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 Szczyski 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.

### 3) INFLUENCE OF WATER LEVEL

- **Reduced Air Pressure**
  - Water level not stable
  - Short period waves

- **Increased Air Pressure**
  - Stable water level
  - Long period waves

### 4) INFLUENCE OF WAVE PERIOD

- **Compressed Volume**
  - Air relief
  - Very compressed volume

### 5) INFLUENCE OF WAVE HEIGHT

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**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. **Predredging.** 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. Unfortu-
nately, 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, Saczynski 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. Unfortunately, the initial fill behind a bulkhead must often be placed underwater. 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. Saczynski and Kulhawy (1982) note that the fill should be placed in even lifts along the length of the wall to avoid local over stressing.

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.