as arches and other roof-supporting members in covered structures. Plywood on the other hand is used both during the construction phase (as temporary concrete formwork, for example) and in the final product as decking or siding. It is made up of thin layers of wood glued together with the fibers arranged cross-wise for maximum strength. Plywood is commonly available in 4 by 8 ft (1.2 by 2.4 m) panels of varying thicknesses (0.25 to 1 in., 6 to 25 mm). Gordon (1979) recommends that where plywood will be exposed to moisture as in the case of marine structures, an exterior type should be used which is bonded with 100 percent waterproof glue.

4.4 ALUMINUM

In spite of its high cost, aluminum is becoming increasingly popular as a structural material on the waterfront. While aluminum and aluminum alloys have high strength and low weight as their prime assets, they are also very corrosion resistant because of an oxide layer that quickly forms upon exposure (Patton, 1976). Since aluminum is easily formed and machined, many attractive finishes are possible. Aluminum is used chiefly in prefabricated, modular construction where the parts can be built in the controlled atmosphere of a shop. Welding of aluminum must be done by the heliarc process which requires special welding equipment not usually available to most contractors, especially in the field.

Aluminum presents some special challenges to the designer with respect to repeated stress. As Figure 4.15 illustrates, aluminum parts (especially welded members) must be designed against fatigue failure since the material does not show a true endurance limit (Patton, 1976). Electrolysis may also be a problem if a dissimilar metal (such as a
Figure 4.15  Typical Fatigue Curves of Aluminum Alloys (Patton, 1976, p. 267)
boat hull) contacts the aluminum for any length of time. Electrolysis and corrosion are discussed by Hubbell and Kulhawy (1979a).

Pure aluminum is not often seen as a structural material since it is soft and relatively weak (ultimate tensile strength only \(1 \times 10^4\) psi or 68.9 MN/m\(^2\)). On the other hand, in the alloy form strengths, as high as \(7.5 \times 10^4\) psi (517 MN/m\(^2\)) are possible. The most common alloys are the 5000 series (magnesium) for sheet and plate members, and the 6000 series (magnesium-silicon) for extrusions. The properties of a low alloy commercial aluminum (99% pure) are presented in Table 4.7.

4.5 WROUGHT IRON

Wrought iron consists of grains of pure iron interspersed with filaments of iron silicate slag (Chaney, 1961). The grain structure is such that the individual crystals are visible to the naked eye on a fractured surface. While many designers consider wrought iron a material of the past, its excellent resistance to corrosion makes it a viable product even when used in saltwater.

The corrosion resistance of wrought iron is two-fold. First, the protrusion of the silicate threads through the surface enables wrought iron to retain a coating of corrosion that serves to protect the base metal. In the case of steel, this layer of "rust" easily falls off to allow continuing corrosion. Secondly, the rough surface typical of wrought iron members causes them to retain a heavier coating of zinc (if galvanized) or other coating than equivalent steel members, resulting in longer design lives.

Because of its grain structure, wrought iron has directional physical
Table 4.7 Properties of Commercial Aluminum (Cordon, 1979, p. 214)

<table>
<thead>
<tr>
<th>Alloy and temper</th>
<th>Tension</th>
<th>Hardness</th>
<th>shear</th>
<th>Fatigue</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Yield strength at 0.2 percent offset, MPa</td>
<td>Ultimate strength, MPa</td>
<td>Elongation in 2 in., percent</td>
<td>Brinell 500-kg, 10-mm ball</td>
</tr>
<tr>
<td>2S-O</td>
<td>34.5</td>
<td>89.7</td>
<td>35</td>
<td>45</td>
</tr>
<tr>
<td>2S-H12</td>
<td>89.7</td>
<td>103.4</td>
<td>12</td>
<td>25</td>
</tr>
<tr>
<td>2S-H14</td>
<td>96.6</td>
<td>117.2</td>
<td>9</td>
<td>20</td>
</tr>
<tr>
<td>2S-H16</td>
<td>117.2</td>
<td>137.9</td>
<td>6</td>
<td>17</td>
</tr>
<tr>
<td>2S-H18</td>
<td>144.8</td>
<td>165.7</td>
<td>5</td>
<td>15</td>
</tr>
</tbody>
</table>

† Temper designation: O, annealed; H18, fully cold-worked (hard); H12, H14, H16, intermediate degrees of cold work between O and H18.
§ Based on 500 million cycles, using R. R. Moore type of rotating-beam machine.
characteristics much like wood. During the rolling and forming process, the slag inclusions become oriented in the longitudinal direction or the direction of rolling. The material therefore has greater tensile strength on the longitudinal axis and is more easily bent (more ductile) in the longitudinal direction than the transverse direction. In general, the physical properties of wrought iron are approximately the same as pure iron (Cordon, 1979). Wrought iron typically has a maximum carbon content of 0.35 percent while medium-high carbon steels range from 0.35 to 1.5 percent (Patton, 1976). Primarily because of its ductility, wrought iron is commonly formed into pipes, plates, sheets, bars, angles and channels.

4.6 SUMMARY

The materials used in the construction of coastal structures must be strong and durable. The waterfront is a very harsh environment for man-made structures, and their success is directly a function of the properties of the materials involved.

The primary coastal construction materials are concrete, steel, and wood. Aluminum and wrought iron are used less frequently. The advantages and disadvantages of each of these materials have been discussed.

Concrete properties such as durability, permeability, and abrasion resistance are more critical than strength. A mix designed for the former considerations is usually assured of adequate strength. The water-cement ratio of the paste and the soundness of the aggregate are the most important parameters determining concrete quality. Steel is a very common material used in civil engineering, and its properties
are well known. Steel must be protected from rapid corrosion for it to reach an acceptable design life. As a natural material, wood and its engineering properties can be quite variable. Wood properties depend on species type, and the presence of defects or imperfections. Grading rules have been developed to reduce the clear wood strength properties because of these defects and to determine working strengths to be used in design. Aluminum is gaining acceptance as a coastal construction material because of its light weight and high strength. It is now used primarily as a material in prefabricated dock systems. Wrought iron consists of pure iron with internal filaments of silicate slag. It has very high corrosion resistance and is one of the preferred materials for hardware and fasteners.
CHAPTER 5
SOLID FILL TYPE DOCKS, PIERS AND WHARVES

Docks, piers and wharves constructed of a natural or artificial fill surrounded by a vertical wall are considered solid fill structures. While there are many variations, anchored bulkheads are the most common wall type. Others include cantilever sheet pile walls, cantilever "I" walls, gabion walls, crib walls, cellular sheet pile walls, concrete caisson walls, and walls supported by relieving platforms. Each wall type is suitable for different applications depending on the required depth of water, character of the foundation material, loads imposed, and the allowable movement once it is put in service. Basin depth depends on boat size and berth layout plan, topics that were discussed in Chapter 2. Soil properties and the loads soils impose on a retaining wall are addressed by Saczynski and Kulhawy (1982). Wall structures are also described by Ehrlich and Kulhawy (1982) with regard to their use for erosion control and wave protection.

This chapter presents a brief discussion of each wall type, the factors involved in selecting the proper wall type for a particular site or application, and the design considerations pertaining to the use of solid fill structures for docks, piers and wharves.

5.1 WALL TYPES

Anchored Bulkheads and Cantilever Sheet Pile Walls

Anchored bulkheads consist of a row of interlocked sheet piles, stiffened across the face by wales and restrained from moving away from the fill by tie-rods connected to anchors (Figure 5.1). A cantilever
Figure 5.1 Anchored Bulkhead Wall (Saczyński and Kulhawy, 1982, p. 8)
sheet pile wall differs from an anchored bulkhead in that it does not have
an anchor system and depends for stability on its embedment and sometimes
heavier cross-section (Figure 5.2). The cantilever wall is better suited
to relatively shallow water or sites where anchorage is poor (Saczynski
and Kulhawy, 1982). Traditionally, anchorage for bulkheads is obtained
from deadmen, braced piles, sheet piles or footings located in the back-
fill. In the case of relatively narrow solid fill piers, the tie rods
extend through the fill to the adjacent bulkhead. This removes the need
for an anchor system, but the wall must be analyzed further for stability
against tilting as a unit. Anchored bulkheads and cantilever sheet pile
walls may be designed as described by Saczynski and Kulhawy (1982).

**Cantilever "L" Walls**

The cantilever "L" wall consists of a concrete stem and concrete
base slab (Figure 5.3). Both the stem and slab are relatively thin and
are steel reinforced to resist the moments and shears to which they are
subjected (Peck, Hanson, and Thornburn, 1974). Cantilever "L" walls
have not found widespread application as bulkheads but are often used in
conjunction with relieving platforms (Chaney, 1961). Where the design
finds them a viable alternative, these walls may be analyzed and designed
using methods described in texts on soil mechanics or foundation engineer-
ing. Peck, Hanson, and Thornburn (1974) and Terzaghi and Peck (1967)
are useful references for rigid retaining walls.

**Gabion Walls**

Gabions (Figure 5.4) are low cost structural walls that offer several
advantages, including: (1) flexibility, allowing them to adjust to
Figure 5.2 Cantilevered Sheet Pile Wall (Saczynski and Kulhawy, 1982, p. 14)
Figure 5.3 Typical Pile Supported "L" Wall for Weak Soils.
(Dunham and Finn, 1974, p. 103)
Figure 5.4 Use of Gabions in Marina Construction (Dunham and Finn, 1974, p. 91)
foundation irregularities and settlement, (2) versatility, or the capability to be placed and filled under water with minimal problems, and (3) permeability, preventing the development of a hydrostatic head in the backfill (Hubbell and Kulhawy, 1979a). Gabion assembly is labor intensive and requires the rock fill to be hand placed before the lids are "sewn" shut with wire. Protection in the form of a fender system or facing material is a must for gabion walls since the wire mesh is susceptible to damage by impact which could allow the rock fill to spill out. When used to support a solid fill structure, it is common practice to cap the wall with a concrete slab (Bekaert Gabions, 1977) which may dramatically reduce flexibility. Gabions as a coastal material are discussed by Hubbell and Kulhawy (1979a) while Ehrlich and Kulhawy (1982) address the use of gabions in coastal protection structures.

Crib Walls

Rock filled crib walls constructed of timber (Figure 5.5) or precast concrete elements act in much the same manner as gabions. They can withstand considerable racking and settlement without rupture, and are permeable enough to relieve excess hydrostatic stress in the backfill. According to Quinn (1972), rock-filled timber cribs were used extensively on the Great Lakes for early construction of piers and wharves. When timbers are used for the cribbing, the wall is usually terminated at low water level and the wall above is constructed of concrete. In this manner, the wood remains saturated and is less susceptible to borers and natural deterioration. Standard designs for pressure-treated timber cribs have been suggested by the American Wood Preservers Institute (1969).
Figure 5.5: Typical Timber-Crib Wharf
(Quinn, 1972, p. 279)
Cellular Sheet Pile Walls

One variation of the conventional steel sheet pile wall is the cellular wall type illustrated in Figure 5.6. The cellular sheet pile wall possesses a high degree of stability in conditions where anchored bulkheads are impractical. It should be considered for dock, pier and wharf construction where the water depth is greater than the feasible anchored bulkhead height, or where sufficient penetration may not be obtained because of shallow bedrock (Cummings, 1957). To avoid stability problems or excessive settlement when used on soft materials, predredging and placement of a foundation mat may be necessary. A stability analysis for cellular sheet pile walls should include sliding along the base, overturning as a unit, and rupture of the web and interlocks (USCOE, 1963). Circular cells connected by intermediate arcs are used more often than the diaphragm type wall (Figure 5.7) since each individual cell may be filled independently of the others and is stable in itself (Quinn, 1972). Cummings (1957) notes that the main cells of a circular cellular wall increase in diameter about 1.5 percent when the pressure of the fill takes up the slack in the sheet pile interlocks. The connecting arcs may then bulge outward beyond the bulkhead line causing construction problems with the deck and fender system. Locating the connecting arcs such that their tangent is about 2 ft. (0.6 m) back from the deck line is a solution recommended by Cummings (1957).

Concrete Caisson Walls

A caisson wall is composed of a row of reinforced concrete shells that are floated into position, sunk, and filled with a granular material. Figure 5.8 illustrates a closed bottom caisson resting on a prepared,
Figure 5.6 Cellular Sheet Pile Wharf
(Quinn, 1972, p. 277)

Figure 5.7 Sheet Pile Arrangement for Cellular Walls
(Cummings, 1957, p. 1366-2)
Figure 5.8  Concrete Caisson Wharf  
(Quinn, 1972, p. 280)
level foundation mat. Open well caissons with cutting edges that obtain support by sinking into a soft bottom are also used (Quinn, 1972). Caissons are usually designed so that their tops lie just above the low-water level. A cast-in-place concrete cap forms the upper part of the dock face, allowing true alignment and grade as well as providing for the attachment of the fender system, cleats, railings, and other hardware. One of the advantages of concrete caissons is that much of the construction work is performed on land for ease of access. In addition, construction is much less dependent on weather and wave conditions.

Relieving Platforms

A relieving platform type bulkhead combines many of the features of walls previously discussed into one system. As Figure 5.9 illustrates, it consists of a concrete wall resting on a pile supported timber platform. A line of sheet piling retains the soil behind the bulkhead while rip-rap under the platform provides stability. The relieving platform is suitable for greater water depths and softer underlying material than are sheet pile walls (Chaney, 1961). To minimize deterioration and prolong its life, the timber members of the relieving platform should be located at or near the low-water level so that they are continuously wet. The rip-rap acts to reduce the stresses in the sheet pile wall while at the same time protecting against loss-of-ground from scour. In addition, its sloped and porous surface absorbs wave energy and creates a calmer berthing environment. Depending on the geometry of the face of the platform, problems can arise because of air pressure that causes
Figure 5.9 Relieving Platform Type Wharf
(Quinn, 1972, p. 270)
structural damage and rip-rap instability (Leitass, 1979). Figure 5.10 illustrates the effect of wave characteristics on this air pressure buildup, while Figure 5.11 shows the reduction of air pressure in relation to relief hole area. Remedial measures include reducing the wave energy with protective structures, resisting the air pressure by stronger platform design, and arranging for air relief. While relieving platforms are the most desirable wall type with respect to permanence and stability, they are also the most costly to construct (Chaney, 1961).

5.2 SELECTION OF WALL TYPE

Each of the wall types discussed above has been constructed and has performed effectively in harbors around the world. None of the wall types are universally applicable to any given location, however. In addition to sound design, construction and maintenance practices, a successful installation requires that the wall be well-suited to the site conditions and its intended application. The designer should consider the following factors when selecting a wall type to be used at a particular location (after Chaney, 1961):

1. **Water depth.** The basin depth at the face of the bulkhead in most marinas ranges from 8 to 12 ft (2.5 to 3.5 m) (See Chapter 2). For this wall height, anchored bulkheads will be the most economical wall type, given sufficient embedment and anchorage for stability.

2. **Soft Substrata.** When the substrata is composed of layers of soft sediments, piles driven to "refusal" will show less settlement than gravity structures such as crib walls or concrete caissons.
AIR PRESSURE INCREASES AS WATER LEVEL RISES REACHING MAX. VALUE AT SOME ELEVATION ABOVE L.W.L. AFTER WHICH IT GRADUALLY DECREASES UNTIL THE HIGH W.L. IS REACHED.

A) INFLUENCE OF WATER LEVEL

B) INFLUENCE OF WAVE PERIOD

C) INFLUENCE OF WAVE HEIGHT

Figure 5.10 Effect of Wave Characteristics on Air Pressure Buildup (Leitass, 1979, p. 1120)
Figure 5.11 Reduction of Air Pressure in Relation to Percentage of Air-Hole Area (Leitass, 1979, p. 1122)
3. **Hard Substrata.** When a dense layer of soil or rock lies at a shallow depth below the dredging line, piles may not penetrate far enough for adequate horizontal stability, and concrete caissons or filled cribs may be more suitable.

4. **Settlement.** The use of gravity walls (rock filled cribs and concrete caissons) causes high contact stresses on the foundation. When placed on relatively soft underlying materials, these walls are subject to settlement and horizontal slippage that may result in damage to walks, buildings, and other structures resting on them.

5. **Preadredging.** In extreme cases, it may be necessary to dredge soft foundation materials and replace them with a bedding layer of sand and gravel. This technique will reduce settlements in gravity walls, and assure adequate anchorage for sheet pile stability. Densification of bottom materials may also be achieved by loading with a layer of rip-rap.

6. **Berthing Access.** The use of a relieving platform with a line of sheet piles driven landward of the platform, or sheet piling alone, driven at the face of the bulkhead will permit dredging to full project depth up to the face of the wall. On the other hand, sloping rip-rap and some crib walls will encroach considerably into the water area and prevent boats from berthing along the wall.

7. **Materials.** The durability and disintegration of materials when subjected to alternate wetting and drying should be
considered when selecting a wall type. For material properties see Chapter 4. Materials are also discussed by Hubbell and Kulhawy (1979a).

5.3 GEOTECHNICAL DESIGN CONSIDERATIONS

Perimeter walls in small craft harbors are seldom used as breasting docks permitting boats to tie up parallel to the wall face (Dunham, 1969). Except for small scale projects or private installations, breasting is an inefficient use of dock space. Bow clamps and stern hooks have been used to moor small craft perpendicular to a perimeter wall, but they are inconvenient to use and pose boarding problems.

Generally, solid fill structures are used to stabilize the boundary walls of a harbor and provide anchorage and access to either a fixed or floating berthing system. In some locations, fire and safety regulations require that the fuel dock be of solid fill construction (See Chapter 8 on Utilities and Services). Where solid fill docks, piers and wharves are to be used, some areas of design deserve special attention. The following discussion addresses foundation design, dredging, and backfill considerations for solid fill walls.

**Foundation Design Considerations**

Foundation design is concerned with the interaction between a structure and the material it rests upon. In the case of waterfront structures such as docks, piers, and wharves, the underlying material usually consists of layers of sand and clay. The in-place or undisturbed density of these marine sediments is often quite low since they are deposited under water in a very loose condition. Although the engineering behavior
of clay in general is very complex, it should be sufficient for this
discussion to note that marine clays are often weak and highly com-
pressible. Sands, on the other hand, are much less compressible and
can be easily densified through vibration. In practice, soils range
continuously from fine-grained (clay) to coarse-grained (sand) sizes.
Since the engineering properties of a soil are highly dependent on
grain size and in-situ condition, a geotechnical investigation is usually
performed to characterize the soil type, extent, and expected behavior.
The scope of such an investigation depends primarily on the scale of
the project and the discretion of the designer.

The character of the underlying soil is an important factor influenc-
ing the stability and settlement of a foundation. In addition, scour
potential is determined by the soil type to be transported as well as
the energy available to move it. Ideally, foundation design is intended
to protect structures from failure because of a lack of bearing capacity,
excessive settlement, rapid scour, or combinations of these. Unfor-
tunately, foundation design is often minimized in coastal structure
design, resulting in problems that are difficult and costly to repair.
The following discussion briefly addresses each of the possible failure
modes with respect to waterfront design. It is not within the scope
of this report to go into the details of geotechnical analysis; the
reader should refer to texts on soil mechanics and foundation engineering
for this information.

**Bearing Capacity.** Bearing capacity refers to the ability of the
foundation to carry a load without failure within the soil. Failure
usually occurs because of shearing of the underlying strata and backfill along a curved surface (Figure 5.12). Stability of a sheet pile wall depends on the depth of embedment; greater embedment depth forces the failure surface to go deeper and thereby mobilizes more resistance. Saczynski and Kulhawy (1982) present the procedures for analysis and design of anchored bulkheads and cantilever sheet pile walls. The stability of gravity walls such as concrete caissons, cribs and gabions is dependent on the size of the base and the wall weight, and may be enhanced by the placement of bedding layers.

According to Quinn (1972), the bedding layer should extend beyond the toe and the critical plane of failure so that its weight and strength increase the factor of safety with respect to a shear failure at the toe (Figure 5.13). A properly designed bedding layer will reduce settlement by spreading out the wall load to decrease its contact pressure below, provide a leveling course that facilitates construction, and protect the foundation material against scour. Foundation blanket design is addressed by Ehrlich and Kulhawy (1982).

Stability against a bearing capacity failure can only be determined through a detailed geotechnical analysis. The approach commonly used is to analyze a number of possible failure planes and determine which is likely to be critical. The conservative assumptions of a fully saturated backfill and extreme low water at the face of the wall are made to simulate the worst expected service condition. A more critical state can be created during construction if poorly administered hydraulic fills are used in conjunction with dredging in front of the wall. According to the Committee for Waterfront Structures (1966), a temporary lateral pressure may exist with an
Figure 5.12 Deep-seated Failure of a Retaining Platform Because of Insufficient Bearing Capacity of Underlying Weak Soil (Dunn, Anderson, and Kiefer, 1980, p. 237)
Figure 5.13 Granular Bedding Layer Used to Increase the Bearing Capacity of an Underlying Weak Soil
intensity somewhere near the hydrostatic pressure of a material with the density of the slurry and the earth pressure at rest of the consolidated hydraulically filled soil. The actual pressure will depend on the degree of consolidation the fill has reached.

**Settlement.** Settlement relates to the downward movement of a structure during and after construction. The two major causes of settlement of waterfront structures are the consolidation of weak, compressible soils in the foundation and the removal of supporting soil from scour. Scour related settlement is discussed in a subsequent section.

Settlement is not always detrimental to solid fill docks, piers and wharves. Uniform settlement can be tolerated as long as the wall remains functional and buried utilities are not damaged. On the other hand, differential settlement from compressible strata of irregular thickness can easily result in structural damage to the wall that will lead to complete failure. Some wall types, notably gabion and timber crib walls, are more resistant to differential settlement and racking than are rigid walls. While a deformed wall may be structurally sound, its appearance can deter users such that it constitutes a functional failure. Although good foundation design cannot eliminate settlement, its magnitude may be reduced and its effects mitigated so that it is no longer harmful to the structure.

Consolidation settlement is a time-dependent phenomenon that occurs when a surcharge load is placed above a layer of soft substrata. One method of controlling this settlement is to place a temporary surcharge to "preconsolidate" the soil. After consolidation is complete, the surcharge is removed and is replaced by a wall structure and backfill. A
disadvantage of preconsolidation is that substantial time (measured perhaps in years) is necessary for completion, especially if the foundation materials are fine-grained with low permeability. An alternative suggested by Quinn (1972) is excavation and replacement of the compressible layer with a more competent material. Foundation mats are commonly used beneath gravity walls to provide a stable base for construction and minimize settlement. Design of these mats is presented by Ehrlich and Kulhawy (1982).

**Scour.** Bulkheads must be both vertical and smooth-faced to serve properly as a dock, pier or wharf. Unfortunately, such a barrier is a very efficient reflection of wave energy and is accepted as the primary cause of bed scour. Since scour potential is greatest at the toe of a wall where its smooth face meets the foundation material, progressive excavation will take place until undermining, stability and settlement problems occur. The situation may be aggravated if excess hydrostatic pressures are allowed to build up in the backfill as in the case of hydraulic fill placement. Water will then flow along a path leading under the toe of the wall and cause a reduction in the soil strength and resistance to erosion.

When scour-induced erosion is expected to be a problem, protection is commonly provided in the form of a foundation blanket. While the blanket serves as a mat to distribute wall loads over a larger area and reduce settlement and bearing capacity problems, it must also be designed as a filter to avoid the loss of fines through its voids. The mechanism
of scour and protective measures including foundation blanket design are addressed by Ehrlich and Kulhawy (1982). Additional references that should be consulted include Hale (1980) on site-specific scour problems, scour control methods and construction techniques, and Keown and Dardeau (1980) on filter fabrics and filter design criteria.

Dredging

While the general topic of dredging is presented later in this report (see Chapter 9), some of the aspects of dredging that relate to solid fill structures should be mentioned here. The dredging process occurs in two phases. The first or initial phase is performed when the harbor basin is originally excavated for navigation. Dredging must precede placement of a foundation mat for gravity type walls. In the case of sheet pile walls, however, Szczynski and Kulhawy (1982) recommend that dredging operations be conducted after wall construction is complete and the backfill has been placed and consolidated. This delay allows arching to occur in the backfill that will reduce the stress level in the wall and result in less outward deflection.

The second or maintenance phase of dredging must be carefully administered to avoid over-dredging and hitting the wall. Over-dredging adjacent to the wall should not be allowed since excavation of material below the original design depth will result in a loss of toe support and possible stability problems. Depending on the dredge method used, it is relatively easy to damage bulkheads structurally by hitting them. The dredge operator must exercise caution and proceed more slowly than usual.

Another important aspect of dredging is the disposal of the excavated
material. Assuming the soil is acceptable backfill material, using it for fill behind the wall is obviously more efficient than wasting it away from the site. The use of dredge spoil for backfill is discussed subsequently while other disposal methods are addressed in Chapter 9.

**Backfill**

The second step in solid fill dock, pier or wharf construction following the completion of the wall is the placement of the backfill. The type of fill material and method of placement used are important parameters determining wall stability and long-term performance. These topics are addressed in the following discussions.

The strength and engineering behavior of cohesive soils or clays is highly variable and depends on mineralogy, structure, stress history and water content. Low permeability and poor drainage is characteristic of clay fills causing them to consolidate for long periods of time, and to develop hydrostatic imbalances under the action of heavy rain or rapid tides. A successful clay backfill requires that the same type of soil be used throughout and that special attention be given to the water content and compactive effort during placement so that a uniform solid mass is achieved. The Committee for Waterfront Structures (1966) suggests, however, that compaction of clay backfills causes considerable additional earth pressure that may damage an otherwise sound wall. In light of these problems, cohesive backfills should be used only when cohesionless materials are not available within a reasonable radius of transportation.

Saczynski and Kulhawy (1982) suggest that a coarse-grained, free-draining backfill should be used whenever possible. Because the engineering behavior of these cohesionless materials (sands and gravels) is
predictable, the resulting wall designs are quite reliable. Bray (1979) recommends that specifications for sand fills should include the following: (1) required grain size distribution - to ensure that the soil can be compacted to a suitable density, (2) minimum acceptable particle size and the percentage of this size which is allowable - to control settlement and to be used in filter design, and (3) acceptable organic content - since the presence of organics affects settlement and soil strength. Compaction specifications should also be written to address in-situ densities and compaction techniques.

Relative density is a qualitative parameter used to measure the degree of compaction of granular soils. In its most convenient form, the relative density, $D_r$, is defined as follows:

$$D_r = \frac{\gamma_m (\gamma - \gamma_o)}{\gamma (\gamma_m - \gamma_o)}$$

where $\gamma_o$ = minimum density of soil in laboratory

$\gamma$ = field density of soil

$\gamma_m$ = maximum density of soil when compacted in laboratory by vibration

The relative density of a soil is usually expressed as a percentage and may vary from 0 percent to 100 percent. A relative density of 0 percent represents the loosest state theoretically possible while a soil at 100 percent relative density is in its most dense condition. While shear strength in a sandy soil also depends on particle size and shape, greater densities result in increased strength and bearing capacity.

Fills derived from sand containing less than 15 percent fines can be placed naturally to a medium relative density (44 to 55 percent) capable of supporting foundation pressures of 500 - 3000 psf (24-144 kN/m²) (Bray, 1979). The Committee for Waterfront Structures (1966)
notes that a relative density of around 85 percent may be obtained by
placing the fill in well-compacted layers.

Placement of backfill material is accomplished by either mechanical
or hydraulic means. Mechanical methods include dumping by truck, or
dropping from a clamshell, dipper, or drag bucket. The fill is first
placed in piles and then distributed into even layers with a bulldozer.
Hydraulic fills are created by pumping a soil/water mixture into a
contaminant area through a pipeline. Hydraulic fills are very convenient
when granular materials must be dredged nearby, but they create some
special problems. Ponding of the water in the reclamation area should
not be allowed since fines may be segregated into mud pockets. Unfortu-
nately, the initial fill behind a bulkhead must often be placed under-
water. Bray (1979) suggests that this initial layer be formed to a
level 2 to 3.5 ft. (0.5 to 1 m) above the maximum level of the water
in front of the wall. Subsequent layers 3.5 ft. (1 m) thick can be
added as compaction and consolidation is achieved. Szczyński and Kulhawy
(1982) note that the fill should be placed in even lifts along the length
of the wall to avoid local overstressing.

Compaction of sandy fills is commonly achieved through the use of
vibroflotation or a vibratory roller. Vibratory compaction is effective
only in well-drained soils and becomes less efficient with increasing
silt or clay content. Vibroflotation can be conducted above or below
the water table and is accomplished by inserting a vibrating probe into
the fill and feeding the annular space around the probe with additional
fill material as it is withdrawn (Bray, 1979). A grid spacing of less
than 15 ft. (5 m) is normally required to obtain full coverage or 80
percent relative density using a 100 horsepower probe. Vibratory rollers are used above the water table where the density that may be achieved depends on the soils moisture content. Bray (1979) suggests that a vibratory frequency of 1500 to 1700 Hz is most effective in compacting sands.

In areas of active seismicity or intense industrial or construction activity, sand backfills are subject to liquefaction. The vibration of an earthquake, blasting, or heavy equipment acts in much the same manner as vibratory compaction but on a much larger scale. The effect is known as liquefaction and is manifested in a sudden, temporary loss of shear strength. Liquefaction potential depends on soil grain size and density and is greatest for silts and fine sands of uniform gradation. The risk of liquefaction is minimized by specifying a well-graded granular backfill to be compacted as dense as possible.

5.4 SUMMARY

Solid fill structures are rarely used for berthing because of their inefficient use of space and high cost compared to fixed or floating docks or piers. They are more suitable for stabilization and protection of the harbor perimeter and for the construction of marginal wharves.

Selection of the type of solid fill wall depends on site specific conditions and the scope of the project. Anchored bulkheads are the most common wall type for recreational marinas because of their low cost and ease of construction.

Solid fill walls must be designed against bearing capacity failures, excessive settlement, undermining from scour, or combinations of these. Design itself follows the procedures of soil mechanics and foundation
engineering and should be performed by a competent geotechnical engineer. Attention must be given to dredging and backfill operations to control the forces acting on a wall and to avoid damage during construction.
CHAPTER 6
FIXED DOCKS, PIERS AND WHARVES

For the purposes of this report, docks, piers and wharves that are pile supported will be considered fixed structures. Fixed docks and piers are generally less expensive to construct than equivalent floating berths. In spite of their economic advantage, however, there has been a trend away from the fixed structure toward the use of floating slips for all small craft marinas (Dunham, 1969). Fixed berths are usually limited to locations where water surface fluctuations do not exceed 4 ft (1.2 m) and the basin depth is less than about 20 ft (6.0 m). Where greater water surface changes occur, boats are difficult to board at extreme low water, and traveling irons must be provided for safe mooring at all levels. Deep water installations (in excess of 20 ft or 6.0 m) are not economically feasible since piles represent a major portion of the cost of a fixed pier structure, and their cost is directly proportional to the length required.

Fixed pier construction is particularly favorable for covered berthing since long piles can be used to support both the deck and roof. While covered slips provide excellent protection from the elements, they have several disadvantages (Dodds, 1971). The cost of constructing and maintaining the roof must be borne by increased slip rental fees. Since wood is the material commonly used for such structures, fire hazards are dramatically increased while the enclosure makes it difficult to fight the fires. Changing the slip sizes of an open slip system is difficult, but the problem is compounded by a covering which interferes with the equipment needed to pull and drive piles. The economics and
feasibility of the covered berth depend on the analysis of the individual marina site, the intended user, and any future plans for expansion. For small craft up to 30 ft (9 m) in length, dry stack storage as shown in Figure 6.1 should be considered an attractive alternative (New York Sea Grant, 1978). Where covered berths are to be built, standard pole-shed construction is recommended as presented by Patterson (1969). Covered berthing will not be addressed further in this report except to note that failure to design properly for wind uplift forces has been the major cause of damage in these structures (Dodds, 1971).

Fixed docks in general are subject to damage by ice in northern areas. Lateral forces because of expansion/contraction of the ice sheet, as well as vertical forces resulting from water level changes, literally tear a structure apart (Wortley, 1981). Bearing piles are abraded at the waterline and "jacked" out of the bottom. Bracing members are knocked off by ice floe impact, while utility lines are bent or broken by protruding ice rubble. Design of harbor structures for such conditions may be one of two types: with or without ice suppression. Design with ice suppression relies on the operation of a bubbler or propeller system to reduce the ice sheet thickness and the resulting forces. Compressed air bubbles (Figures 6.2 and 6.3) circulate warm water from the harbor bottom by entraining water into the rising bubble plume. Propeller systems operate in the same manner but are more suitable to warmer water of 33 to 36°F (0.5° to 2.0°C). Harbor structures designed without ice suppression must resist the full forces of the ice mass which may approach its crushing strength of about 400 psi (2.8 MN/m²). Design of dock, pier, and wharf structures for northern small craft harbors is addressed
Figure 6.2 Compressed Air-Ice Suppression System
(Wortley, 1979, p. 5)

Figure 6.3 Air Bubbler Layout (Dunham and Finn, 1974, p. 216)

Several manufacturers have developed modular fixed dock and pier systems which offer the marina designer several advantages over a system specifically designed for one location. Design costs are absorbed by the manufacturer and spread out over several installations, thus reducing the total cost per unit. From experience gained by the building of similar structures, many construction problems can be eliminated and necessary design modifications made. Prefabricated systems also lend themselves to rapid installation and ease of expansion. On the other hand, Dunham (1969) notes that most steel and aluminum prefabricated docks have had problems with corrosion in salt water, while Dunham and Finn (1974) state that they are more suitable for individual docks than for large installations as required in marinas. Since these limitations must be considered minor for such a rapidly evolving industry, the marina designer should consider modular dock and pier systems a viable solution to marina berthing design.

The following discussion addresses the design considerations of the components of a fixed dock, pier, or wharf. These topics include structural geometry, pile types by material of construction, design of pile foundations, and decking and framing details. Mooring provisions and fenders as they relate to fixed docks and piers are also addressed briefly.

6.1 STRUCTURAL GEOMETRY

The structural geometry of fixed docks, piers and wharves is relatively simple, as illustrated in Figures 6.4 and 6.5. Piles are arranged
Figure 6.4 Fixed Pier Construction (Dunham, 1969, p. 97)

Figure 6.5 High-level Fixed Wharf Construction (AAPA, 1964, p. 38)
in rows or "bents" spaced 10 to 14 ft. (3 to 4 m) apart. A pile cap connecting all the piles in a bent runs from one side of the pier to the other and supports the stringers and deck. Lateral cross-bracing is used to resist lateral loads and provides stability and a sense of rigidity. Where lateral loads are large, inclined or batter piles are used instead. The fixed wharf closely resembles a solid fill relieving platform as described in the previous chapter. The fundamental difference lies in the fact that the fill of a relieving platform extends over the deck to provide additional weight for stability. Fixed wharves typically have high level decks in which the deck superstructure system is supported directly on piles arranged in transverse rows. A lighter deck is therefore acceptable and fewer piles are required since the vertical loads are greatly reduced. Open type fixed wharves are less expensive and easier to construct than are relieving platform wharves.

Fixed docks and piers are smaller (See Chapter 2) and therefore less substantial than fixed wharves. Their structural geometry is the same, however, with pile bents, caps, and stringers supporting a continuous deck. Since fixed docks and piers are built for the purpose of berthing boats, they are generally constructed with a deck elevation 1 ft. (0.3 m) above extreme high water. Sloping gangways connect the low level decks of the berthing system with the perimeter wharf or bulkhead wall.

6.2 PILE FOUNDATIONS

The basic material types for piles used in waterfront construction are timber, steel, and concrete (Figure 6.6). Composite piles formed by combinations of these materials are also used for special conditions. There are two general classes of piles including bearing piles and sheet
Figure 6.6 Typical Pile Types Used in Waterfront Construction
(Tobiasson, 1979, p. 2)
piles. Bearing piles are used to support structural loads (both lateral and vertical) while sheet piles form continuous walls to resist horizontal soil and water pressures. Bearing piles may be further classified as "end-bearing" or "frictional" piles. End-bearing piles rely on point bearing on a firm stratum to support the pile load. Frictional piles transfer the applied load into the surrounding soil along the pile embedded length (Figure 6.7). The following section focuses on the bearing pile types by material of construction, the selection of the proper pile type for a given location, the design of pile foundations, and the installation of piles. Pile foundations are also addressed by Cheung and Kulhawy (1981).

**Timber Piles**

Timber piles are probably the most commonly used pile type on the waterfront because of their availability, constructability, and low cost. According to Tobiasson (1979), timber piles often cost less per foot when in place than other pile types. For many applications, however, their use is limited by their load carrying capacity, length availability, and susceptibility to deterioration. In addition to soil conditions at the point of installation, the capacity of a timber pile is determined by its axial strength which is a function of the material defects inherent in a wood member. Wood defects and strength properties are discussed in Chapter 4. Timber piles are best used as friction piles in soft soils because of their relatively small cross-sectional area and tapered shape. Where they are intended to be used in end-bearing, hard driving through highly resistant soils may cause crushing and damage that is mistaken for additional penetration. The result of over-driving is a structural
Figure 6.7 Bearing and Batter Piles (Tobiasson, 1979, p. 4)
failure of the pile before it is even loaded. For the above reasons, wood piles must be considered low capacity foundations when compared with steel or concrete piles. This is of little consequence in marina design where the pile spacing is determined by superstructure framing details and only a portion of the piles load carrying capacity is used.

The standard lengths of timber piles are limited by the height of suitable wood species. Southern pine and Douglas fir are the principal species used for treated piling in saline environments. Red pine, Norway pine, Oak, Red cedar and other species are also acceptable if properly treated with preservatives. Southern pine is readily obtained in lengths to 60 ft. (18 m) while Douglas fir is available on the West Coast in lengths up to 100 ft. (30 m). With the low applied loads and shallow water depths typical of the recreational marina, timber piles are rarely found to be inadequate because of their available length.

Unlike load capacity or length availability, rapid deterioration in the marina environment is a serious problem. Timber piling comes under the attack of insects, marine borers, organic decay, and abrasion from boats and scour currents. Protection in the form of a pressure impregnated preservative is required in most cases. Material deterioration and preservative treatment is discussed by Hubbell and Kulhawy (1979a). Encasement in a concrete jacket or various patented methods of plastic wrap have also been used to protect timber piles in saltwater locations where marine organism attack is especially severe (Tobiasson, 1979).

Additional protection must be provided to the top of timber piles where they extend above dock level. First, the top of the pile where it is cut off must be sealed to keep moisture from penetrating the end
grain and starting decay (Chaney, 1961). Chaney suggests that the pile butt be swabbed with creosote, followed by a thick coat of tar and a concrete or cast iron cap (Figure 6.8). Molded plastic or fiberglass pile caps (Figure 6.9) are suggested by Dunham and Finn (1974). The conical shape of these caps sheds the rain and helps keep birds off the piles. Secondly, when creosoted piles are used for support members, the portion above the deck line will always be dark and oily as the creosote seeps out of the wood. To avoid users clothing from coming in contact with this creosote, Dunham and Finn (1974) recommend a wood batten system as illustrated in Figure 6.10. Another alternative is to splice on a salt treated pile butt that is stained to match the creosote as in Figure 6.11.

Several agencies have prepared specifications for wood poles to be used as pile foundations. Among these, the American National Standard (ANSI 05.1-1979), American Society for Testing and Materials (Standards D25 and D2899), and American Wood Preservers Institute (Technical Guidelines Pl through P5) are recommended references for information on wood species, dimensions, general quality, strength, decay resistance, preservative treatment, inspection, splicing, storage, and handling of timber piles.

Steel Piles

Steel piles in the form of H-sections or pipes are widely used for pile foundations. Steel piles are especially applicable to conditions that require hard driving, great lengths, or high single pile capacities. Since H-piles displace relatively small volumes of soil, they are more easily driven than other pile types and are commonly used to reach strong
Nails driven into the pile to hold on the concrete cap

Concrete

Creosoted timber bearing pile

Figure 6.8 Concrete and Tar Cap

Molded plastic or fiberglass conical cap

Bearing pile

Figure 6.9 Molded Synthetic Cap
Figure 6.10 Wood Batten Pile Protection
Figure 6.11 Timber Pile Splice Detail
(Van Blancom, 1970, p. 3)
bearing strata at great depths. Pipe piles capped on the end by flat plates or conical points are sometimes harder to drive than H-piles because of their larger end area. Unlike H-piles which are most efficient in end bearing, pipe piles are more suitable as friction piles. A significant advantage of pipe piles is that they can be visually inspected after driving to identify any damaged casings and allow repair or replacement. Pipe piles are usually filled with concrete to increase their compressive strength and control internal corrosion.

Length is not a critical factor in the use of steel piles since they are easily joined on the job site by welding. At the other extreme, steel piles that are too long can be quickly trimmed to the proper height by oxyacetylene cutting, even under water. Large steel piles are considered very high capacity foundations and are rarely used in marina construction. Smaller sections with correspondingly lower capacities are more suitable but are also more easily damaged by handling and boat impact. As in the case of timber piles, deterioration proves to be the major factor determining the lifetime performance of steel piles.

Deterioration of steel usually takes the form of oxidation corrosion, more commonly called rust. In the tidal range, bare steel exposed to saltwater may corrode up to 0.020 in. (0.5 mm) per year (AAPA, 1964). In such an environment, steel piles must be considered temporary unless some effective form of protection can be devised. Epoxy coatings, concrete encasement, and sacrificial cathodes are some of the techniques that are successful depending on site specific conditions. For example, cathodic protection is not reliable in the tidal zone (Peck, Hanson,
and Thornburn, 1974), and coatings may be worn away by sand blast abrasion near the harbor bottom (AAPA, 1964). Hubbell and Kulhawy (1979a) discuss the corrosion process of steel in the marine environment with various coatings or cathodic protection.

Specifications for steel piles are much less detailed than those for wood because, as a man-made material, steel is much more uniform and predictable. Material properties and standard dimensions for steel H-piles and pipe piles are specified by the American Society for Testing and Materials (1980) in standards A690-77 and A252-77a respectively. Manufacturers of steel piles (Bethlehem Steel Corporation or the U.S. Steel Corporation, for example) also publish product literature and are available for technical consultation.

Concrete Piles

Concrete piles may be divided into two main categories, cast-in-place and precast. Cast-in-place piles may be further classified as cased or uncased. The concrete of a cased pile is poured inside a form that remains in the ground. The form is usually a steel shell or thin pipe that has negligible strength with respect to the structural capacity of the pile. In some ground conditions, the shell may be driven alone, but often it must be supported by an internal driving mandrel to prevent collapse. The mandrel is withdrawn and reused on subsequent piles but it still represents a source of expense and construction difficulties. Uncased cast-in-place piles are less expensive since elimination of the casing lowers the material costs. A mandrel is again driven and withdrawn before or during the placement of the concrete. These piles should be considered only where it is certain that the hole will not be partially
or completely closed by soil stresses after the removal of the shell, since imperfections or discontinuities will result that severely weaken the pile (AAPA, 1964). Waterfront construction with uncased piles presents a problem above the mudline where the concrete is not retained by the surrounding soil. Figure 6.12 illustrates the various types of cast-in-place piles used in North America.

Precast concrete piles have found more extensive use in marine installations (AAPA, 1964). Square, round, or octagonal shapes are common with tapered or constant cross-sections. Conventionally reinforced precast piles generally have pointed driving ends and hollow cores for low weight. Examples of precast conventional piles are illustrated in Figure 6.13. Prestressed, precast concrete piles have also come into general use (AAPA, 1964). Prestressing reduces the incidence of tensile cracking during handling and driving since the piles are stronger in bending when subject to lateral loads and buckling. Theoretically, prestressed piles should therefore be more durable, but Buslov (1979) noted in a study of the durability of wharves that after 15 years in service, "no major differences in performance were found between regular and prestressed piles."

The load capacity of concrete piles is highly variable depending on cross-sectional area, concrete quality, thickness of the steel shell, and the amount of reinforcing steel. Very large concrete piles are of medium to high capacity, being somewhat less than large steel piles but considerably greater than the average timber pile. As in the case of steel piles, only smaller sections have found extensive use in marina construction since high single pile capacity is rarely required.
Figure 6.12  Examples of Cast-in-Place Concrete Piles
(Pack, Hanson and Thornburn, 1974, p. 205)
Figure 6.13 Examples of Precast Concrete Piles
(Peck, Hanson and Thorburn, 1974, p. 206)