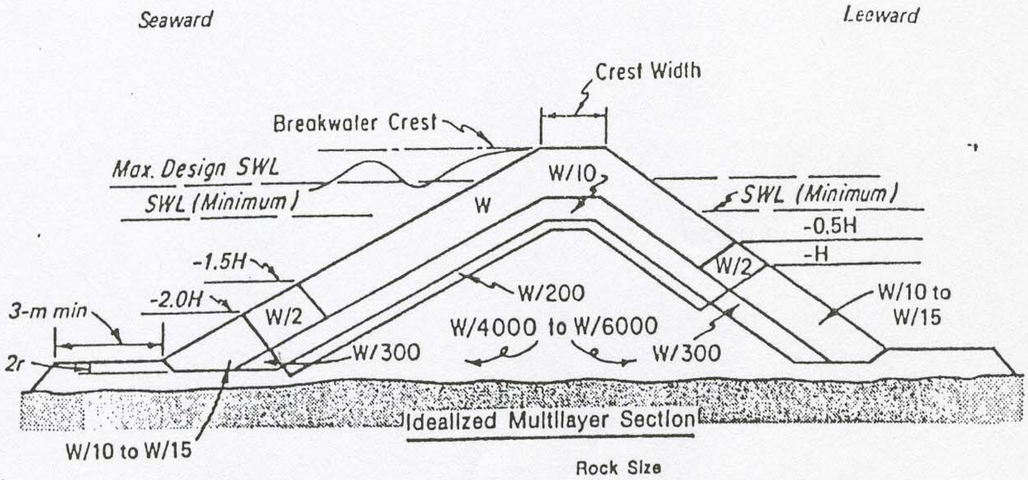
5 Until more information is available on the variation of K_D value with slope, the use of K_D should be limited to slopes ranging from 1 on 1.5 to 1 on 3. Some armor units tested on a structure head indicate a K_D-slope dependence.

6 Special placement with long axis of stone placed perpendicular to structure face.

7 Parallelepiped-shaped stone: long slab-like stone with the long dimension about 3 times the shortest dimension (Markle and Davidson, 1979).

8 Refers to no-damage criteria (<5 percent displacement, rocking, etc.); if no rocking (<2 percent) is desired, reduce K_D 50 percent (Zwamborn and Van Niekerk, 1982).

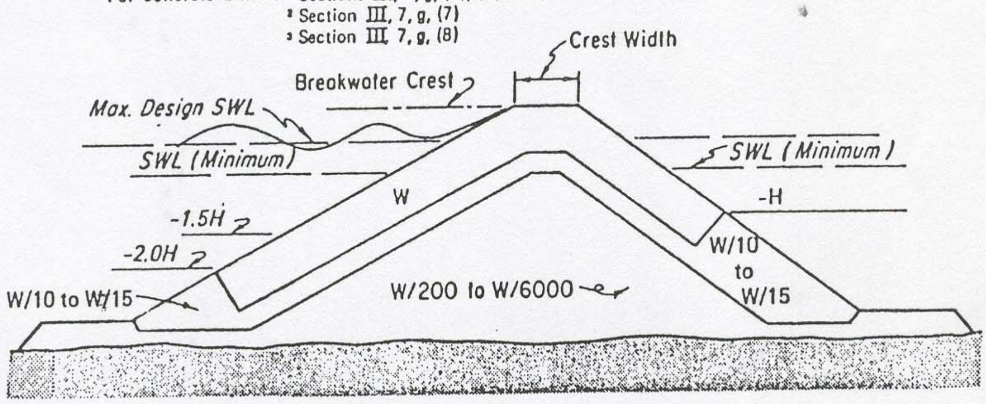
9 Stability of dolosse on slopes steeper than 1 on 2 should be substantiated by site-specific model tests.



Rock Size	Layer	Rock Size Gradation (%)
W	Primary Cover Layer[1]	125 to 75
W/2 and W/15	Secondary Cover Layer[2]	125 to 75
W/10 and W/300	First Underlayer[3]	130 to 70
W/200	Second Underlayer	150 to 50
W/4000-W/6000	Core and Bedding Layer	170 to 30

H = Wave Height
W = Weight of Individual Armor Unit
r = Average Layer Thickness

For concrete armor: [1] Sections III, 7, g, (1), (2) and (6)
[2] Section III, 7, g, (7)
[3] Section III, 7, g, (8)



Recommended Three-layer Section

Rubble-mound section for seaward wave exposure with zero-to-moderate overtopping conditions.

CHAPTER 4

COASTAL AND HYDRAULIC STUDY

4.0 PENGENALAN

Kajian Hakisan Pantai Negara merumuskan bahawa punca utama kejadian hakisan pantai dan kerosakan struktur marin yang serius adalah berpunca daripada lokasi penempatan, perancangan dan rekabentuk projek dan aktiviti pembangunan pantai yang kurang teratur. Akta Kualiti Alam Sekeliling (1974) dan Akta Kualiti Alam Sekeliling (Pindaan) (1985) merupakan undang-undang persekutuan yang mengenakan unsure kawalan perundangan atas kesemua aktiviti pembangunan dari segi kesan yang mungkin akan timbul terhadap alam sekeliling. Perintah Kualiti Alam Sekeliling (Aktiviti Yang Ditetapkan) 1987 menetapkan satu senarai aktiviti pembangunan yang mewajibkan penyerahan laporan Penilaian Kesan Kepada Alam Sekeliling (Laporan EIA) bagi aktiviti-aktiviti yang diwajibkan.(Rujuk Penilaian Kesan Kepada Alam Sekeliling (EIA) – Prosedur dan Keperluan Di Malaysia)

Sehubungan dengan itu, pada tahun 1997 kabinet telah meluluskan penggubalan ‘Garis Panduan Kawalan Hakisan Berikutan Dari Pembangunan Di Kawasan Pantai’ bertujuan bagi mengawal pembangunan di kawasan pantai.

Oleh yang demikian, Unit Maritim juga tidak terkecuali daripada Akta-Akta yang telah termaktub. Pelaksanaan Kajian Hidraulik di dalam skop kerja unit ini adalah berkaitan dengan kesan pembinaan struktur marin (jeti, pemecah ombak), penambakan atau pengorekan dan perlindungan pantai.

Rekabentuk kajian hidraulik perlu mengambil kira data-data alam sekitar seperti keadaan ombak, angin dan arus laut, keadaan topografi dan hidrografi dan pasang-surut air. Maklumat bagi data kualiti air akan diperolehi daripada hasil Kajian Kesan Alam Sekitar (EIA).

4.1 OBJEKTIF KAJIAN HIDRAULIK

Tujuan kajian hidraulik dilaksanakan disenaraikan seperti berikut :

1. Mengumpul dan menganalisa data sediada untuk menentukan samada boleh digunapakai dalam kajian tersebut.

2. Melaksanakan penganalisaan tapak kajian menggunakan model komputer yang bagi mendapatkan situasi semasa dan impak yang terjadi disebabkan kepada kerja-kerja penambakan atau pengorekan pantai dan perlindungan pantai.

3. Mengenal pasti parameter rekabentuk yang akan digunakan terutama bagi kerja-kerja penambakan atau pengorekan, perlindungan pantai dan pembinaan struktur (jeti, pemecah ombak).

4.2 SENARAI MODULE

Module hidraulik yang paling relevan dalam menilai impak sesuatu pembangunan adalah:

1. **Module Hidrodinamik –** meramalkan perubahan paras air dan respon aliran (misalnya keadaan halaju arus di pantai 'nearshore') akibat beberapa punca (pasang-surut dan ombak).

2. **Module Pergerakan Lumpur/Pasir –** meramalkan keadaan hakisan atau pemendapan.

3. **Module Perubahan Garis Pantai –** meramalkan perubahan garis pantai.

4.3 SENARAI MODEL KOMPUTER

Pada masa sekarang terdapat beberapa model dalam pasaran yang boleh digunakan untuk menjalankan kajian hidraulik di kawasan pantai. Beberapa daripada model kajian hidraulik yang berada di pasaran adalah seperti berikut :

1. **TELEMAC** untuk mengkaji arus yang berpunca daripada pasang-surut.

2. **MIKE 21** untuk mengkaji transformasi ombak, arus akibat ombak dan pasang-surut, ‘dispersion’, kualiti air, pergerakan lumpur dan pasir.

3. **SURFACEWATER MODELLING SYSTEM (SMS)** untuk mengkaji transformasi ombak dan pasang-surut, kualiti air, pergerakan Lumpur dan pasir.
   Kelebihan : Dapat memaparkan perubahan permukaan arus dan aliran air berdasarkan kepada perubahan dasar pantai dalam bentuk satu, dua dan tiga dimensi.

4. **UNIBEST** untuk membuat ramalan terhadap perkembangan garis pantai dan mengira kadar pergerakan sedimen untuk kawasan pantai berpasir.

5. **LITPACK** untuk membuat ramalan terhadap arus’longshore’, pergerakan sedimen ‘longhore’ dan perkembangan garis pantai untuk kawasan pantai berpasir.

6. **DIVAST** untuk mengkaji arus akibat ombak dan pasang surut serta dapat menentukan pergerakan sedimen untuk bahan ‘cohesive’ dan ‘non-cohesive’.

7. **REFRAC** untuk digunakan menukar ombak di air yang dalam kepada keadaan ombak di air yang cetek.

Untuk permodelan hidrodinamik model yang dipilih seharusnya berupaya meramalkan keadaan aliran arus akibat ombak dan pasang-surut. Walaubagaimanapun, sekiranya model yang dipilih hanya berupaya meramalkan keadaan aliran arus yang berpunca daripada pasang-surut sahaja, maka pengesahan perlu dibuat bahawa lokasi kajian berada dalam satu kawasan yang terlindung dan keadaan ombak adalah lemah di sepanjang tahun dan dengan itu tidak mempunyai kesan yang ketara terhadap pergerakan arus dan sedimen.

Module pergerakan lumpur/pasir akan memerlukan input ciri-ciri sedimen. Perekabentuk perlu menjalankan penyiasatan sedimen untuk menentukan cirri-ciri sedimen yang akan digunakan sebagai input kepada model.

Halaju arus, kadar pergerakan sedimen dasar dan kadar hakisan atau pemendapan pantai berubah di sepanjang tahun bergantung kepada keadaan ombak, pasang-surut dan lain-lain faktor. Pada amnya, perekabentuk seharusnya membuat kajian pada kes paling serius/dominan bergantung kepada tujuan kajian. Untuk mencapai nilai yang lebih relistik (bila menilai kadar hakisan atau pemendapan tahunan), perekabentuk seharusnya menimbangkan kombinasi kes dominan (ringan, sederhana dan ekstrem) bergantung kepada peratusan ianya berlaku.

Secara amnya untuk permodelan kajian hidraulik, penggunaan Model Komputer *MIKE 21* adalah memadai bagi kerja-kerja penambakan atau pengorekan, perlindungan pantai dan pembinaan struktur (jeti, pemecah ombak). Walaubagaimanapun, sekiranya maklumat-maklumat yang lebih terperinci diperlukan penggunaan model-model yang lain boleh digunakan bergantung kepada peruntukan pelanggan yang mencukupi.

4.3.1 Short Description of the Numerical Model *MIKE 21*

MIKE 21 comprises of several modules. Short descriptions of the modules relevant for the study are given below:

1. ***MIKE 21 HD***

   This is a basic module of the entire ***MIKE 21*** system. It provides the hydrodynamic (HD) basis for the computations performed in most of the other modules. It simulates water level fluctuations and flows in responses to a variety of forcing functions in lakes, estuaries, bays and coastal areas. The water levels and flows are resolved on a rectangular grid covering the area of interest when provided with the bathymetry, bed resistance coefficients, wind and wave field and hydrographic boundary conditions.

2. ***MIKE 21 NSW***

   This is a wind-wave model describing the propagation, growth and decay of short-period and short-crested waves in near shore areas. The model takes into account effects of refraction and shoaling due to vary depths, local wind generation and energy dissipation due to bottom friction and wave breaking. This model is a stationary, directionally decoupled parametric model. The basic equations are solved using Eulerian finite difference technique.

3. ***MIKE 21 MT***

   The model will be used to study the transport, deposition, erosion and consolidation of fine sediments (mud). ***MIKE 21 MT*** is a module in the ***Mike 21*** modeling system for 2D flows developed by DHI. This model is capable in resolving lateral changes in water depth (average cross-sectional depth is used). It will take into account changes in the flow field or wave conditions in 2D. These processes are important for the evaluation of the erosion and siltation.

4. ***MIKE 21 AD***

   The model simulates the transport, dispersion, deposition and erosion of specified sediment fraction under the influence of tides and currents.

4.3.2 List of Model *MIKE 21*

Table 4.1: Senarai *MODEL MIKE 21*

Module	Primary Data	Output
MIKE 21 HD	- Bathymetry - Bed resistance coefficients - Wind data - Wave data - Hydrographic boundary conditions	- Water level fluctuations and flows - Responses to a variety of forcing functions.
MIKE 21 NSW (Wind-wave Model)	- Wind data - Wave data - Bathymetry	- Refraction effects - Shoaling effects - Local wind generation - Energy dissipation
MIKE 21 MT	- Bathymetry - Sea bed sampling - Wave data - Wind data	- Transport - Deposition - Erosion - Consolidation of fine sediment.
MIKE 21 MT (Under influence of tides and currents)	- Bathymetry - Wave data - Wind data	- Transport - Dispersion - Deposition - Erosion
MIKE 21 ST (Non-Cohesive Sediment Transport)	- Bathymetry - Wave data - Wind data - Sea bed sampling	- Transport - Deposition - Erosion - Consolidation of the sediment.
MIKE 21 PP (Pre & Post Processing)	Input from all module	Impact of Development Vs Existing Condition

4.3.3 Methodology of *MIKE 21* Modeling Works

Two dimensional modeling works for the study require several steps and processes which are as follows:

1. Data collection and field investigation
2. General modeling concepts and procedure
3. Defining the hydrodynamic module.
4. Setting up the hydrodynamic (HD) module.

5. Calibrating and verifying the HD Module.
6. Setting up the sediment transport and dispersion (MT and AD) module
7. Performing simulations for the HD, MT and AD models
8. Analysis of simulation results

(Refer *Appendix A*)

4.4 ARRANGEMENT OF REPORT

The arrangement of the report should be include as follows:

1. Introduction
   Introductory material and scope of the Supplementary Hydraulic Report are given.

2. Site Assessment and Data Collection
   To describes the findings from site assessment and the result of the data collection.

3. Coastal Engineering Study
   Work was done in assessing the hydrodynamic influence of the proposal project. This includes the setting-up, simulations and findings of the modeling works.

4.5 SENARAI SEMAKAN BAGI KAJIAN HIDRAULIK

4.5.1 Input Data

Jenis input data bergantung kepada module yang dipertimbangkan dan proses yang akan dinilai . Untuk module hidrodinamik, input data pada amnya terdiri daripada paras pasang-surut, ombak dan bathymetri. Untuk module pergerakan lumpur atau pasir, input ialah ciri-ciri sedimen dan keputusan module hidrodinamik. Untuk module perubahan garis pantai, input ialah ciri-ciri sedimen, ombak dan bathymetri.

Walaubagaimanapun, oleh kerana module hidrodinamik ialah module asas yang perlu dijalankan terlebih dahulu, adalah penting supaya input data untuk module ini disahkan :

a. **Data pasang-surut**

   Diperolehi dengan menjalakan pengukuran sebenar di lokasi yang ditetapkan terlebih dahulu. Pengukuran ini mesti dijalankan untuk tempoh sekurang-kurangnya dua (2) minggu dan termasuk air pasang dan surut. Data berikut mesti ditunjukkan dengan jelas :

- Lokasi/koordinat stesen pasang-surut
- Jenis pengukur pasang-surut yang digunakan
- Data pasang-surut

Data pasang surut ini boleh disemak dengan menggunakan jadual pasang surut terbitan Jabatan Hidrografi Tentera Laut Diraja Malaysia atau data yang diterbitkan oleh negara-negara jiran sekiranya stesen pasang-surut adalah terletak dekat pelabuhan di mana ramalan pasang-surut telah dibuat.

Dilaksanakan ???	**Ya**	**Tidak**

b. **Data ombak**

Boleh diperolehi daripada pangkalan data SSMO atau pun teknik ‘hindcasting’ atau lain-lain sumber. Sekiranya data adalah dalam bentuk data asal, maka perekabentuk mesti menjalankan:

- Satu analisa statistik terhadap kesemua ombak dalam kawasan terlibat untuk menentukan ombak dalam ketara *(significant wave)*.
- Menjalankan transformasi ombak untuk menentukan ciri-ciri ombak di kawasan *nearshore.*

Sekiranya teknik ‘hindcasting’ digunakan, maka perekabentuk mesti :

- Mendapatkan gambarajah *Wind Rose Chart (*daripada Perkhidmatan Kajicuaca Malaysia)
- Menentukan *Critical Fetch Lengths.*
- Menggunakan teknik ‘hindcasting’ untuk menentukan *deepwater significant wave.*
- Menjalankan transformasi ombak untuk menentukan ciri-ciri ombak di kawasan *nearshore.*

Pada amnya, ‘significant wave height’ di sepanjang pantai barat Semenanjung Malaysia adalah tidak melebihi 2 m, manakala di sepanjang pantai timur Semenanjung Malaysia dan Sabah dan Sarawak adalah tidak melebihi 4m. Walaubagaimanapun, mungkin terdapat kes tertentu di mana ketinggian ombak mungkin berlainan.

Dilaksanakan ???	**Ya**	**Tidak**

c. **Bathymetri**

Untuk kerja pemodelan biasanya diperolehi daripada *Carta Admiralty* untuk kawasan laut yang dalam manakala satu kerja ukur hidrografi dijalankan untuk kawasan *nearshore.* Bathymetri yang digunakan boleh disahkan daripada *Carta Admiralty* dan/atau maklumat ukur hidrografi yang lain.

Dilaksanakan ???

Ya	**Tidak**

4.5.2 Penentukuran

Penentukuran model biasanya dijalankan dengan membandingkan paras air dan halaju arus pada *spring* dan *neap tide* yang diramalkan dengan nilai yang diukur dan seterusnya membuat pembetulan yang perlu kepada beberapa parameter model supaya dua set data ini akan mencapai persetujuan yang sebaik mungkin. Pengukuran paras air mesti dijalankan untuk sekurang-kurangnya dua (2) minggu dan termasuk *neap* dan *spring tides.* Pengukuran halaju mesti dijalankan untuk sekurang-kurangnya 3 hari semasa *spring* dan *neap tides.* Data seperti berikut mesti ditunjukkan dengan jelas :

- Lokasi/koordinat stesen penentukuran;
- Nilai parameter model yang telah diperbetulkan (misalnya kekasaran, *eddy viscosity dll)*
- Gambarajah beza paras air (menunjukkan perbandingan paras air untuk nilai ramalan dan yang terukur)
- Gambarajah beza halaju (menunjukkan perbandingan halaju dan arah untuk nilai ramalan yang terukur)

Dilaksanakan ???

Ya	**Tidak**

4.5.3 Verifikasi

Verifikasi model dijalankan dengan membandingkan paras air dan halaju yang diramalkan pada set masa dan lokasi yang berlainan. Stesen verifikasi mesti cukup jumlahnya dan lokasi mereka mesti berdekatan dengan tempat yang perlu dikaji (kawasan di mana impak mungkin wujud) dalam kajian. Paras air mesti diukurkan untuk sekurang-kurangnya dua (2) minggu manakala pengukuran halaju arus mesti dijalankan untuk sekurang-kurangnya 3 hari semasa *spring* dan *neap tides*. Data seperti berikut mesti ditunjukkan dengan jelas :

- Lokasi/koordinat stesen penentukuran;
- Nilai parameter model yang telah diperbetulkan (misalnya kekasaran, *eddy viscosity dll)*
- Gambarajah beza paras air (menunjukkan perbandingan paras air untuk nilai ramalan dan yang terukur)
- Gambarajah beza halaju (menunjukkan perbandingan halaju dan arah untuk nilai ramalan yang terukur)
- Jenis peralatan yang digunakan.

Perbezaan antara halaju dan arah mesti tidak melebihi 30% dan 45 ^{o}. Perbezaan antara paras air mesti tidak melebihi 20%. Paten am untuk halaju dan kelajuan seharusnya sama.

Dilaksanakan ???	**Ya**	**Tidak**

4.5.4 Pelaksanaan Model

Model harus dijalankan untuk kes <u>sebelum projek</u> dan <u>selepas projek.</u> Kes selepas projek boleh merupakan satu larian yang telah mengambil kira kesemua ciri-ciri yang dicadangkan atau pun sekiranya cadangan alternatif dipertimbangkan, model mesti dijalankan untuk setiap alternatif.

Module yang mesti dijalankan boleh terdiri daripada:

- **Module Hidrodinamik –** menunjukkan paten halaju
- **Module Pergerakan Lumpur/Pasir** – menunjukkan paten hakisan atau pun pemendapan
- **Module Perubahan Garis Pantai**

Module pertama tidak memerlukan sebarang input berkaitan dengan saiz sedimen. Maka ia boleh digunakan untuk kawasan berlumpur atau berpasir. Module kedua akan dipilih untuk menampung pantai berlumpur atau pantai berpasir. Module ketiga hanyalah untuk pantai berpasir sahaja.

Dilaksanakan ???	**Ya**	**Tidak**

4.5.5 Rumusan dan Cadangan

Kajian mesti mengenal pasti kesemua impak yang mungkin berlaku terhadap kawasan bersebelahan akibat projek yang dicadangkan seperti :

- Hakisan atau pemendapan (menunjukkan kawasan yang terlibat untuk jangka pendek dan jangka panjang)
- Masalah saliran.
- Pemendapan salur pelayaran.

Ia mesti juga mencadangkan langkah tebatan yang berkesan untuk mengatasi kesemua impak yang telah dikenalpasti.

Dilaksanakan ???	**Ya**	**Tidak**

**Catatan : Sumber Ibu pejabat JPS Malaysia.*

4.6 REFFERENCES

1. Guidelines For Preparation Of Coastal Engineering Hydraulic Study And Impact Evaluation (JPS); Fifth Edition, December 2001.

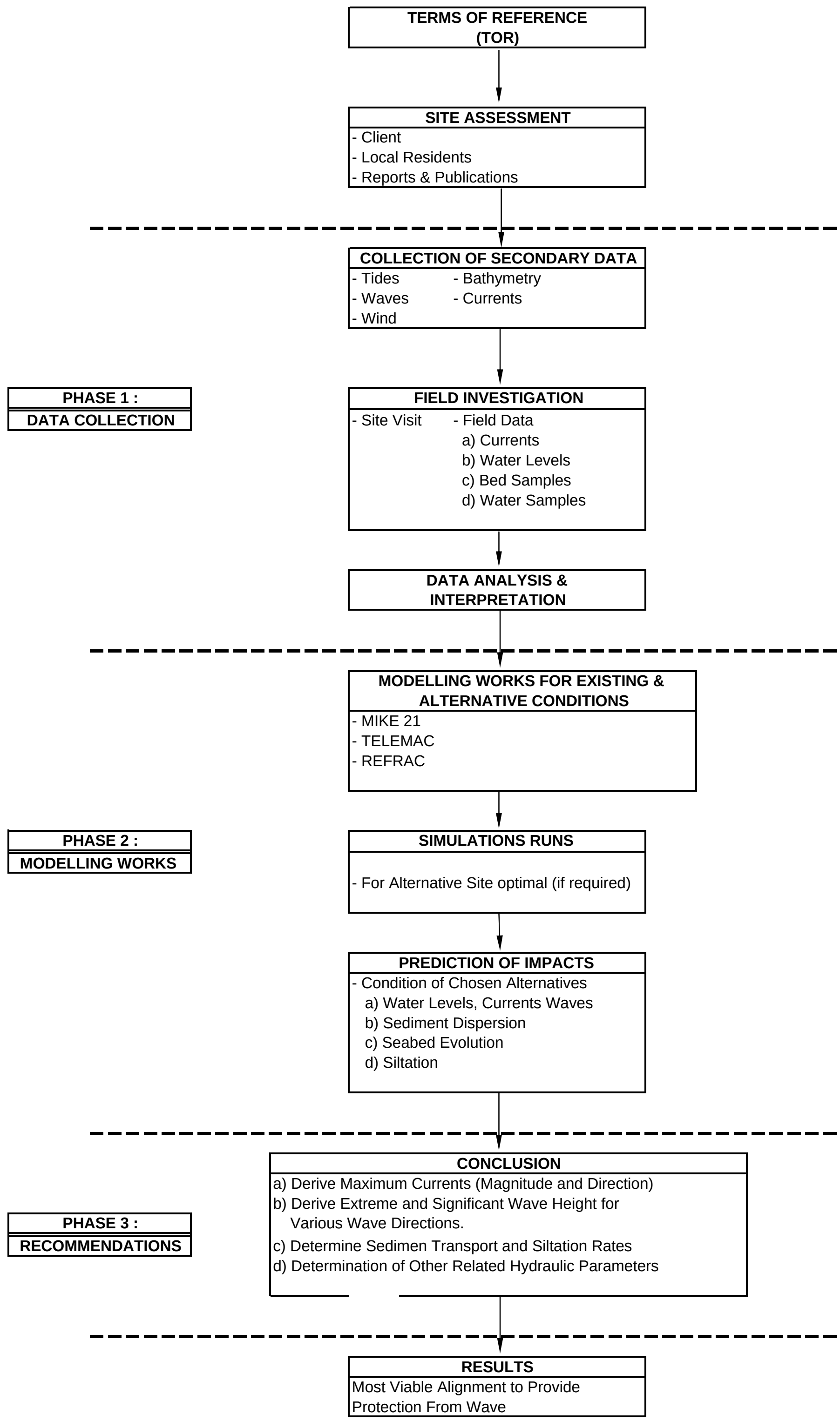
2. Guidelines on Erosion Control for Development Projects in The Coastal Zone (JPS), December 2001.

3. Penilaian KesanKepada Alam Sekeliling (EIA) – Prosedur dan Keperluan Di Malaysia. Cetakan Pertama 21 Oktober 1994.

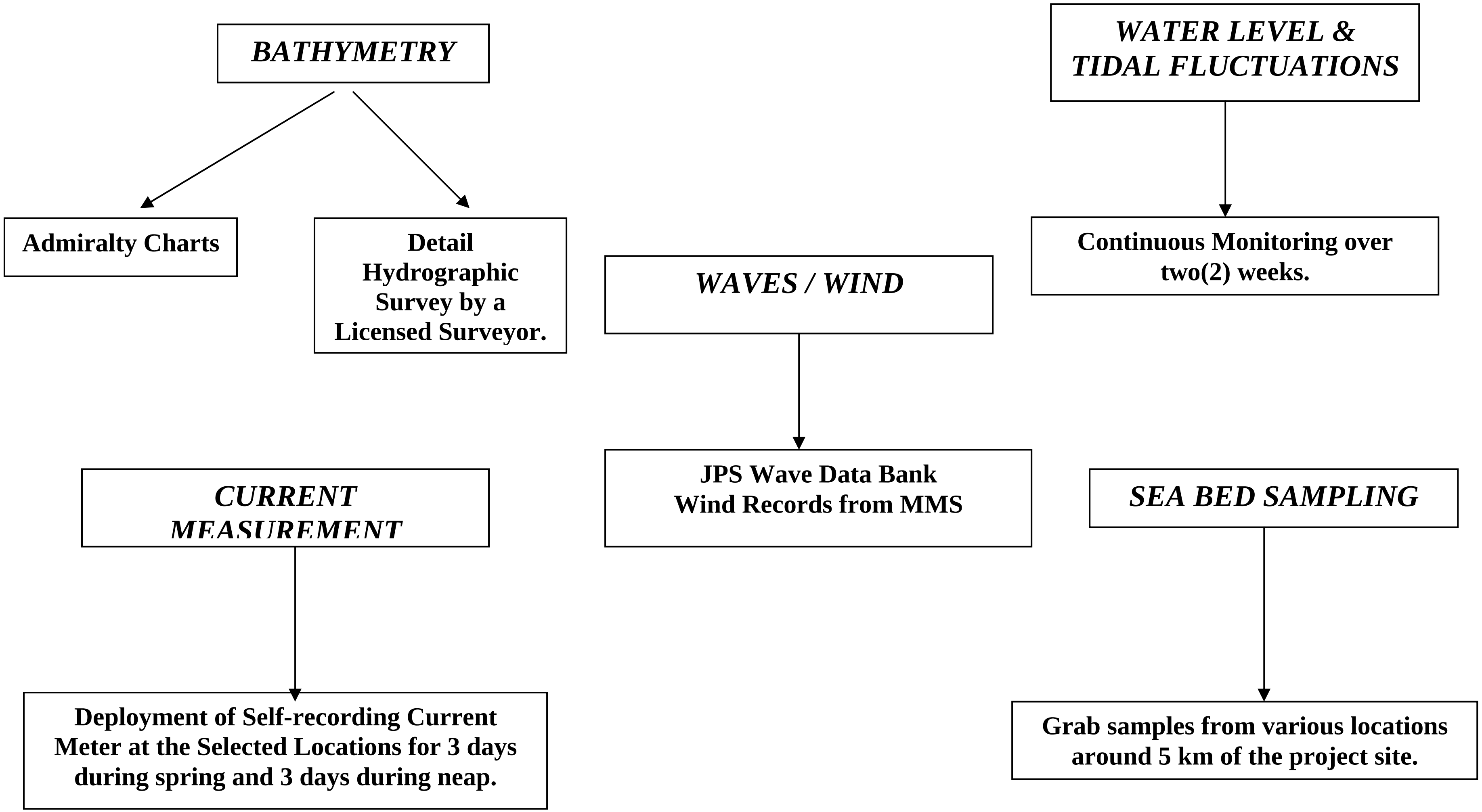
4. Surat Pekeliling Am Bil.5 Tahun 1987, Jabatan Perdana Menteri.

APPENDIX

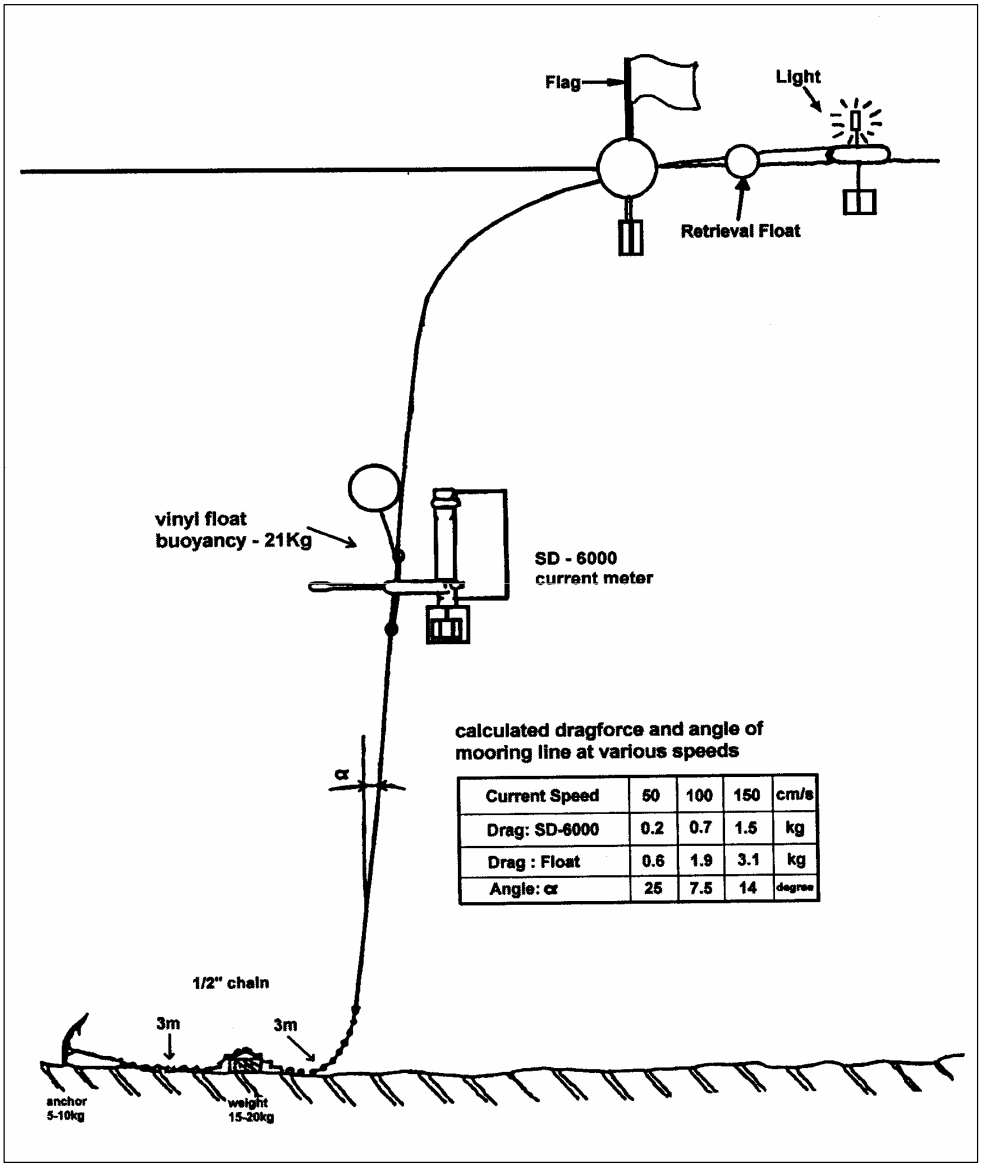
APPENDIX A: FLOW CHART FOR THE HYDRAULIC STUDY



APPENDIX B: DATA COLLECTION



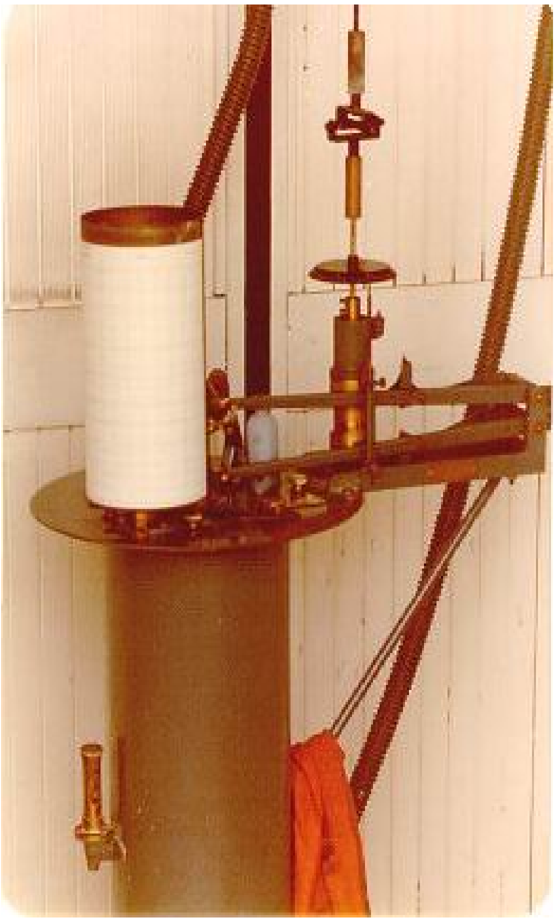
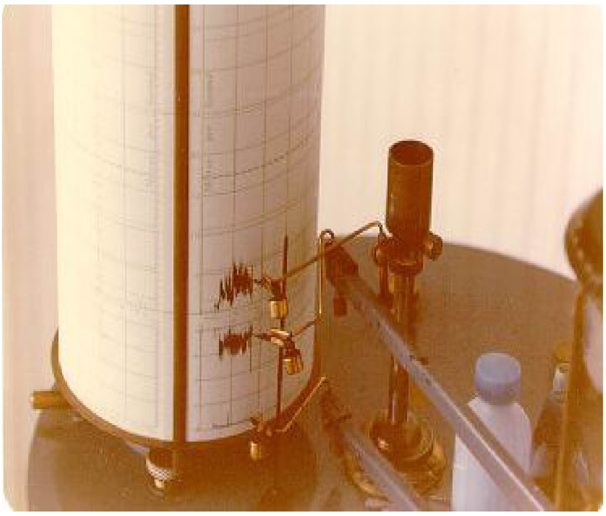
APPENDIX C: MOORING ARRANGEMENT OF A CURRENTMETER



Current Speed	50	100	150	cm/s
Drag: SD-6000	0.2	0.7	1.5	kg
Drag : Float	0.6	1.9	3.1	kg
Angle: α	25	7.5	14	degree

MOORING ARRANGEMENT OF A CURRENTMETER

APPENDIX D: WIND RECORDER (ANEMOMETER)

ANEMOMETER

The anemometer is used to measure wind direction and speed.

APPENDIX E: DELIVERABLES AND REPORTING

DELIVERABLES AND REPORTING	
Report	**Contents**
Inception Report (10 copies)	- Proposed Study Programme - Methodology - Assesment of Data Availability - Recommendations for Additional / necessary data
Interim Report (10 copies)	- Study Area Description - Preliminary Analysis of the Coastel Processes - Interpretation of Modelling Results
Draft Final Report (10 copies)	- All Works Performed in the Study Including Recommendations for Most Optimal Site, Potential Impacts, Mitigating Measures, EMP and Preliminary Design Parameters
Final Report (15 copies)	All Revisions given by the Government on the Draft Final Report Deemed Appropriate to be incorporated in the Final Report.(Within 2 weeks)

PROPOSED WORK SCHEDULE					
	Month 1	**Month 2**	**Month 3**	**Month 4**	**Month 5**
Desk Study / Data Collection Programme					
Data Analysis					
Modelling Work					
Analysis of Output and Recommendations					
Report / Documentation		◆		◆	◆

◆ Report