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COASTAL AND PORT ENGINEERING

Coastal and port engineering encompasses planning, design, and construction of projects to satisfy society's needs and concerns in the coastal environment, such as harbor and marina development, shore protection, beach nourishment, and other constructed systems in the coastal wave and tide environment.

Over time, the scope of this field of engineering has broadened from only navigation improvement and property protection to include recreational beaches and environmental considerations. It takes into account the environmental conditions unique to the coastal area, including wind, waves, tides, and sand movement. Thus, coastal engineering makes extensive use of the sciences of oceanography and coastal geomorphology as well as of geotechnical, environmental, structural, and hydraulic engineering principles.

23.1 Risk Level in Coastal Projects

Because of the nature of littoral drift, or longshore sand transport along the coasts, erosion caused by coastal engineering projects along adjacent shorelines, sometimes several miles away, has been a recurring problem. Tools for prediction and evaluation of such shoreline dynamics are continually improving but are still limited, in part because of nature's unpredictability. Hence, post-construction monitoring of the response of nearby beaches is often a required component of coastal engineering projects.

The design level of risk in many coastal engineering projects may be higher than in other civil engineering disciplines because the price of more effective design is often not warranted.

The design environment is very challenging. It varies with time, since design conditions are often affected by storms that contain much more energy and induce very different loadings from those normally experienced. Also, because the physical processes are so complex, often too complex for theoretical description, the practice of coastal engineering is still much of an art. Consequently, practitioners should have a broad base of practical experience and should exercise sound judgment.

The practice of coastal engineering has changed rapidly in the last several decades owing to increases in natural pressures, such as that created by sea-level rise, and societal pressures, such as those from growing populations along the coast with greater environmental awareness. The changes are recorded in the proceedings of specialty conferences, such as those of the American Society of Civil Engineering (ASCE), including Coastal Engineering Practice; Dredging, Ports, Coastal Sediments, Coastal Zone, International Coastal Engineering Conference, and the Florida Shore and

Beach Preservation Association's Beach preservation Technology Conference series.

Coastal Hydraulics and Sediments

Waves often apply the primary hydraulic forces of interest in coastal engineering. Tides and other water-level fluctuations control the location of wave attack on the shoreline. Waves and tides generate currents in the coastal zone. Breaking waves provide the forces that drive sand transport along the coast and can cause beach changes, including erosion due to coastal engineering projects.

23.2 Characteristics of Waves

Water waves are caused by a disturbance of the water surface. The original disturbance may be caused by wind, boats or ships, earthquakes, or the gravitational attraction of the moon and sun. Most of the waves are initially formed by wind. Waves formed by moving ships or boats are **wakes**. Waves formed by earthquake disturbances are **tsunamis**. Waves formed by the gravitational attraction of the moon and sun are **tides**.

After waves are formed, they can propagate across the surface of the sea for thousands of miles. The properties of propagating waves have been the subject of various wave theories for over a century. The most useful wave theory for engineers is the linear, or small-amplitude, theory.

23.2.1 Linear Wave Theory

Essentially, linear wave theory treats only a train of waves of the same length and period in a constant depth of water. As in optics, this is called a monochromatic wave train. Linear wave theory relates the length, period, and depth of waves as indicated by Eq. (23.1).

$$L = \frac{gT^2}{2\pi} \tanh \frac{2\pi d}{L} \tag{23.1}$$

where L = wavelength, ft, the horizontal distance between crests

d = vertical distance, ft, between mean or still water level and the bottom

g = acceleration due to gravity, 32.2 ft/s

T = wave periods, the time required for propagation of a wave crest over the wavelength (Fig. 23.1)

Wave height H, the fourth value needed to completely define a monochromatic wave train, is an independent value in linear wave theory, but not for higher-order wave theories (Art. 23.2.2).

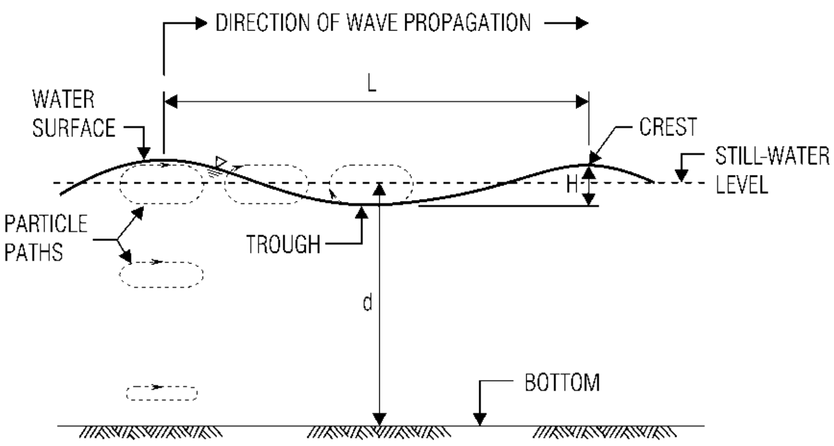

Fig. 23.1 Wave in shallow water. Water particles follow an elliptical path. L indicates length of wave, crest to crest; H wave height, d depth from still-water level to the bottom. The wave period T is the time for a wave to move the distance L.

Equation (23.1), implicit in terms of L, requires an iterative solution except for deep or shallow water. When the relative depth d/L is greater than ½, the wave is in deep water and Eq. (23.1) becomes

$$L = \frac{gT^2}{2\pi} \tag{23.2}$$

For shallow water, $d/L < 1/25$, eq. (23.1) reduces to

$$L = T\sqrt{gd} \tag{23.3}$$

Individual water particles follow a closed orbit. They return to the same location with each passing wave. The orbits are circular in deep water and elliptical in shallow water. Linear wave theory equations for the water-particle trajectories, the fluctuating water-particle velocities and accelerations, and pressures under wave trains are given in R. G. Dean and R. A. Dalrymple, "Water Wave Mechanics for Scientists and Engineers," Prentice-Hall, Englewood Cliffs, N. J. (www.prenhall.com); R. M. Sorenson, "Basic Wave Mechanics: For Coastal and Ocean Engineers," John Wiley & Sons, Inc., New York (www.wiley.com).)

23.2.2 Higher-Order Wave Theories

The linear wave theory provides adequate approximations of the kinematics and dynamics of wave motion for many engineering applications. Some areas of concern to civil engineers where the linear theory is not adequate, however, are very large waves and shallow water. Higher-order wave theories, such as Stokes' second order and cnoidal wave theories, address these important situations. Numerical wave theories, however, have the broadest range of applicability. Useful tables from stream-function wave theory, a higher-order, numerical theory, are given in R. G. Dean, "Evaluation and Development of Water Wave Theories For Engineering Applications," Special Report No. 1, U.S. Army Coastal Engineering Research Center, Ft. Belvoir, Va.

Determination of the water surface elevations for large waves or waves in shallow water requires use of a higher-order wave theory. A typical waveform is shown in Fig. 23.2. The crest of the wave is more peaked and the trough of the wave is flatter than for the sinusoidal water surface profile in linear wave theory. For a horizontal bottom, the height of the wave crest above the still-water level is a maximum of about $0.8d$.

("Shore Protection Manual," 4th ed., U.S. Army Coastal Engineering Research Center, Government Printing Office, Washington, D.C. (www.gpo.gov); "Coastal Engineering Manual," (www.usace.mil/inet/usace-docs/eng-manuals/em-htm).)

23.2.3 Wave Transformations

As waves move toward the coast into varying water depths, the wave period remains constant (until breaking). The wavelength and height, however, change because of shoaling, refraction, diffraction, reflection, and wave breaking.

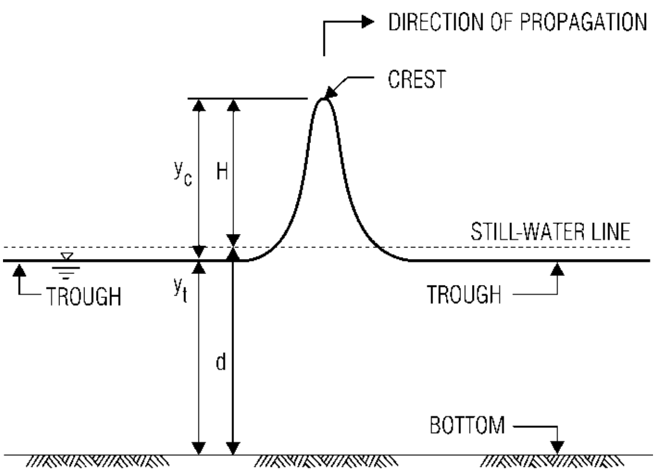

Fig. 23.2 Water surface for a large wave in shallow water.

Shoaling ▪ As a wave moves into shallower water the wavelength decreases, as indicated by Eq. (23.1), and the wave height increases. The increase in wave height is given by the shoaling coefficient K_s.

$$K_s = \frac{H}{H_o'} \tag{23.4}$$

where H = wave height in a specific depth of water

H_o' = deep-water unrefracted wave height

K_s varies as a function of relative depth d/L as shown in Table 23.1. For an incident wave train of period T, Table 23.1 can be used to estimate the wave height and wavelength in any depth with Eq. (23.2) for L_o.

Refraction ▪ This is a term, borrowed from optics, for the bending of waves as they slow down. As waves approach a beach at an angle, a portion of the wave is in shallower water and moving more slowly than the rest. Viewed from above, the wave crest appears to bend.

Refraction changes the height of waves as well as the direction of propagation. Refraction can cause wave energy to be focused on headlands and defocused from embayments.

There are two general types of refraction models. Wave-ray models trace the path of wave rays, lines perpendicular to the wave crests. The other type of computer refraction model computes solutions to differential equations for the wave-height field. The physics simulated varies slightly from model to model.

Table 23.1 Shoaling Coefficient and Wavelength Changes as Waves Move into Shallower Water

d/L_o	d/L	K_s
0.005	0.028	1.70
0.010	0.040	1.43
0.020	0.058	1.23
0.030	0.071	1.13
0.040	0.083	1.06
0.050	0.094	1.02
0.10	0.14	
0.20	0.22	
0.30	0.31	
0.50	0.50	1.0

Diffraction ▪ Another term borrowed from optics, this is the spread of energy along a wave crest. An engineering example of wave diffraction is the spreading of energy around the tip of a breakwater into the lee of the breakwater. The wave crest wraps around the tip of a breakwater and appears to be propagating away from that point. Diffraction also occurs in open water where refraction occurs. It can reduce the focusing and bending due to refraction.

Reflection ▪ Waves are reflected from obstructions in their path. Reflection of wave energy is greatest at vertical walls, 90% to 100%, and least for beaches and rubble structures. Undesirable wave-energy conditions in vertical-walled marinas can often be reduced by placing rubble at the water line.

Breaking ▪ This happens constantly along a beach, but the mechanics are not well modeled by theory. Thus, much of our knowledge of breaking is empirical. In shallow water, waves break when they reach a limiting depth for the individual wave. This depth-limited breaking is very useful in coastal structure design and surf-zone dynamics models.

For an individual wave, the limiting depth is about equal to the water depth and lies in the range given by Eq. (23.5.).

$$0.8 < \left(\frac{H}{d}\right)_{\max} < 1.2 \tag{23.5}$$

where $(H/d)_{\max}$ = maximum ratio of wave height to depth below mean water level for a breaking wave. The variation in $(H/d)_b$ (the subscript b means breaking) is due to beach slope and wave steepness H/L.

Equation (23.5) is often useful in selecting the design wave height for coastal structures in shallow water. Given an estimate of the design water depth at the structure location, the maximum wave height $H_{\max}$ that can exist in that depth of water is about equal to the depth. Any larger waves would have already broken farther offshore and been reduced to $H_{\max}$.

23.2.4 Irregular Waves

The smooth water surfaces of monochromatic wave theories are not realistic representations of the real surf zone. Particularly under an active wind, the water surface will be much more irregular. Two different sets of tools have been developed by oceanographers to describe realistic sea surfaces. One is a statistical representation and one is a spectral representation.

Statistics of Wave Height ▪ The individual waves in a typical sea differ in height. The heights follow a theoretical Rayleigh distribution in deep water. In shallow water, the larger individual waves break sooner, and thus the upper tail of the distribution is lost.

A commonly used, single wave-height parameter is the significant wave height $H_{1/3}$. This is the average of the highest one-third of the waves. Other wave heights used in design can be related to $H_{1/3}$ via the Rayleigh distribution as indicated in Table 23.2.

23.2.5 Wave Spectra

Spectral techniques are available that describe the amount of energy at the different frequencies or wave periods in an irregular sea. They provide more information about the irregular wave train and are used in some of the more advanced coastal-structure design methods. A wave-height parameter that is related to the total energy in a sea is H_{m_o}. (H_{m_o} is often called significant wave height also.)

Significant wave height H_s is a term that has a long history of use in coastal engineering and oceanography. As indicated above and in Art. 23.2.4, two fundamentally different definitions for significant wave height are used in coastal engineering. One is statistically based and the other is energy- or spectral-based. Since they are different, the notations, $H_{1/3}$ and H_{m_o} are recommended to avoid confusion in use of H_s:

$$H_{1/3} = \text{statistical significant wave height}$$
$$H_{m_o} = \text{spectral significant wave height}$$

In deep water, H_{m_o} is approximately equal to $H_{1/3}$. In shallow water, and in particular in the surf zone, the two parameters diverge. (There is little that is truly significant about either parameter. Few of the waves in an actual wave train will have the significant height. It is basically a statistical artifact.)

Transformations of actual wave seas such as shoaling, refraction, diffraction, and breaking are not completely understood and not well modeled. Although the monochromatic wave transformations are well modeled, as described in the preceding, in actuality the individual waves and wave trains interact with each other and change the wave field. (These wave-wave interactions are the subject of significant research efforts.) Thus, the more realistic conditions, that is, irregular seas, are the least understood. However, models that account for the transformation of wave spectra across arbitrary bottom contours are available.

23.2.6 Wave Generation by Wind

Waves under the influence of the winds that generated them are called **sea**. Waves that have propagated beyond the initial winds that generated them are called **swell**.

Fetch is the distance that a wind blows across the water. For enclosed bays, this is the distance across the water body in the direction of the wind. **Duration** is the time that a wind at a specific speed blows across the water. The waves at any spot may be fetch-limited or duration-limited. When a wind

Table 23.2 Wave Heights Used in Design

Symbol	Description	Multiple of $H_{1/3}$
$H_{1/3}$	Average height of highest one-third of waves	1.0
H_{av}	Average wave height	0.6
H_{10}	Average height of highest 10% of waves	1.3
$H_{1\%}$	Wave height exceeded 1% of the time	1.6
H_{sin}	Height of simple sine waves with same energy as the actual irregular height wave train	0.8

starts to blow, wave heights are limited by the short time that the wind has blown; in other words, they are duration-limited. Seas not duration-limited are *fully arisen*. If the waves are limited by the fetch, they are fetch-limited.

For enclosed bay and lake locations, simple parametric models can provide useful wave information. Table 23.3 gives wave height and wave period estimates for deep water for different fetch distances and different wind speeds. The values are based on the assumption that the wind blows for a sufficient time to generate fully arisen conditions. In shallow water, the wave heights will be less.

On the open ocean, waves are almost never fetch-limited. They are free to continue to move after the wind ceases or changes. Swell wave energy can propagate across entire oceans. The waves striking the beach at any moment in time may include swell from several different locations plus a local wind sea. Thus, for an open-ocean situation, numerical models that grid the entire ocean are required to keep track of wave-energy propagation and local generation.

Wave-generation models can forecast waves for marine construction operations. They can also hindcast, that is, estimate waves based on measured or estimated winds at times in the past, for wave climatology studies, probabilistic design, or historic performance analysis. The U.S. Army Corps of Engineers "Wave Information Study (WIS)" has hindcast 40 years of data, 1956–1995, to generate probabilistic wave statistics for hundreds of locations along the coasts of the United States. The wave statistics are available in tabular form, and the actual time sequence of wave conditions is available in digital form.

(J. B. Herbich, "Handbook of Coastal and Ocean Engineering," Gulf Publishing Company, Houston, Tex (www.gulfpub.com).)

23.2.7 Ship and Boat Wakes

Ship wakes are sometimes the largest waves that occur at a location and thus become the design wave. Vessel wakes from large ships can be up to 6 ft high and have wave periods less than 3 s. Ship wakes can be estimated with methods presented in J. R. Weggel and R. M. Sorensen, "Ship Wave Prediction for Port and Channel Design," Proceedings, Port Conference, 1986, ASCE. Approaches for estimating the wakes due to recreational boats are presented in ASCE Manual 50, "Planning and Design Guidelines for Small-Craft Harbors," and R. R. Bottin et al., "Maryland Guide Book for Marina Owners and Operators on Alternatives Available for the Protection of Small Craft against Vessel Generated Waves," U.S. Army Corps of Engineers Coastal Engineering Research Center, Washington, D.C.

Table 23.3 Spectral Significant Heights and Periods for Wind-Generated Deep-Water Waves*

Wind speed, knots	Fetch length, statute miles				
	0.5	1	2	10	50
20					
H_{m_o}, ft	0.6	0.8	1.1	2.2	4.1
T_p, s	1.3	1.6	2.0	3.2	4.7
40					
H_{m_o}, ft	1.3	1.8	2.5	5.4	11
T_p, s	1.7	2.2	2.7	4.5	7
60					
H_{m_o}, ft	2.2	3.1	4.2	9.1	18
T_p, s	2.1	2.6	3.2	5.4	8

* Based on method presented in S. L. Douglass et al., "Wave Forecasting for Construction in Mobile Bay," Proceedings, Coastal Engineering Practice, 1992, pp. 713–727, American Society of Civil Engineers. H_{m_o} = spectral significant wave height and T_p = wave period.

23.3 Design Coastal Water Levels

The design water level depends on the type of project. For design of some protective coastal structures, for example, a water level based on a recurrence interval such as a 10-year or 100-year return period often is selected. The Federal Emergency Management Agency (FEMA) "Flood Insurance Rate Maps (FIRM)" are based on such a concept. They provide a first estimate of high-water levels along the U.S. coastlines. Since the design of some coastal structures can be extremely sensitive to the design water level, more in-depth analysis may be justified. For engineering projects concerned with normal water levels, for example, where dock elevations and beachfill elevations are determined by the water level, an estimate of the normal water level and the normal range around that mean is needed. All coastal engineering projects should be designed to take into account the full range of potential water levels.

The water level at any time in a specific location is influenced by the tides, mean sea-level elevation, storm surge, including wind influence, and other local influences, such as fresh-water inflow in estuaries.

Tides ▪ The tide is the periodic rise and fall of ocean waters produced by the attraction of the moon and sun. Generally, the average interval between successive high tides is 12 h 25 min, half the time between successive passages of the moon across a given meridian. The moon exerts a greater influence on the tides than the sun. Tides, however, are often affected by meteorological conditions, including propagation of storm tides from the sea into coastal waters.

The highest tides, which occur at intervals of half a lunar month, are called **spring tides**. They occur at or near the time when the moon is new or full, i.e., when the sun, moon, and earth fall in line, and the tide-generating forces of the moon and sun are additive. When the lines connecting the earth with the sun and the moon form a right angle, i.e., when the moon is in its quarters, then the actions of the moon and sun are subtractive, and the lowest tides of the month, the **neap tides**, occur.

Tidal waves are retarded by frictional forces as the earth revolves daily around its axis, and the tide tends to follow the direction of the moon. Thus, the highest tide for each location is not coincident with conjunction and opposition but occurs at some constant time after new and full moon. This interval, known as the *age* of the tide, may amount to as much as $2\frac{1}{2}$ days.

Large differences in tidal range occur at different locations along the ocean coast. They arise because of secondary tidal waves set up by the primary tidal wave or mass of water moving around the earth. These movements are also influenced by the depth of shoaling water and configuration of the coast. The highest tides in the world occur in the Bay of Fundy, where a rise of 100 ft has been recorded. Inland and landlocked seas, such as the Mediterranean and the Baltic, have less than 1 ft of tide, and the Great Lakes are not noticeably influenced.

Tides that occur twice each lunar day are called **semidiurnal tides**. Since the lunar day, or time it takes the moon to make a complete revolution around the earth, is about 50 min longer than the solar day, the corresponding high tide on successive days is about 50 min later. In some places, such as Pensacola, Florida, only one high tide a day occurs. These tides are called **diurnal tides**. If one of the two daily high tides is incomplete, i.e., if it does not reach the height of the previous tide, as at San Francisco, then the tides are referred to as **mixed diurnal tides**. Table 23.4 gives the spring and mean tidal ranges for some major ports.

There are other exceptional tidal phenomena. For instance, at Southampton, England, there are four daily high waters, occurring in pairs, separated by a short interval. At Portsmouth, there are two sets of three tidal peaks per day. **Tidal bores**, a regular occurrence at certain locations are high-crested waves caused by the rush of flood tide up a river, as in the Amazon, or by the meeting of tides, as in the Bay of Fundy.

The rise of the tide is referred to some established datum of the charts, which varies in different parts of the world. In the United States, it is mean lower low water (MLLW).

Mean high water is the average of the high water over a 19-year period, and **mean low water** is the average of the low water over a 19-year period. **Higher high water** is the higher of the two high waters of any diurnal tidal day, and **lower low water** is the lower of the two low waters of any diurnal tidal day. **Mean higher high water** is the average height of the higher high water over a 19-year period, and **mean lower low water** is the

Table 23.4 Mean and Spring Tidal Ranges for Some of the World's Major Ports*

	Mean range, ft	Spring range, ft
Anchorage, Alaska	26.7	29.6†
Antwerp, Belgium	15.7	17.8
Auckland, New Zealand	8.0	9.2
Baltimore, Md	1.1	1.3
Bilboa, Spain	9.0	11.8
Bombay, India	8.7	11.8
Boston, Mass.	9.5	11.0
Buenos Aires, Argentina	2.2	2.4
Burntcoat Head, Nova Scotia (Bay of Fundy)	41.6	47.5
Canal Zone, Atlantic side	0.7	1.1†
Canal Zone, Pacific side	12.6	16.4
Capetown, Union of South Africa	3.8	5.2
Cherbourg, France	13.0	18.0
Dakar, Africa	3.3	4.4
Dover, England	14.5	18.6
Galveston, Tex	1.0	1.4†
Genoa, Italy	0.6	0.8
Gibraltar, Spain	2.3	3.1
Hamburg, Germany	7.6	8.1
Havana, Cuba	1.0	1.2
Hong Kong, China	3.1	5.3†
Honolulu, Hawaii	1.2	1.9†
Juneau, Alaska	14.0	16.6†
La Guaira, Venezuela		1.0†
Lisbon, Portugal	8.4	10.8
Liverpool, England	21.2	27.1
Manila, Philippines		3.3†
Marseilles, France	0.4	0.6
Melbourne, Australia	1.7	1.9
Murmansk, U.S.S.R.	7.9	9.9
New York, N.Y.	4.4	5.3
Osaka, Japan	2.5	3.3
Oslo, Norway	1.0	1.1
Quebec, Canada	13.7	15.5
Rangoon, Burma	13.4	17.0
Reikjavik, Iceland	9.2	12.5
Rio de Janeiro, Brazil	2.5	3.5
Rotterdam, Netherlands	5.0	5.4
San Diego, Calif.	4.2	5.8†
San Francisco, Calif.	4.0	5.7†
San Juan, Puerto Rico	1.1	1.3
Seattle, Wash.	7.6	11.3†
Shanghai, China	6.7	8.9
Singapore, Malaya	5.6	7.4
Southampton, England	10.0	13.6
Sydney, Australia	3.6	4.5
Valparaiso, Chile	3.0	3.9
Vladivostok, U.S.S.R.	0.6	0.7
Yokohama, Japan	3.5	4.7
Zanzibar, Africa	8.8	12.4

* "Tide Tables," National Ocean Service.
† Diurnal range.

average height of the lower low waters over a 19-year period (tidal epoch). **Highest high water** and **lowest low water** are the highest and lowest, respectively, of the spring tides of record. **Mean range** is the height of mean high water above mean low water. The mean of this height is generally referred to as **mean sea level (MSL)**. **Diurnal range** is the difference in height between the mean higher high water and the mean lower low water.

The National Ocean Service annually publishes tide tables that give the time and elevation of the high and low tides at thousands of locations around the world and that can be used to forecast water levels at all times. The tide tables forecast the repeating, astronomical portions of the tide for specific locations but do not directly account for the day-to-day effects of changes in local winds, pressures, and other factors. Along most coasts, the tide table forecasts are within 1 ft of the actual water level 90% of the time.

Relative sea-level rise is gradually changing all of the epoch-based datum at any coastal site. Although, the datum that is used for design and construction throughout an upland area is not particularly important, the relation between construction and actual water levels in the coastal zone can be extremely important. The level of the oceans of the world has been gradually increasing for thousands of years. The important change is the relative sea-level change, the combined effect of water level and land-mass elevation changes due to subsidence (typical of the U.S. Atlantic and Gulf coasts) or rebound or emergence (Pacific coast of the U.S.). Measured, long-term tide data for major U.S. ports show that the relative sea-level rise differs from location to location. For example, at Galveston, Tex., there has been about 1 ft of relative sea-level rise during the last 50 years. At Anchorage, Alaska, there has been about 2 ft of relative sea-level fall during the last 50 years.

The impact of long-term sea-level rise has rarely been taken into account in design, except when it has already impacted the epoch-based tidal datum, such as MLLW. The National Geodetic Vertical Datum (NGVD) was established at the mean sea level (MSL) of 1929. Since sea-level rise has continued since then, the NGVD is now below the current day MSL along much of the U.S. Atlantic and Gulf coasts. At many locations, it is between the MSL and the MLLW. For accurate location of the NGVD relative to the MSL or MLLW, analysis with data from a local tide gage is required. For some harbor and coastal design, a staff gage is installed for recording water levels for a sustained period of time to confirm the relation between the local surveyor's elevation datum, the assumed tidal datum, and the actual water surface elevation.

Storm Surge ▪ This can be defined broadly to include all the effects involved in a storm, including wind stress across the continental shelf and within an estuary or body of water, barometric pressure, and wave-induced setup. The combined influence of these effects can change the water level by 5 to 20 ft depending on the intensity of the storm and coastal location. Engineers can use return-period analysis curves to estimate the likelihood of any particular elevation. The Federal Emergency Management Agency and the various Corps of Engineer Districts have developed such curves based on historic high-water-mark elevations and numerical models of the hydrodynamics of the continental shelf.

23.4 Coastal Sediment Characteristics

Most beach sediments are sand. The day-to-day dynamics of the surf zone usually ensure that most fines, silts, and clays will be washed away to more quiescent locations offshore. Some beaches have layers of cobbles, rounded gravel, or shingles, flattened gravel.

The size and composition of beach sands varies around the world and even along adjacent shorelines. Essentially, the beach at any particular site consists of whatever loose material is available. Quartz is the most common mineral in beach sands. Other constituents in sands include feldspars and heavy minerals. Some beaches have significant portions of seashell fragments and some beaches are dominated by coral carbonate material.

Beach sands are usually described in terms of grain-size distribution. The median diameter d_{50} is a common measure of the central size of the distribution. The range of the distribution of sand sizes around this median is usually discussed in terms of sorting.

The color of the sand depends primarily on the composition of the grains. The black sand beaches of Hawaii are derived from volcanic lava. The white sands of the panhandle of Florida are quartz that has developed a white color owing to miniature surface abrasions and bleaching.

23.5 Nearshore Currents and Sand Transport

As wave energy enters the surf zone, some of the energy is transformed to nearshore currents and expended in sand movement. The nearshore current field is dominated by the incident wave energy and the local wind field. The largest currents are the oscillatory currents associated with the waves. However, several forms of mean currents (longshore currents, rip currents associated with nearshore circulation cells, and downwelling or upwelling associated with winds) can be important to sand transport.

Longshore current is the mean current along the shore between the breaker line and the beach that is driven by an oblique angle of wave approach. The waves provide the power for the mean longshore current and also provide the wave-by-wave agitation to suspend sand in the current. The resulting movement of sand is **littoral drift** or **longshore sand transport**. This process is referred to as a *river of sand* moving along the coast. Although the river-of-sand concept is an effective, simple explanation of much of the influence of engineering on adjacent beaches, the actual sand transport paths are more complex. This is particularly so near inlets with large ebb-tidal shoals that influence the incident wave climate.

Even on an open coast with straight and parallel offshore bottom contours, the longshore-sand-transport direction changes constantly in response to changes in the incident wave height, period, and direction. The so-called **CERC equation**, or **energy flux method**, provides a rough estimate of the instantaneous longshore sand transport rate. The instantaneous rate can vary from nil up to several million cubic yards per year in either direction during storms on some coasts. The variations in the instantaneous rate are so significant that 90% of the sand transport occurs during only 10% of the time—the storms. Even when averaged over a year, the net sand transport rate can vary significantly from year to year and can even change directions. The daily variability in the longshore-sand-transport rate follows a joint, log-normal distribution. This can be used to simulate rates for the design of sand bypassing systems.

Knowledge of the long-term transfer of sand across the continental shelf to or from beaches is limited. For short time frames, the cross-shore transport of sand during storms can be modeled to estimate the beach and dune erosion and the shoreline recession that will occur during some design storm.

Harbor and Marina Engineering

A **harbor** is a bay, cove, inlet, or recess of the sea or a lake, or the mouth of a river in which ships can enter and be sheltered from wind and waves. A **port** is a harbor with facilities for the docking of ships, cargo handling and storage, and transfer of passengers between land and waterborne transportation. A **marina** is a shallow draft harbor for small, predominately recreational craft. Small-craft harbors accommodate commercial operations or waterborne transportation operations, or both, as well as recreational boats of various sizes. Harbor and marina engineering is concerned with the design of navigable waterways in harbors, protective structures, docks, and the facilities for servicing boats or ships.

23.6 Types of Ports and Harbors

Harbors may be classified as natural, seminatural, or artificial, and as harbors of refuge, military harbors, or commercial harbors. Commercial harbors may be either municipal or privately owned.

A **natural harbor** is an inlet or water area protected from storms and waves by the natural configuration of the land. Its entrance is so formed and located as to facilitate navigation while ensuring comparative quiet within the harbor. Natural harbors are located in bays, tidal estuaries, and river mouths. Well-known natural harbors are New York, San Francisco, and Rio de Janeiro.

A **seminatural harbor** may be an inlet or a river sheltered on two sides by headlands requiring artificial protection only at the entrance. Next to a purely natural harbor, it forms the most desirable harbor site, other things being equal. Plymouth and Cherbourg take advantage of their natural location to become well-protected harbors by the addition of detached breakwaters at the entrances.

An **artificial harbor** is one protected from the effect of waves by breakwaters or one created by dredging. Buffalo, New York; Matarani, Peru; Hamburg, Germany; and Le Havre, France, are examples of artificial harbors.

A **harbor of refuge** may be used solely as a haven for ships in a storm, or it may be part of a commercial harbor. Sometimes an outer harbor serves as an anchorage, while a basin within the inner breakwater constitutes a commercial harbor. The essential features are good anchorage and safe and easy access from the sea during any condition of weather and state of tide. Well-known harbors of refuge are the one at Sandy Bay, near Cape Ann, Massachusetts, and that at the mouth of Delaware Bay. A fine example of a combined harbor of refuge and commercial harbor exists at Dover, England.

A **military harbor** or naval base accommodates naval vessels and serves as a supply depot. Guantanamo, Cuba; Hampton Roads, Virginia; and Pearl Harbor, Hawaii, are some well-known naval bases.

A **commercial harbor** is one in which docks are provided with the necessary facilities for loading and discharging cargo. Drydocks are sometimes provided for ship repairs. Many commercial harbors are privately owned and operated by companies representing the steel, aluminum, copper, oil, coal, timber, fertilizer, sugar, fruit, chemical, and other industries. Municipal- or government-controlled harbors, often operated by port authorities, exist in many countries and are usually part of extensive port works, such as the harbors in New York, Los Angeles, and London.

A **port** is a harbor where marine terminal facilities are provided. These consist of piers or wharves at which ships berth while loading or unloading passengers and cargo, transit sheds and other storage areas where ships may discharge incoming cargo, and warehouses where goods may be stored for longer periods while awaiting distribution or sailing. The terminal must be served by railroad, highway, or inland-waterway connections. In this respect the area of influence of the port reaches out for a considerable distance beyond the harbor.

A **port of entry** is a designated location where foreign goods and foreign citizens may be cleared through a custom house.

23.7 Harbor Layout

The number and size of ships using a harbor determine its size to a large extent, but existing site conditions are also an important influence. Generally, unless the harbor is a natural one, its size will be kept as small as feasible for safe and reasonably comfortable operations to take place. Use of tugs to assist maneuvering of ships in docking may also influence the size of the harbor.

23.7.1 Turning Basins

A turning basin is a water area inside a harbor or an enlargement of a channel to permit the turning of a ship. When space is available, the area should have a radius of at least twice the length of the ship to permit either free turning or turning with the aid of tugs, if wind and water conditions require. When space is limited, the ship may be turned by warping around the end of a pier or turning dolphin, either with or without the use of its lines. In those cases, the turning basin will be much smaller and of a more triangular or rectangular shape. The minimum diameter should be at least 20% greater than the length of the largest ship to be turned.

The usual minimum harbor area is the space required for docks plus a turning basin in front of them. In some layouts, where a ship is turned by warping it around the end of the pier or turning dolphin, the harbor may be even smaller. For instance, a minimum harbor with a single pier and turning basin and a long approach channel from the open sea (Fig. 23.3) can accommodate two 500-ft ships. This artificial harbor may be formed by dredging a channel through shallow water, protected by offshore reefs and islands, and enlarging the inshore end to provide the minimum area of harbor that will meet the shipping requirements specified for the project. In leaving its berth, a ship must warp itself around the end of the pier so as not to have to back out through the long approach channel.

Another, less restricted type of harbor is nearly square, protected by two breakwater arms, with one opening. The harbor has several docks and a turning basin with an area sufficient to inscribe a

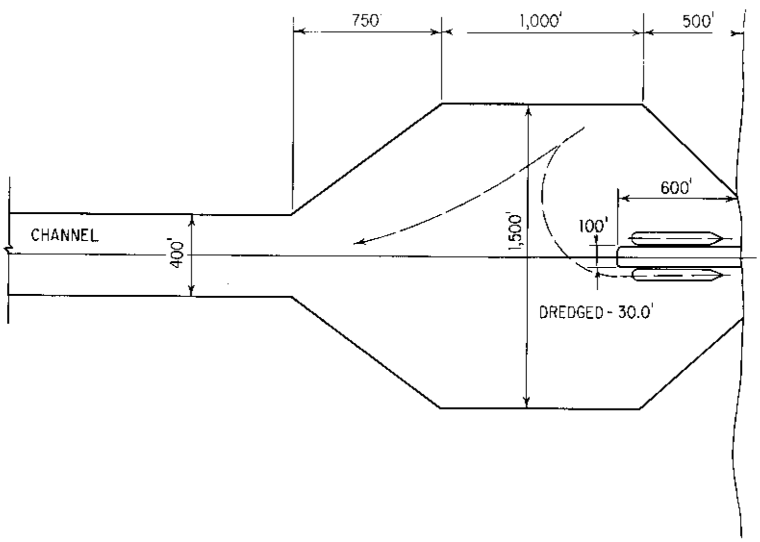

Fig. 23.3 Typical layout for a small artificial harbor.

turning circle with a radius equal to at least twice the length of the largest ship. This is the smallest radius a ship can comfortably turn on, under continuous headway, without the help of a tug. Figure 23.4 shows such a harbor.

23.7.2 Arrangements of Breakwaters

Breakwaters are required for protection of artificial and seminatural harbors. Their location and extent depend on the direction of maximum waves, configuration of the shoreline, and minimum size of harbor required for the anticipated traffic in the port. They may consist of two "arms" out from the shore, plus a single breakwater, more or less parallel to the shore, thereby providing two openings to the harbor; or the harbor may be protected with a single arm out from shore. Or the harbor may be protected by two arms converging near their outshore ends and overlapping to form a protected entrance to the harbor.

Selection of the most suitable arrangement of breakwaters depends principally on the direction of the maximum waves. The effectiveness of the chosen arrangement in quieting the harbor may be checked by model tests. For comfortable berthing, the wave height should not exceed 2 ft and winds should not exceed 10 to 15 mi/h. But wave heights up to 4 ft have been allowed where bulk cargo is being handled and where the wind direction is such as to hold a docked ship off the dock. In general, winds and current are more bothersome in docking a vessel when it is light than are relatively small harbor waves and may necessitate use of a tug.

Rarely will a location be found where the waves are from one direction only. Generally, it is better

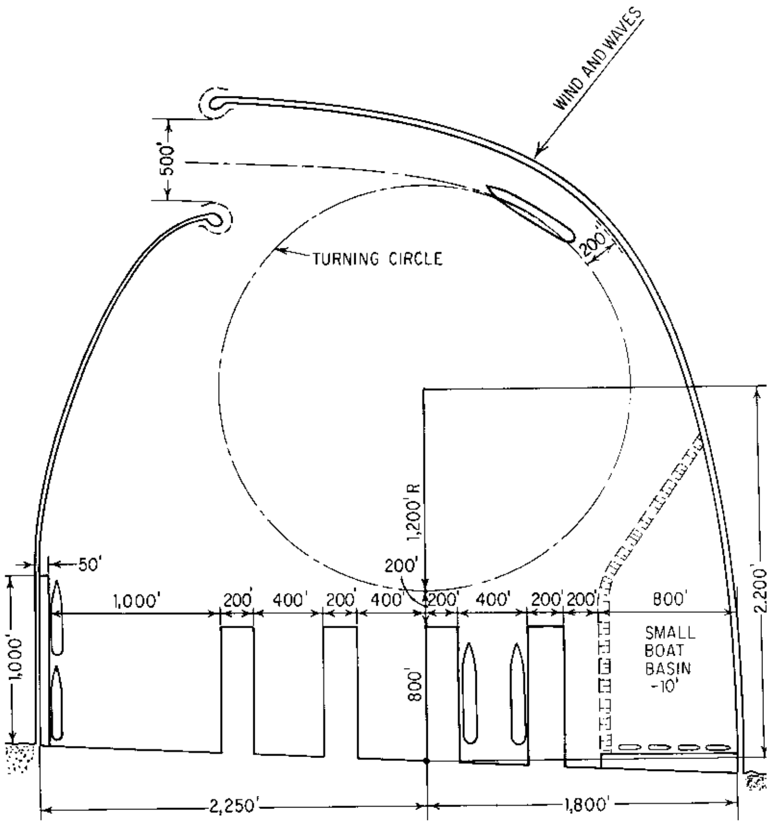

Fig. 23.4 Medium-sized artificial harbor with full-sized turning basin.

in a harbor having two openings for ships to enter from the direction of the minimum wind and waves and to leave toward the direction of the maximum wind and waves. On leaving the harbor, the ships usually have open water in which to maneuver, whereas on entering the harbor, they are immediately in a restricted area and must approach the docks at reduced speed and at a certain inclination to its face.

A single breakwater arm may be used where the waves predominate from one direction only. It also may serve where the configuration of the shoreline reduces the fetch in the opposite direction to such an extent that the wave-generating area is not sufficient to permit formation of bothersome waves within the harbor.

23.7.3 Use of Moorings and Anchorages

An **offshore mooring** is provided usually where it is not feasible or economical to construct a dock or provide a protected harbor. Such an anchorage consists of a number of anchorage units, each consisting of one or more anchors, chains, sinkers, and buoys to which the ship will attach its mooring lines. These anchorages are supplemented in most cases by the ship's bow anchors. Bulk cargo is usually transported to or from the ship by pipeline or trestle conveyor; other cargo may be transferred by lighter.

An **anchorage area** is a place where ships may be held for quarantine inspection, to await docking space, sometimes while removing ballast in preparation for taking on cargo, or to await favorable weather conditions. Special anchorages are sometimes provided for ships carrying explosives or dangerous cargo and are usually so designated on harbor maps by name and depth of water.

23.7.4 Harbor Entrances

To reduce wave height within a harbor, entrances should be no wider than necessary for safe navigation and for preventing dangerous currents when the tide is coming in and going out. The entrance width should be in proportion to the size of the harbor and the ships using it. In general, the following widths will be satisfactory: small harbors, 300 ft; medium harbors, 400 to 500 ft; and large harbors, 500 to 1000 ft. When the entrance is between breakwaters with sloping faces, the width is measured at the required harbor or channel depth below low water. Thus, the entrances will be appreciably wider than the recommended widths at low-water level. In such cases, it is advisable to mark the full harbor depth of the entrance with buoys, placing one or more on each side of the entrance channel.

The entrance should be on the lee side of the harbor, where possible. If the entrance must be located at the windward end of the harbor, breakwaters should overlap so vessels may pass through the restricted entrance and be free to turn with the wind before being hit broadside by the waves. Also, the interior of the harbor will be protected from the waves.

When the entrance to a harbor is unobstructed, storm waves from the sea pass through the opening into the harbor. Unless they are reflected by a vertical surface, they will gradually decrease in height as they progress away from the entrance and as the harbor widens relative to the entrance width. Model tests will give an indication of wave conditions and are essential for studying various arrangements of breakwaters for important harbors.

In tidal harbors where there are strong currents, the entrance width should be sufficient to prevent the velocity of the current through the opening at ebb tide from exceeding 4 ft/s; otherwise, it may affect navigation of ships and create scour at the base of adjacent breakwaters.

If waves pass through an entrance and strike a vertical face on the opposite side of the harbor, they reflect. The result is an increase in wave height within the harbor. This condition can be corrected by building wave-absorbing beaches, flat slopes of rock or granular material, in front of the vertical surface. However, when the vertical surface is a wharf or bulkhead used for berthing of ships, it is impossible to use a beach. Where conditions will permit use of a hollow structure, the outside vertical wall may be perforated or slotted and the energy of the waves absorbed in the chamber in back of the wall. Other means may be resorted to, such as installation of short wave-deflecting walls or wave traps along the approach channel to the dock.

23.7.5 Channel Depth

The harbor and approach channel for ideal operating conditions should be of sufficient depth to permit navigation at lowest low water when ships are fully loaded. This depth must include an allowance for the surge of the ship, which is about one-half the wave height, the out-of-trim or squat when in motion, and from 2- to 4-ft clearance under the keel, the larger figure being used when the bottom is of hard material such as rock. When there is a very soft mud bottom, a keel may at times touch bottom because of surge and squat without damaging the ship, but it would be disastrous to have its fully loaded weight bump a hard rock bottom. Therefore, greater allowances must be made in computing the depth when the bottom is hard. Also, the harbor and approach channel or approach sea lanes must be carefully swept to make sure there are no obstructions, such as reefs or rocky pinnacles, boulders, or sunken ships, above the required depth for safe navigation.

In some harbors, some ships arrive or depart on the rising tide. But the advent of supertankers with deadweight tonnage over 300,000 tons and a draft exceeding 80 ft creates a need for deeper harbors. Otherwise, the cargo has to be transferred to smaller tankers or piped ashore through submarine lines from offshore anchorages.

Tides significantly influence harbor depth. Table 23.4 (p. 23.8) gives the tidal ranges in feet at principal ports throughout the world. Note that the tidal range along the coasts of the United States seldom exceeds 10 ft, and therefore the harbors are dredged to provide the required depth for navigation at lowest low water. The condition is entirely different in the British Isles and on the western coast of Europe, where the port of Liverpool, England, has a spring tidal range of 27 ft; London, England, 20 ft; Calais, France, 20 ft; and others have even greater variations. In most cases, this fluctuation in sea level has resulted in the use of wet docks in all stages of the tide. These dock systems require entrance locks with massive gates, heavy swing or bascule bridges and the machinery for working them, pumping equipment, and other accessories. Since all this results in great cost, as well as continuing operational and maintenance expense, the question arises as to the limiting range for the natural tidal working of ports without recourse to enclosed docks. Generally, about 10 to 15 ft is considered the dividing point.

23.7.6 Channel Width

Width of a channel may be measured between the toes of the bordering side slopes or at the design depth.

Minimum nominal width required for a channel depends on many factors. The following are the most important:

1. Maximum beam of traversing ships
2. Length and maneuverability of the longest vessels
3. Accuracy and reliability of navigational aids
4. Speed, volume, and nature of traffic
5. Nature, intensity, and variation of currents along the channel
6. Ability and experience of mates and pilots
7. Channel depth and curvature
8. Whether ships are to pass each other

Because of insufficient knowledge of the influence of these factors when determining a minimum design width, standard channel widths have not been established. The Permanent International Association of Navigation Congress, however, recommended in 1978 that the nominal width of a one-way channel under ideal conditions should be at least five times the beam of the largest ship expected to use the channel. On a curve, this width should be increased by $L^2/8R$, to account for the effect on the width of the ship's path because of the length L of the vessel and the radius R of the curve. The width on a curve should be additionally increased to take into account maneuvering difficulties. If ships are to pass in a one-way channel, the nominal width of the channel should be at least 12 times the largest beam (ship width). For a two-way channel, the width should be adequate to provide a distance between passing ships of at least two times the beam of the larger vessel. The preceding recommended minimum widths should be increased, if necessary, to allow for the effects of crosswinds and crosscurrents.

23.7.7 Channel Alignment

Basically, a channel should provide a path for a ship through the harbor entrance directly to a berth without requiring difficult maneuvering or subjecting the ship to strong crosscurrents or high waves. If the channel must be curved, the radius of the curves should be at least five times the length of the longest ship expected to use the channel.

Straight tangents should be used between successive curves. (S curves should be avoided.)

The tangents should have a length of at least 10 times that of the longest vessel.

In a sea strait, the channel preferably should coincide with the deepest troughs of the strait and have as few curves as possible. Sight distance should be at least 0.5 mi.

23.7.8 Port Structures

A marine terminal is that part of a port or harbor that provides docking, cargo-handling, and storage facilities. When only passengers embark and disembark along with their baggage and miscellaneous small cargo, generally from ships devoted mainly to the carrying of passengers, it is called a *passenger terminal*. When the traffic is mainly cargo carried by freighters, although many of these ships may carry also a few passengers, the terminal is commonly referred to as *freight* or *cargo terminal*. In many cases, it is known as a *bulk cargo terminal*, where such products as petroleum, cement, and grain are stored and handled.

Docking facilities may consist of a single pier or as many as 1000 piers. The number of berths depends on the anticipated number of ships that will use the port and the time it will take to discharge and take on cargo or passengers. This will vary for different kinds of cargo, but usually a vessel will not be in port more than 48 h. Many bulk cargo ships are loaded in 24 h or less.

Wharves and piers should be located in the most sheltered part of the harbor or along the lee side of the breakwaters. Where possible, piers should be so oriented as to have ships alongside headed as nearly into the wind and waves as possible. This is particularly important if the harbor is not well-protected.

Onshore marine-terminal facilities may consist of one or more of the following, depending on the size of the port and the service it renders:

Transit sheds are located immediately in back of the apron on a pier or wharf. Their function is to store for a short period of time cargo awaiting loading or distribution after being unloaded from ships.

Warehouses may replace transit sheds at some marine terminals. But when used to supplement sheds, warehouses are usually located inland and not on the pier structure.

Bulk storage may be in open piles over conveyor tunnels, which may be covered with sheds when protection from the elements is required; in bins and silos or elevators (for grain storage); or in storage tanks (for liquids). These should be located as near the waterfront as possible, and sometimes directly alongside the wharf or pier, to enable direct loading into the hold of the ship.

A **terminal building** houses port-administration personnel and custom officials if a separate custom house is not provided. The terminal building should be located in a prominent and convenient location with respect to the docks.

Guardhouses are located at strategic points in the port area, such as the entrance gates of highways and railways, entrances to piers or terminal areas, bonded storage, and so on.

Stevedores' warehouses house cargo-handling gear, wash and locker rooms, and other facilities for stevedores.

Miscellaneous buildings and structures include a fire house and fire-fighting equipment, power plant, garages, repair shops, drydocks, marine railways, fishing piers, or yacht basins.

(P. Bruun, "Port Engineering," 4th ed., Gulf Publishing Company, Houston, Tex (www.gulfpub.com).)

23.8 Hydrographic and Topographic Surveys

After preliminary layouts of a port have been completed and before the final design is started, it is necessary in most instances to obtain additional site information.

A **hydrographic survey**, if not already available, should be made to determine the elevations of the bottom of the body of water and should extend over an area somewhat larger than the proposed channel and harbor. In addition, the survey should locate the shoreline at low and high water and all structures or obstructions in the water and along the shore, such as sunken ships, reefs, or large rocks.

Determination of the relief of the bottom of the body of water is made by soundings or by the use of a fathometer designed for hydrographic surveys. The latter method is being used by the National Ocean Survey and has superseded lead-line soundings to a large extent. A fathometer or depth-recording instrument is usually mounted in a motorboat, which is kept on course on established range lines or its location is followed by use of an electronic positioning system as the recording chart registers a natural profile of the bottom. The fathometer, when operated by experienced personnel and properly adjusted and calibrated daily, is superior to lead-line soundings in both accuracy and speed with which a survey can be made.

The depths of soundings are referred to water level at the time made and later corrected to the datum water level with tide gages or tide tables. Therefore, it is important to keep a record of the time and day the soundings are made and to have a concurrent tide- or water-level gage operating in the immediate vicinity.

Soundings should be made at about 25-ft intervals along lines from 50 to 100 ft apart, depending on the irregularity of the bottom. Closer spacing may be needed where greater detail is required to determine sharp changes in the profile of the bottom or to outline obstructions.

Soundings are plotted, usually relative to low-water datum, on a drawing (hydrographic map), which should show the datum, high- and low-water lines, contour lines of equal depth interpolated from the soundings, and principal land and water features. Contour depths may be in either feet, meters, or fathoms, although the last is not used generally for making harbor and marine-terminal studies and layouts. Since the sea bottom is usually less precipitous and the slopes more gentle and uniform than those on land, the scale of the hydrographic map may be somewhat smaller than would normally be used for plotting land topography. Unless the harbor area is very large, a scale of 1 in = 200 ft or 1:2000 in a proportional scale is satisfactory. It is desirable to have all the hydrography on one sheet because this gives a better overall picture of the harbor. In general, the scale should be large enough so that not more than 10 contour lines, in 2-ft intervals, occur within 1 in.

If dredging of a harbor or channel is required, the material is usually measured in place to determine the quantity for payment. To determine this quantity, soundings on fixed sections are taken before and after dredging, and the changes in cross sections are determined by computation or planimeter. It is usually specified that payment will be made for material removed to a maximum of 2 ft below the required dredged bottom, but all material must be removed to at least the minimum depth specified.

A **topographic survey** of the marine-terminal area should be made, to obtain ground contours at 2- to 5-ft intervals. The larger figure is used where terrain is rough and in areas where there is to be little or no construction of importance. In building areas, elevations on 25-ft centers in two directions, with additional elevations at abrupt changes in ground, provide satisfactory information. Where there is dense ground cover, the cross-profile method is most suitable. The profiles may be made with level and tape or stadia, on about 100-ft centers, by clearing paths to permit an unobstructed line of sight. The ground between the 100-ft profiles should be examined, as far as possible, and any prominent irregularities in ground level estimated and noted, so that contours, which are interpolated from elevations along the profiles, can be estimated for the areas in between.

Topographic maps, besides showing the contours of the ground, should locate all borings and test pits, buildings, utilities, and any prominent landmarks. Contours generally are referred to high-water datum. The map scale should be such that the contour lines are not spaced closer than 30 to the inch. Where considerable detail is involved the scale should be 1 in = 100 ft or 1:1000 or less, but for small-scale maps 1 in = 1000 ft or 1:10,000 or more may be used.

(H. Agerschou et al., "Planning and Design of Ports and Marine Terminals," John Wiley & Sons, Inc., New York (www.wiley.com); J.B. Gerbich, "Handbook of Coastal and Ocean Engineering," Gulf Publishing Company, Houston, Tex. (www.gulfpub.com); A. D. F. Quinn, "Design and Construction of Ports and Marine Structures," McGraw-Hill Publishing Company, New York (books.mcgraw-hill.com).)

23.9 Ship Characteristics

The length, beam (breadth), and draft of ships that will use a port have a direct bearing on the design of the approach channel, harbor, and marine-terminal facilities. The last is affected also by the type of vessel, its wind area, and its capacity or tonnage. The **draft** of a ship, expressed in relation to its displacement of water as either loaded or light draft, is the depth of the keel of the ship below water level for the particular condition of loading.

Displacement tonnage is the actual weight of the vessel, or the weight of water displaced when afloat, and may be either "loaded" or "light." **Displacement loaded** is the weight, in metric tons, of the ship and its contents when fully loaded with cargo to the Plimsoll mark, or load line, painted on the hull of the ship (1 ton = 2205 lb). The **Plimsoll mark**, used on British ships, and the **load line**, commonly used on American vessels, designate the depth under the maritime laws to which a ship may be loaded in different bodies of water during various seasons of the year. **Displacement light** is the weight, in metric tons, of the ship without cargo, fuel, and stores.

Deadweight tonnage is the carrying capacity of a ship in metric tons and the difference between displacement light and displacement loaded to the Plimsoll mark or load line. It is the weight of cargo, fuel, and stores a ship carries when loaded to the load line, as distinguished from loaded to space capacity. This tonnage varies with latitude and season. It also depends on salinity of the water because of the effect of temperature and salinity on the specific gravity and buoyancy of the water in which the vessel is operating. Unless otherwise indicated, deadweight tonnage is the mean of tropical, summer, and winter deadweight. Deadweight tonnage is indicated by weight, and gross tonnage by volume measurement; both indicate carrying capacity.

Ships are registered with gross or net tonnage expressed in units of 100 ft^3. **Gross tonnage** is the entire internal cubic capacity of a ship, and **net tonnage** is the gross tonnage less the space provided for the crew, machinery, engine room, and fuel.

Cargo or **freight tonnage**, a commercial expression, is the basis of the freight charge. This tonnage may be measured by either weight or volume.

An ordinary seagoing vessel that can carry a nominal deadweight of 8000 tons of cargo, fuel, and stores will have a displacement of about 11,500 tons, a gross of about 5200 tons, and a net of about 3200 tons.

Ballast is the weight added in the hold or ballast compartments of a ship to increase its draft after it has discharged its cargo and to improve its stability. It usually consists of water and is expressed in long tons. In an oceangoing tanker, saltwater ballast replaces a certain amount of petroleum when the ship is unloaded, whereas a dry-cargo or passenger vessel has separate compartments for ballast.

23.10 Types of Ship Mooring Structures

Facilities for mooring of ships are major elements of ports and harbors. Such facilities include docks, wharves, bulkheads, piers, dolphins, fixed mooring berths, and ground tackle in fixed positions for attachment of a ship's mooring lines. Appurtenances for these facilities include fenders for absorption of ship impact during mooring or departure, trestles, catwalks, bitts, bollards, cleats, chocks, hooks, and capstans.

A **dock**, in general, is a marine structure for mooring or tying up of vessels, loading and unloading cargo, or embarking and disembarking passengers. Often, piers, wharves, bulkheads, and, in Europe, jetties, quays, or quay walls, are called docks. In Europe also, where there are large variations in tide level, a dock is commonly considered an artificial basin for vessels and is called a **wet dock**. When the basin is pumped out, it is termed a **dry dock**.

A **wharf** or **quay** is a dock that parallels the shore. It is generally contiguous with the shore but may not necessarily be so. On the other hand, a **bulkhead** or **quay wall**, although similar to a wharf and often referred to as such, is backed up by ground; the name is derived from the very nature of holding or supporting ground in back of it.

In many locations where industrial plants are to be built adjacent to water transportation, the ground is low and marshy; it is therefore necessary to fill it in. The fill is often obtained by dredging the adjacent waterway, creating a navigable channel or harbor along the property. To retain the made ground, which will now be at a much higher elevation along the waterway, a bulkhead is usually installed. This, or a part of its length, may be used as a wharf for docking vessels if mooring appurtenances, paving, and facilities for handling and storing cargo are added. It is then called a **bulkhead wharf**.

A **pier** or **jetty** is a dock that projects into the water. Sometimes it is referred to as a **mole**. When built in combination with a breakwater, it is termed a **breakwater pier**. In contrast with a wharf, which can be used for docking on one side only, ships may use a pier on both sides. But there are instances where only one side is used, owing to either the physical conditions of the site or the lack of need for additional berthing space.

A pier may be more or less parallel to the shore and connected to it by a mole or trestle, generally at right angles to the pier. In this case, the pier is commonly referred to as a **T-head pier** or **L-shaped pier**, depending on whether the approach is at the center or at the end.

Dolphins are marine structures for mooring vessels. They are commonly used in combination with piers and wharves to shorten the length of these structures. Dolphins are a principal part of the fixed-mooring-berth type of installation used extensively in bulk-cargo loading and unloading installations. Also, they are used for tying up ships and for transferring cargo between ships moored along both sides of the dolphins. There are two types of dolphins: breasting and mooring.

Breasting dolphins, usually the larger of the two types, are designed to take the impact of a ship when docking and to hold the ship against a broadside wind. Therefore, they are provided with fenders to absorb the impact of the ship and to protect the dolphin and ship from damage. Breasting dolphins usually have bollards or mooring posts to take the ship's lines, particularly springing lines for moving a ship along the dock or holding it against the current. These lines are not very effective in a direction normal to the dock, particularly when a ship is light.

To hold a ship against a broadside wind blowing in a direction away from the dock, additional dolphins must be provided off the bow and stern, some distance in back of the face of the dock. These are called *mooring dolphins*. They are not designed for the impact of the ship since they are away from the face of the dock, where they will not be hit. If two mooring dolphins are to be used, they should be located about 45° off the bow and stern and permit mooring lines not less than 200 ft or more than 400 ft long. The largest ships may require two additional dolphins, off the bow and stern. These dolphins are usually located so that the mooring lines will be normal to the dock, which makes them most effective for holding the ship against an offshore wind. Mooring dolphins are provided with bollards or mooring posts and with capstans when heavy lines are to be handled. The maximum pull usually should not exceed 50 tons on a single line, or 100 tons on a single bollard if two lines are used.

A **fixed mooring berth** is a marine structure consisting of dolphins for tying up a ship and a platform for supporting the cargo-handling equipment. The platform is usually set back 5 to 10 ft from the face of the dolphins so that the ship will not come in contact with it. Therefore, the platform does not have to be designed to take the impact of the ship when docking.

Offshore moorings for ships consist of ground tackle placed in fixed positions for attaching a ship's lines. Each unit of ground tackle consists of one or more anchors with chain, sinker, and buoy to which the ship's line is attached. These mooring units are usually located so as to take the bow and stern lines and, if the ship is large, one or more breasting lines. For some moorings, where the wind is in one direction, the ship may use its own bow anchor and the fixed tackle off the bow may be omitted.

(P. Bruun, "Port Engineering," Gulf Publishing Company, Houston, Tex. (www.gulfpub.com).)

23.11 Dock and Appurtenance Design for Ship Mooring

Wharves, piers, bulkheads, and fixed mooring berths fall generally into two broad classifications: docks of open construction with their decks supported by piles or cylinders; and docks of closed or solid construction, such as sheetpile cells, bulkheads, cribs, caissons, and gravity (quay) walls.

The following codes, references, and standards are recommended as a basis for analysis and design of a wharf structure, fill containment structure, mooring and fender devices, and associated equipment.

Primary Codes for Design

- American Concrete Institute (ACI) "Building Code Requirements for Reinforced Concrete"
- American Institute of Steel Construction (AISC) "Manual of Steel Construction"
- Uniform Building Code (UBC)
- American Society of Civil Engineers (ASCE) "Minimum Design Loads for Buildings and Other Structures"

Information and Reference

- American Association of State Highway and Transportation Officials (AASHTO) "Standard Specification for Highway Bridges" (Reference Document)

- Military Handbook "Piers and Wharves" (MIL-HDBK-1025/1) (Reference Document)
- Naval Facilities Engineering Command (NAVFAC) Design Manual "Fixed Moorings" (DM 26.4) (Reference Document)
- Naval Facilities Engineering Command (NAVFAC) Design Manual "Seawalls, Bulkheads, and Quaywalls" (DM 25.04) (Reference Document)
- U.S. Army Corps of Engineers (USACE) "Shore Protection Manual"
- U.S. Army Corps of Engineers (USACE) "Coastal Engineering Manual"

Two structure classifications are considered for wharf design:

- Open-Type Marginal Wharf
- Solid-Type Marginal Wharf

These options and their methods of construction are discussed below.

23.11.1 Open-Type Marginal Wharf

A marginal wharf is a wharf structure that is connected to the upland shore area along its full length. Such an arrangement is adaptable to the land transfer of containers, luggage, and passengers for cruise and/or cargo transport. The alternatives that lend themselves to this arrangement are the open-type marginal wharf with reveted slope, and the solid-type marginal wharf.

For an open-type marginal wharf with reveted slope, the wharf geometry is dictated by the relatively deep-water conditions at the face of the berth.

The following five pile supported structure configurations for use as open-type marginal wharf structures with reveted slope.

- Precast Box Beam
- Cast-in-Place Flat Plate
- Composite Panel Flat Plate
- Cast-in-Place Slab and Bent
- Ballasted Flat Plate

A summary of advantages and disadvantages of the open type marginal wharf alternatives are provided in Table 23.5.

Precast Box Beam Wharf ▪ The first marginal open wharf alternative presented consists of a composite concrete deck surface supported by precast box beams which in turn are supported by concrete bent caps and prestressed concrete piles. A schematic drawing of the precast box beam alternative can be found in Fig. 23.5.

The main working surface is a concrete composite deck composed of precast panels and a cast-in-place concrete topping slab. The deck would be supported by standard AASHTO precast prestressed concrete box beams which would be spaced to optimize structure costs.

The advantages of this alternative include minimization of the total number of piles supporting the wharf structure and maximization of the use of precast construction, which will reduce the field labor and construction time required. Precast deck panels would serve as a bottom form for the cast-in-place concrete deck thus reducing costly formwork placement and removal. The slab design is such that the cast-in-place deck slab will be composite with the precast bottom form.

The disadvantages of this alternative are related to the use of precast elements. The size of the precast box beams will require specialized material handling equipment to place the box beams. The use of precast elements results in the need for tighter construction tolerances. Unlike with cast-in-place construction where adjustments are easily made to accommodate misalignments or elevation variations, precast construction requires very accurate construction and constant monitoring of precast element placement. Finally, precast construction inherently has two problems—special detailing required for continuity of connections between individual precast elements, and high cost of specialty elements. Continuity is required because vertical and lateral loads must have a continuous, structural pathway between precast elements to allow transfer of these loads to the piles and subsequently to the soils.

Cast-in-Place Flat Plate Wharf ▪ The second marginal, open wharf type is a pile supported cast-in-place concrete flat plate slab. The cast-in-place concrete deck provides two-way slab action to resist vertical and lateral loading. This type of construction is relatively new in the heavy waterfront construction industry but has been gaining in popularity due to the initial construction cost savings that can be achieved.

Table 23.5 Open Type Marginal Wharf Alternatives

Precast Box Beam	Cast-in-Place Flat Plate	Composite Panel	Cast-in-Place Slab and Bent	Ballasted Cast-in-Place Flat Plate
		ADVANTAGES		
Minimizes number of piles required Maximizes use of precast elements Minimizes field labor Reduced construction time Reduces need for formwork	Affords economy of materials Provides highest degree of structural continuity Minimizes formwork costs Less specialized equipment required for construction	Maximizes use of precast elements Minimizes field labor Precast elements are relatively small Requires no special equipment for construction Reduced construction time	Construction tolerances are not as critical Conventional type of construction Easy to form recessed utility pits, trenches, and other items No special equipment required for construction	Affords economy of materials Provides highest degree of structural continuity Minimizes formwork costs Less specialized equipment required for construction Flexibility for adding rail and utilities
		DISADVANTAGES		
Requires special materials handling equipment Requires tighter construction tolerances Requires deeper pile embedment Requires special detailing for continuity of connections Difficult to expand for rail	Requires highly qualified labor Labor intensive construction Increased construction time Requires special detailing for ductility Requires special concrete casting sequence Difficult to expand for rail	Requires tighter construction tolerances Requires special detailing for continuity of connections Requires a large member of piles Difficult to expand for rail	Highly dependent on qualified labor Labor intensive construction Increased construction time Requires special concrete casting sequence Formwork relatively costly Difficult to expand for rail	Requires highly qualified labor Labor intensive construction Increased construction time Requires special detailing for ductility Requires special concrete casting sequence

Note: All open-type wharves require excavation and complete demolition of existing structures.

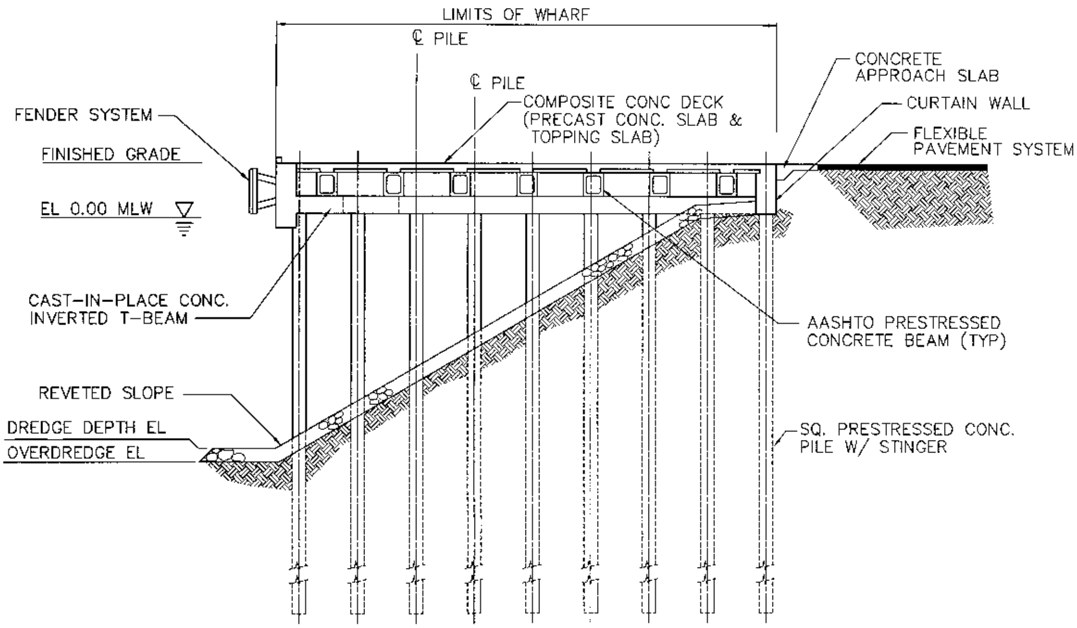

Fig. 23.5 Precast box beam open wharf.

The flat plate design consists of a cast-in-place concrete deck, which provides two-way slab action to resist vertical and lateral loading. A drop-panel fascia on the waterside face of the wharf accommodates the dock fender system and the mooring devices, as well as providing a transition area that would accommodate any required utility or service pits.

The flat plate slab would be supported by precast prestressed concrete piles. Piles are spaced to match slab and pile capacities. The advantages of the flat plate alternatives include minimization of forming costs for cast-in-place concrete construction and economy of materials. The driven piles act as supports for the bottom form, which is level. Because the structure is principally cast-in-place construction, less specialized equipment is required during construction. Most general contractors can perform the necessary forming and concrete placement required. Finally, since the structure consists only of cast-in-place concrete and precast piles, there are fewer structural components to integrate together, and thus, this design provides a high degree of structural continuity.

The disadvantages associated with this design are related to the extensive use of cast-in-place concrete. The techniques employed for construction are labor intensive. This includes the placement of formwork and the need to strip the formwork and move it to the next slab unit being placed. To minimize the time on the site and to account for shrinkage effects, a special concrete casting sequence must be used. A large pool of relatively skilled workmen will be required. In addition, this type of construction requires more time in the field than a precast alternative as the contractor must wait for adequate strength development of the concrete slab before form stripping can begin. Also, with this type of wharf future expansion for rail is very difficult and costly.

Note that the pile spacing for this type of structure is controlled primarily by span limitations of the cast-in-place concrete deck slab under vertical loading. Therefore, pile spacing is selected independent of lateral resistance requirements of the substructure. Lateral resistance is incorporated into the substructure by selecting a pile size capable of resisting the lateral design loads.

Composite Panel Wharf ■ The third marginal open wharf design consists of a composite concrete deck surface supported by either a cast-in-place or precast pile cap which in turn is supported by a precast prestressed concrete pile. A schematic drawing of the composite panel alternative can be found in Fig. 23.6.

The main working surface of this design is a concrete composite deck composed of precast concrete panels and cast-in-place concrete topping slab. Precast reinforced or precast prestressed concrete deck panels would be placed upon masonite bearing pads, which in turn would be supported by precast or reinforced cast-in-place concrete pile caps. Each pile cap would in turn be supported by a square precast prestressed concrete pile. A cast-in-place concrete drop-panel fascia on the waterside face of the wharf accommodates the dock fender system and the mooring devices, as well as providing a transition area that would accommodate any required utility or service pits.

One of the main advantages of composite panel construction is the minimization of labor intensive concrete formwork over water, resulting in reduced field labor costs and generally a reduction in the construction time. Additionally, precast items theoretically afford a greater degree of quality control. Modular type construction is also inherently less time consuming than more labor intensive methods such as the cast-in-place concrete alternatives. Most of the precast elements for this alternative are of the size and configuration that would facilitate casting on site.

The disadvantages of the composite panel alternative include the relative discontinuity of adjacent structural members. This results in the need for tighter construction tolerances and special connection details. This alternative requires a large volume of material and special connection details to make up for the relatively flexible nature of this type of structure and to effectively transfer lateral loads from the deck elevation into the supporting pile system. Also, with this type of wharf future expansion for rail is very difficult and costly.

Cast-in-Place Slab with Bent Wharf ▪ The fourth, marginal open wharf design considered is a cast-in-place slab and bent system. This is a more conventional means of construction that utilizes pile supported cast-in-place concrete bent caps and deck slab. A schematic drawing of the cast-in-place slab with bent design can be found in Fig. 23.7.

This alternative consists of a cast-in-place concrete deck that provides one-way slab action to resist vertical and lateral loading. The slab is cast integrally with a series of pile supported concrete bent caps. A drop-panel fascia on the waterside face of the wharf accommodates the dock fender

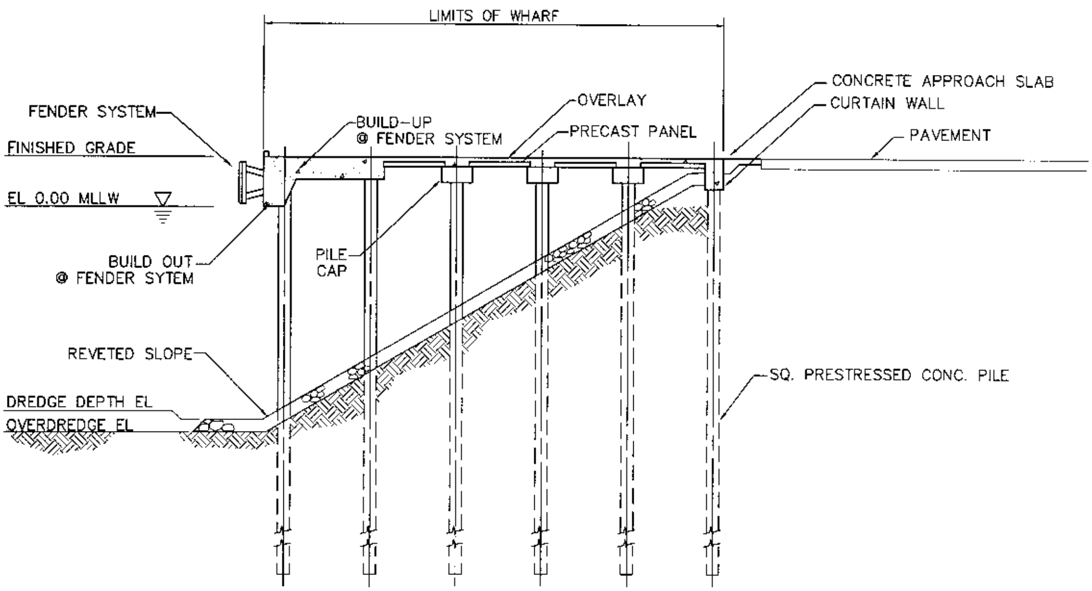

Fig. 23.6 Composite panel open wharf.

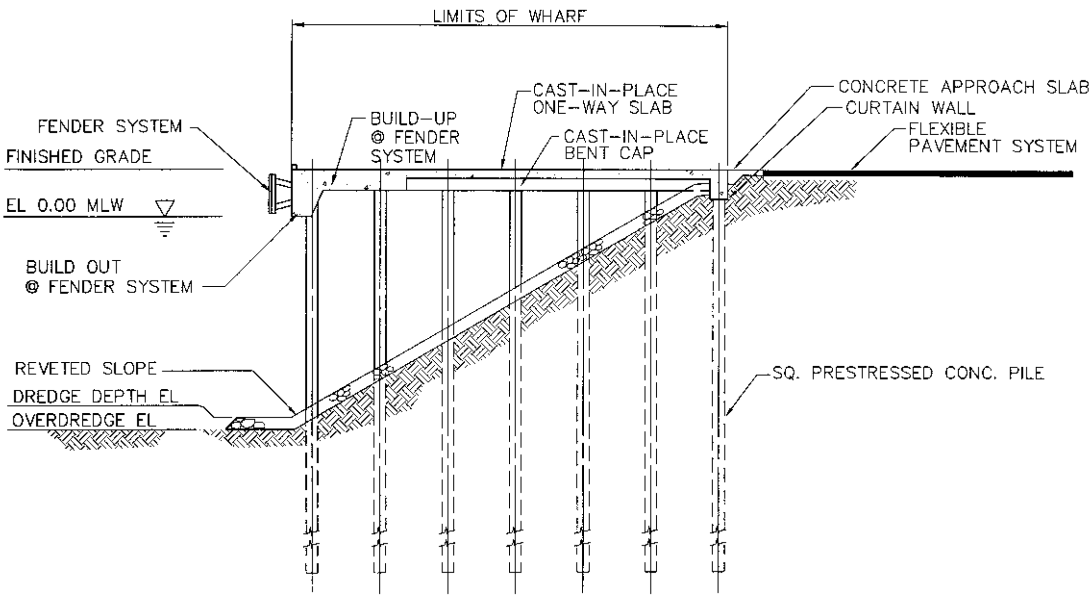

Fig. 23.7 Cast-in-place slab with bent.

system and the mooring devices, as well as providing a transition area that would accommodate any required utility or service pits. The transverse beams would be supported by prestressed concrete piles.

The advantage of this cast-in-place slab and bent configuration is that construction tolerances are generally not as critical as that of precast or modular systems. While cast-in-place construction is usually more time consuming and labor intensive than precast methods, it may be feasible alternative where there is an available supply of low cost labor. In addition, this type of construction can be completed by most general contractors and requires no special construction equipment or handling techniques.

The disadvantages of this type of construction principally involve the use of cast-in-place construction. This type of work is extremely labor intensive, so a dependable supply of low cost labor must be available. In addition, this type of construction requires more time in the field than a precast alternative as the contractor must wait for adequate strength development of the concrete slab before form stripping can begin. Finally, to minimize the time on the site and to account for shrinkage effects, a special concrete casting sequence must be used.

Ballasted, Cast-in-Place Flat Plate Wharf ▪ The fifth, marginal open wharf design considered is a ballasted open marginal wharf, which can be constructed with or without a toe wall. This structure is identical to the cast-in-place flat plate wharf described, but the deck is constructed at a lower elevation, then covered with a granular fill material, which can then be paved with asphalt. A schematic drawing of the ballasted flat plate alternative can be found in Fig. 23.8.

This consists of an 18-inch thick, cast-in-place concrete deck, which provides two-way slab action to resist vertical and lateral loading. An upset fascia system on the waterside face of the wharf retains the fill and accommodates the dock fender system and the mooring devices. Utility trenches can be constructed in the fill material between the deck and the asphalt.

The main advantages of the ballasted construction are the flexibility provided by asphalt paving and the lower deck elevation saves a row of piles. The advantages of the flat plate deck construction include minimization of forming costs for cast-in-place concrete construction and economy of materials. Because the structure is principally cast-in-place construction, less specialized equipment is required during construction. Most general contractors can perform the necessary finishing and

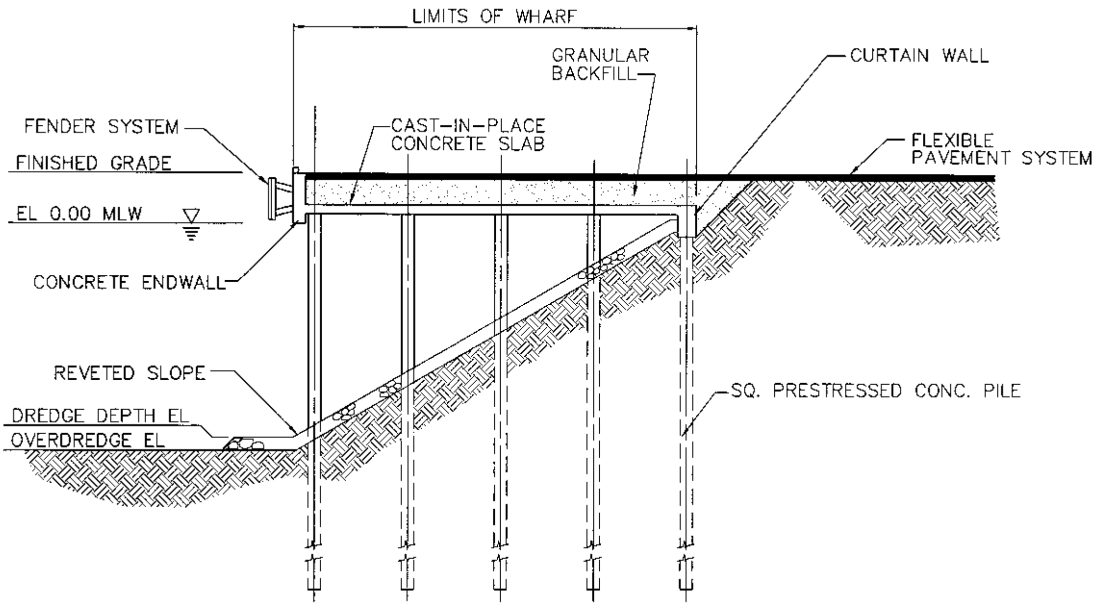

Fig. 23.8 Ballasted cast-in-place slab open wharf.

concrete placement required. Finally, since the structure consists only of cast-in-place concrete and precast piles, there are fewer structural components to integrate together, and thus, this alternative provides a high degree of structural continuity.

The disadvantages associated with this alternative are related to the extensive use of cast-in-place concrete. To minimize the time on the site and to account for shrinkage effects, a special concrete casting sequence must be used. A large pool of relatively skilled workmen will be required.

Note that the pile spacing for this alternative is controlled primarily by span limitations of the cast-in-place concrete deck slab under vertical loading. Lateral resistance is incorporated into the substructure by selecting a pile size capable of resisting the lateral design loads.

23.11.2 Solid-Type Marginal Wharf

The term solid-type marginal wharf refers to a wharf that has a continuous vertical face at or near the pierhead line. Such structures would include cellular cofferdams, either steel or concrete sheet piles walls, or some other type of structure that can contain backfill while providing a safety factor against sliding and overturning.

The six solid-type wharf alternatives considered here as follows:

- Cellular Cofferdam
- Open Cell Cofferdam
- Steel King Pile Bulkhead
- Steel Sheet Pile Bulkhead with Relieving Platform
- Precast Concrete Caisson
- Precast Concrete Gravity Wall

Each of these alternatives is best located so that the vertical load bearing elements of the structures are used. With slid-type marginal wharf structures, the width of the wharf is not constrained by geometry as with the open, pile-supported wharves where the minimum width is controlled by the reveted slope and cut-off wall. Solid-type wharves are only limited by the width of the cap.

A summary of advantages and disadvantages of the solid type marginal wharf alternatives are provided in Table 23.6. A description of each alternative follows thereafter.

Cellular Cofferdam ▪ A cellular cofferdam alternative can be considered a solid-type marginal

Table 23.6 Solid Type Marginal Wharf Alternatives

Cellular Cofferdam	Open Cell Cofferdam	Steel King Pile Bulkhead	Steel Pile with Relieving Platform	Precast Concrete Caisson	Precast Concrete Gravity Quaywall
ADVANTAGES					
Facilitates backfilling and vibro-compaction	Facilitates backfilling and vibro-compaction	Combines vertical and horizontal load bearing elements	Combines vertical and horizontal load bearing elements	Precast Elements Afford Better Quality Control	Precast Elements Afford Better Quality Control
Reduces width of wharf	Reduces width of wharf	Reduces initial dredging	Reduces initial dredging	Minimizes Field Labor	Combines Vertical and Horizontal Load Bearing Elements
Stable in seismic event	Stable in seismic event	Minimizes structure requirements—fewer driven piles	Minimizes structure requirements—fewer driven piles	Combines Vertical and Horizontal Load Bearing Elements	Reduces Width of Wharf
Reduced settlements	Reduced settlements			Reduced Settlements	Reduced Settlements
Constructed from land	Constructed from land				
DISADVANTAGES					
Sheet piling requires corrosion protection	Sheet piling requires corrosion protection	Highly dependent on quality of backfill material	Highly dependent on quality of backfill material	Special equipment required	Special equipment required
Requires complicated concrete closure pours for fascia	Requires complicated concrete closure pours for fascia	Requires geosynthetics for materials segregation	Requires geosynthetics for materials segregation	Requires use of dry-type casting yard	Requires use of dry-type casting yard
Increased time for construction	Dependent on quality of soils	Increased settlement potential		Complicated construction sequence	Requires geosynthetics for materials segregation
Highly dependent on quality of soils	Requires partial demolition of existing structure			Requires demolition prior to construction	Requires removal of existing structure prior to construction
Requires partial demolition of existing structure				Construction from barge	Requires construction from barge
				Large excavation and backfill quantities	Requires large quantities from excavation and backfill

Note: All solid-type wharves require less excavation and demolition than open-type wharves.

wharf containment structure. Although normally associated with providing a dry work area in waterfront construction, the inherent mass and stability of a cofferdam structure lends itself to fill containment for construction staging. Cofferdams may also be integrated with landside and waterside structures to provide a working platform for port operations.

Cellular cofferdams are generally constructed using straight web, steel sheet piling with controlled fill materials placed within the cells. Several geometric configurations of cellular cofferdams are available, including circular, diaphragm, and parallel wall configurations with open anchor cells. Circular cells are generally preferred for offshore construction and their circular shape favors single unit stability, east of forming templates for construction, and economy of material.

Cellular cofferdams depend on the interaction of the steel sheet piling and the material used to fill the cell to provide stability. Quality of the fill material has a direct effect on the size and, therefore, the expense of the sheet pile cofferdam. An ideal fill material would be a dense, granular, free-draining material with a high angle of internal friction and a low percentage of silt and clay materials. Fill materials that do not meet all of these requirements may be improved somewhat by vibratory compaction after placement within the cell. Unsuitable soils can be removed and replaced with suitable soils.

A schematic drawing of a cellular cofferdam alternative, intended for use as a solid-type marginal wharf, and combined with dock fendering devices is provided as Fig. 23.9.

A cellular cofferdam structure is generally most feasible when the underlying materials are dense gravel, rock, or other similar dense and hard materials. Founding the coffer structure in this type of material allows the structure to develop

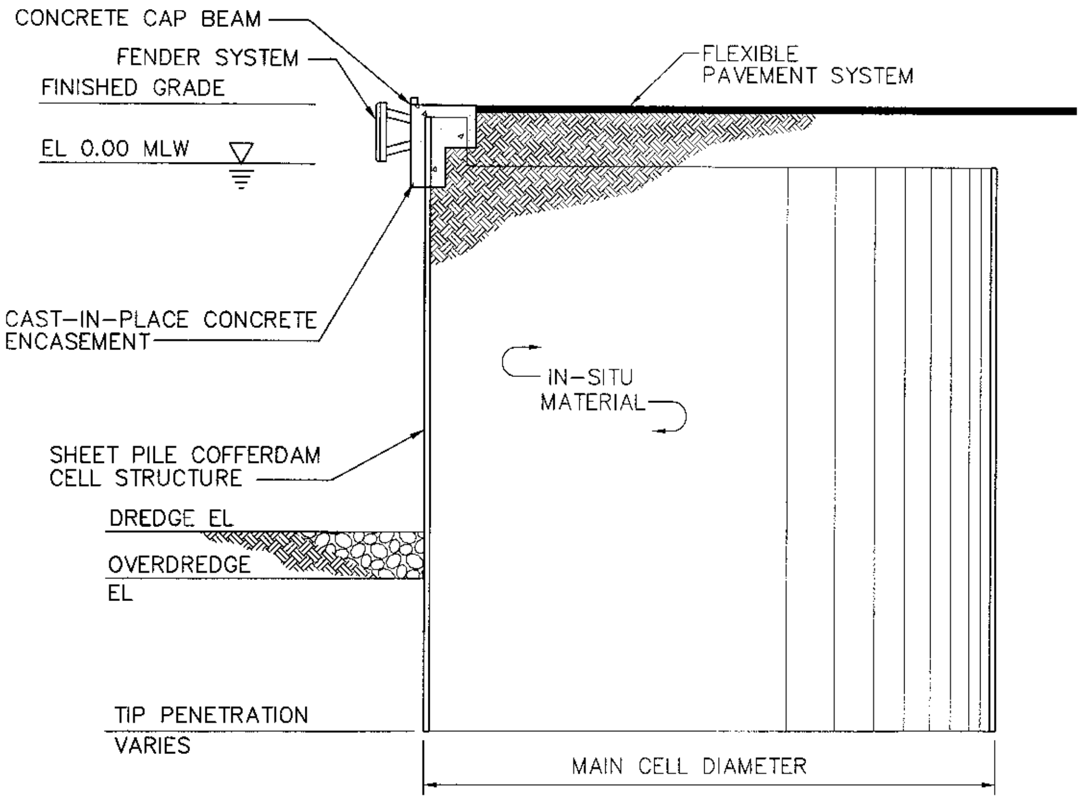

Fig. 23.9 Cellular cofferdam.

sufficient strength to resist the overturning and sliding forces that act on such a gravity structure. The drawback is that pre-drilling is required when shallow rock layers are encountered.

Transfer of the anticipated horizontal loads from mooring and berthing would be accomplished through special details which connect the facing structure to the cofferdam. These details could consist of a continuous reinforced concrete closure pour and additional horizontal restraining features such as longitudinal and transverse reinforced concrete shear keys.

Open Cell Cofferdam ▪ An open-cell cofferdam is a variation of the cellular cofferdam and presented as a solid type marginal wharf. For a sketch of the configuration of an open-cell cofferdam, see Fig. 23.10.

The tail walls anchor the system and extend landward from the curved segments of sheet pile wall. The driving tolerances and template requirements are not as tight as those for the cellular cofferdam structures. Embedment depths on this type of structure are shallower than for the other structures discussed, reducing material costs. The tail walls can be manipulated to avoid obstructions illustrating the exceptional tolerances inherent in the system.

Open cell cofferdams depend on the interaction of the steel sheet piling and the material used to fill the cell to provide stability. Quality of the fill material has a direct effect on the size and, therefore, the expense of the sheet pile cofferdam. An ideal fill material would be a dense, granular, free draining material with a high angle of internal friction and a low percentage of silt and clay materials. Fill materials that do not meet all of these requirements may be improved somewhat by vibratory compaction after placement within the cell. Unsuitable soils can be removed and replaced with suitable soils.

In evaluating an open cell cofferdam as a solid-type marginal wharf structure, consideration should be given to construction sequencing, and vertical and horizontal load carrying capability of the structure.

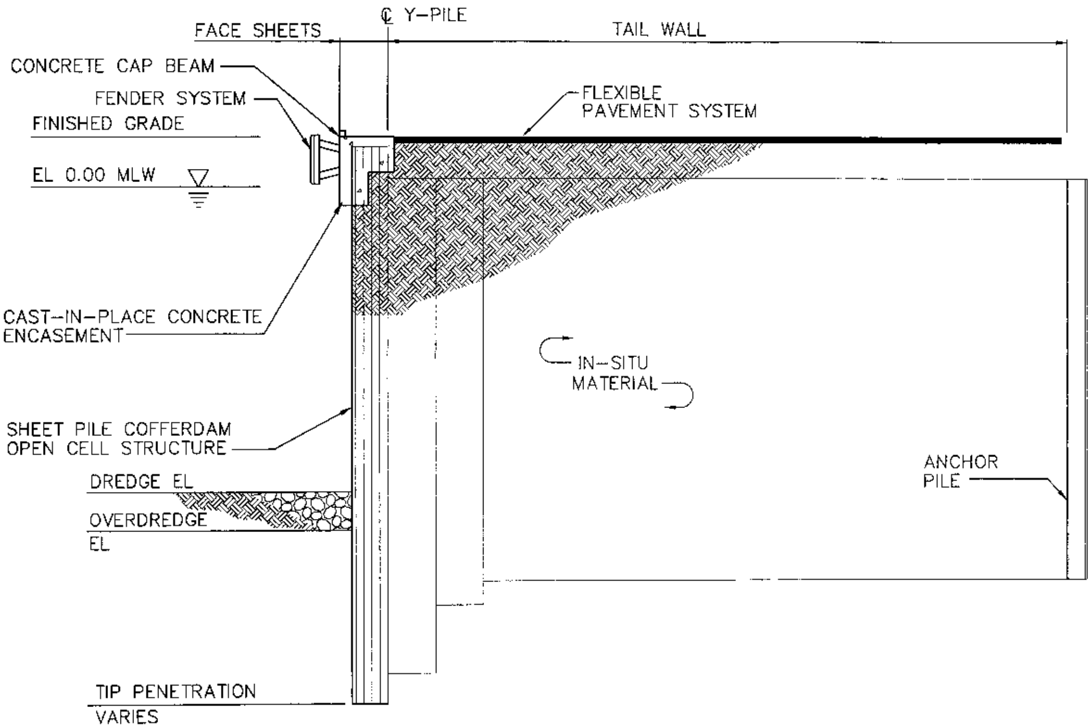

Fig. 23.10 Open cell cofferdam.

An open cell cofferdam structure is generally most feasible when the underlying materials are dense gravel, rock, or other similar dense and hard materials. Founding the coffer structure on this type of material allows the structure to develop sufficient strength to resist the overturning and sliding forces, which act on such a gravity structure. The drawback is that pre-drilling is required when shallow rock is encountered, which drives up the cost of construction.

Transfer of the anticipated horizontal loads from mooring and berthing would be accomplished through special details that connect the facing structure to the cofferdam. These details could consist of a continuous, reinforced concrete closure pour and additional horizontal restraining features such as longitudinal and transverse reinforced concrete shear keys.

Steel King Pile Bulkhead ▪ A steel king pile bulkhead is another form of a solid-type marginal wharf. For this type of structure, the bulkhead face is located as close as practical to the pierhead line. For a sketch of a steel king pile bulkhead, see Fig. 23.11.

The sheet pile bulkhead can consist of a high-modulus, H-piles interlocked with "Z" sheeting. This will develop a significant vertical capacity, accomplished by driving the H-piles into any underlying hardrock layer. The "Z" sheeting can be driven to considerably less depth than the vertical load bearing elements (H-piles). This form of sheet piling is widely used, especially in Europe, with a great deal of success. Its main drawback is the need to provide an effective corrosion protection system for the steel. With proper corrosion protection measures, the steel sheet piling will provide a service life that meets or exceeds the service design life requirements of the port facility. Depending on the depth of intermediate sheet piles, this configuration can be susceptible to scour. In order to provide lateral capacity to resist the loads generated from retained fills and moored ships, the use of a conventional tieback system of tie rods and deadmen is typically incorporated.

Sheet Pile Bulkhead with Relieving Platform ▪ A sheet pile bulkhead with relieving platform is another solid-type marginal wharf containment structure. In order to reduce the size of the bulkhead structure required, a relieving platform would be incorporated into the system. The relieving platform transfers the vertical live loads

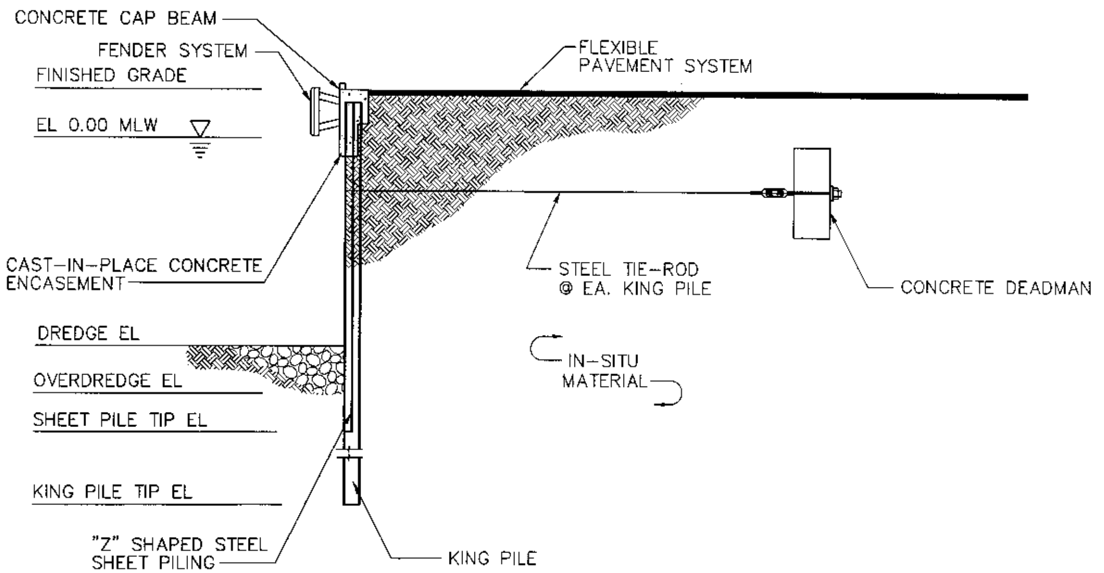

Fig. 23.11 King pile bulkhead with tie-rod and deadman.

from the deck directly into the underlying soils. Thus, the bulkhead is not required to resist the lateral surcharge, which would be generated from the live loads. To minimize the material requirements, the bulkhead has been developed to support some of the vertical loads from the wharf deck to supplement to relieving platform load bearing elements. A schematic of this is shown in Fig. 23.12. This design uses "Z" configuration steel sheet piles.

The relieving platform portion of this design is a cast-in-place concrete structure supported on prestressed concrete piles. The bottom of the relieving platform deck would be constructed at or near the mean high water elevation. At the outboard face of the relieving platform a gravity-type retaining wall would serve to contain the pavement supporting fill material.

In order to provide lateral capacity to resist the loads generated from retained fills and moored ships, it would be possible to incorporate a series of battered piles into the relieving platform structure. Another alternative for lateral restraint involves the use of a conventional tieback system of tie rods and deadmen.

Precast Concrete Caisson ▪ The most common of the available configurations of precast concrete caissons is that of a multicell box-type design. Structures of this type are usually constructed in drydocks, cofferdams, or other similar dry-type basins. Upon completion, the caissons are then floated into position, where they are flooded and lowered onto a prepared leveling course. The construction sequence is completed by the filling of the cells with a suitable ballast material. The inherent mass and stability of a large caisson structure lends itself to fill containment during construction staging. Caissons may also be integrated with landside and waterside structures to provide a working platform for port operations. For a sketch of a proposed configuration of a precast concrete caisson, see Fig. 23.13.

One of the most important aspects in the utilization of a caisson structure is that of preparation of a proper foundation course. It is also important to design into the structure tolerances that allow for differential settlement between adjacent caisson units. Joint design also merits careful consideration so that the structure is soil-tight.

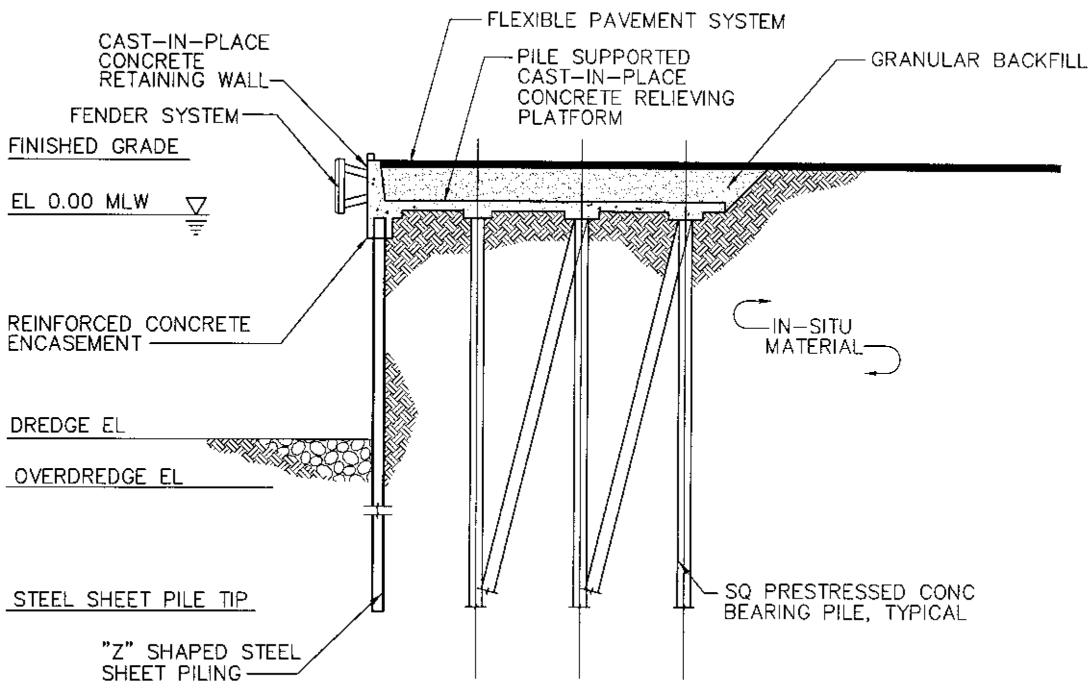

Fig. 23.12 Sheet pile bulkhead with relieving platform.

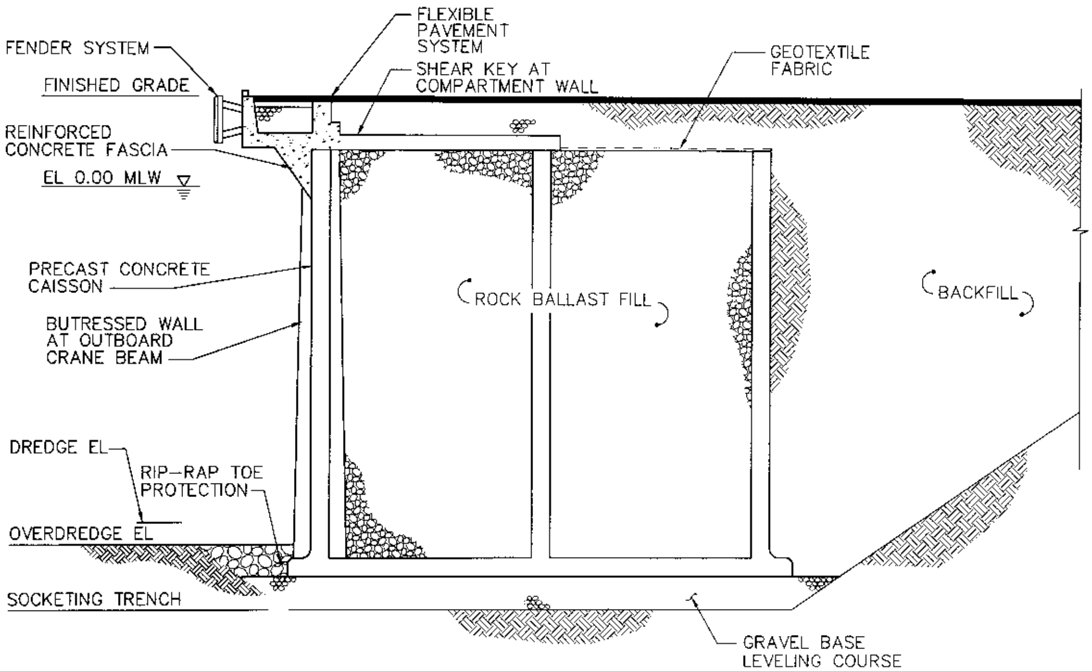

Fig. 23.13 Precast concrete caisson.

The anticipated sequence of construction for a precast concrete caisson solid-type wharf structure is as follows:

- Perform the initial dredging of existing river silt material in the entire area of the caisson structure. Dredge a keyway within the limits of the caisson.
- Construct the compartmentalized precast concrete caisson. The length of caisson will be dependent on the configuration of the proposed casting facility.
- Provide a gravel base leveling course in the socketing trench and graded to suit the dimensions of caisson selected.
- Transfer the caisson unit(s) from the casting yard to the site using tugs. Maneuver the precast concrete caisson into place, and begin lowering the unit by filling with water.
- Align and adjust the structure while it is buoyant. When alignment is achieved, begin placement of the rock ballast fill. Adjust the leveling and placing process by controlled placement of ballast.
- Provide a base protection material of graded riprap. At this point, the terminal side fill material may be placed.

In order to accomplish the transfer of the anticipated horizontal loads from mooring and berthing, special details would need to be developed to connect the facing structure to the caisson. These details could consist of a continuous reinforced concrete closure pour and additional horizontal restraining features such as reinforced concrete shear keys.

Precast Concrete Gravity Quaywall ▪ A precast concrete gravity quaywall design is presented as a solid-type marginal wharf structure. Components of these structures are usually cast in work yards and then transported to the site on barges, where they are lowered onto a prepared leveling course. The inherent mass and stability of a large gravity wall structure lends itself to fill containment during construction staging. In order to achieve a properly interlocked structure, the separate concrete elements are usually match-cast and marked before transport to the field for placement. For a sketch of a configuration of a precast concrete gravity quaywall, see Fig. 23.14.

One of the most important aspects in the utilization of a precast concrete gravity structure is that of providing a properly prepared foundation course. It is also important to design into the structure tolerances that allow for differential settlement between adjacent structure units. Joint design and soil key interlock design also merit careful consideration so that the structure is stable and soil-tight.

The anticipated sequence of construction for a precast concrete gravity containment structure would be as follows:

- Perform the initial dredging of existing materials in the entire area of the wall structure. Dredge a keyway within the limits of the gravity wall.
- Construct the precast concrete wall sections. The size of the blocks will be dependent on the capabilities of materials handling equipment, cost for transport, and other similar factors.
- Provide a gravel base leveling course in a socketed trench graded to suit the dimensions of the precast concrete gravity wall.
- Transport the concrete blocks to the site, and lower the blocks onto the prepared bed. Backfill the area behind the wall with select materials so that the resultant active earth pressures on the wall are minimized.
- Provide a base protection material of graded rip-rap.

Transfer of the anticipated horizontal loads from the wharf structure to the gravity quaywall would be required. This load transfer would be accomplished through the use of a continuous concrete cap block that links the wharf to the wall. Additional horizontal restraining features may also include transverse reinforced concrete shear keys. An additional advantage of this type of system is that the wall structure may serve as a vertical load bearing element for a waterside crane beam, thereby reducing some of the structure costs associated with the crane beam support.

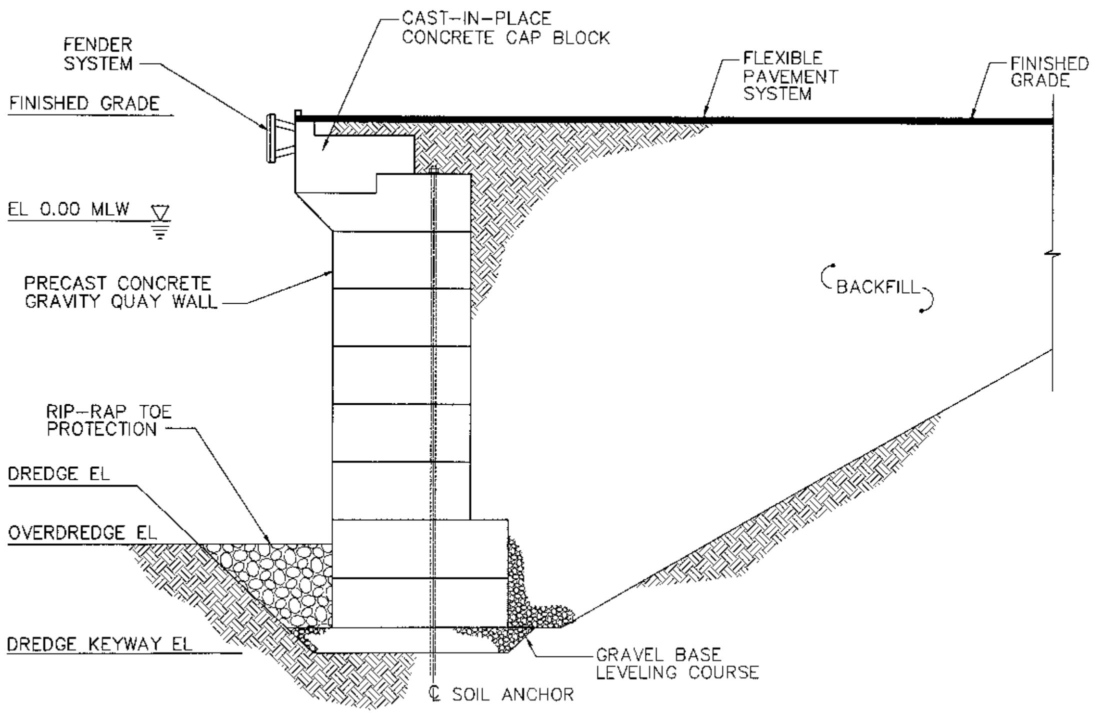

Fig. 23.14 Precast concrete gravity quaywall.

23.11.3 Loads on Docks

When the type of dock and its general construction features have been determined, it is necessary to establish the lateral and vertical loads for which the dock is to be designed. These consist of the following:

Wind Forces ▪ Mooring lines, which pull the ship into or along the dock or hold it against the force of the wind or current, exert lateral forces on a dock. The maximum wind force equals the exposed area, ft^2, of the broadside of the ship in a light condition, multiplied by the wind pressure, psf, to which a shape factor 1.3 is applied. This is a combined factor that takes into consideration a reduction due to height and an increase for suction on the leeward side of the ship. The wind force varies with the location and local building codes.

Current Forces ▪ The force of the current, psf, equals $wv^2/2g$, where w is weight, lb/ft^3, of water, v is the velocity of the current, ft/s, and g is 32.2 ft/s^2. For salt water this results in a pressure, psf, equal to v^2. The velocity of current usually varies between 1 and 4 ft/s, which results in pressures of 1 to 16 psf, respectively. Current pressure is applied to the area of a ship below the water line when the ship is fully loaded. Since ships are generally berthed parallel to the current, this force is seldom a controlling factor in design of the structure. However, currents are important in fender system design.

Impact ▪ Docking impact is caused by a ship striking the dock when berthing. The assumption is usually made that the maximum impact to be considered is that produced by a ship fully loaded (displacement tonnage) striking the dock at an angle of 10° with the face of the dock, with a velocity normal to the dock of 0.25 to 0.5 ft/s (Fig. 23.15). A few installations have been designed for as much as 1.0 ft/s, but this may be excessive; it corresponds to a velocity of approach of about 3½ knots at an angle of 10° to the face of the dock, and such an impact could damage a ship.

Fender systems are designed to absorb the docking energy of impact. The resulting force to be resisted by a dock depends on the type and construction of the fender and the deflection of the dock if it is designed as a flexible structure.

Earthquake Forces ▪ These have to be considered if a dock is in an area where seismic disturbances may occur. The horizontal force, applied at the dock's center of gravity, may vary between 0.025 and 0.15 of the acceleration of gravity g times the mass. The force also can be expressed as 0.025 to 0.15 of the weight, respectively. The weight to be used is the total dead load plus one-half the live load. Unless the dock is of massive or gravity-type construction, seismic effect on the design will usually be small since the allowable stress, when combined with dead- and live-load stresses, may be increased by 33⅓%.

Gravity Loads ▪ These consist of the dead weight of the structure, or dead load, and the live load, which usually consists of a uniform load and

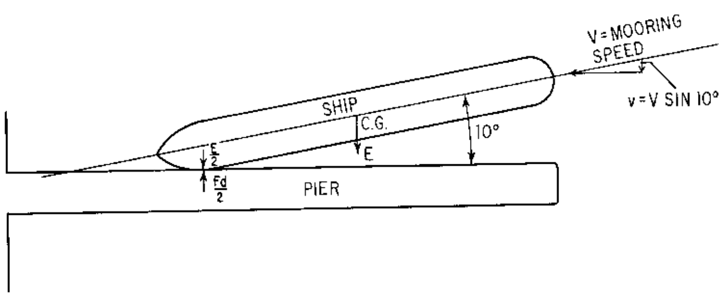

Fig. 23.15 Ship is assumed to strike pier at 10° angle for design against impact.

wheel loads from trucks, railroad cars or locomotives, cargo-handling cranes, and equipment. The uniform live load may vary from 250 to 1000 psf on the deck area. The smaller figure is used for oil docks and similar structures that handle bulk materials by conveyor or pipeline and where general cargo is of secondary importance. General-cargo piers usually are designed for heavier live loads, ranging from 600 to 800 psf. Piers handling heavy metals, such as copper ingots, may be designed for 1000 psf or more. The uniform live load controls design of the piles and pile caps, whereas the concentrated wheel loads, including impact, usually control the design of the deck slab and beams. A reduction of $33\frac{1}{3}\%$ is sometimes made in the uniform live load in figuring the pile loads and designing the pile caps or girders, based on the assumption that the entire deck area of adjoining bays will not be fully loaded at one time.

23.11.4 Dock Fenders

Fenders act as the interface between the vessel and structure. They serve to protect both the structure and the vessel from damage while the vessel is moored. There are several categories of commercially available fenders including buckling fenders with low friction panels, foam filled and pneumatic floating fenders, cylindrical side loaded fenders, and molded arch type fenders with low friction panels. Low friction panels are coated with an ultra high molecular weight polyethylene (UHMW-PE).

Buckling fenders include cell fenders and Π-shaped fenders. A small length to height ratio is preferred for Π-shaped fenders. The panels distribute the reaction force over a large area thereby exerting a low hull pressure while absorbing a high quantity of energy. These fender systems are often used for cruise ships and container vessels. They are moderately priced.

Foam filled and pneumatic floating fenders are most widely used for ship-to-ship operations and in areas where tidal variations are extreme. They are easily moved, absorb a high quantity of energy, and exert very low hull pressures. Due to their buoyancy, they move vertically with the vessel when the water level changes and allow berthing of several types of surface vessels. The fender requires a backing system to distribute the load.

Cylindrical side loaded fenders are used predominately for berthing small ships and pleasure craft. They are usually constructed of rubber and attached to the structure with chains. Cylindrical fenders absorb less energy than other fender systems and are often not used alone. They exert a moderately high hull pressure on the vessel and do not allow the ship to slide as well as low friction (UHMW-PE) panels. They are moderately priced.

Molded arch type fenders with low friction panels are similar to buckling rubber element fenders, but they are smaller and do not absorb as much energy due to their increased length to height ratio. The rubber fender is attached to the berthing structure and steel panels topped with UHMW-PE provide the contact surface for the vessels.

References for the design and selection of fender systems are the Military Handbook "Piers and Wharves" MIL-HDBK-1025/1 and PIANC 1984 Report of the International Commission for Improving the Design of Fender Systems.

23.11.5 Dolphins

Dolphins are designed principally for horizontal loads of impact, wind, and current forces from a ship when docking or moored. These forces are determined in the same manner as for design of docks.

Dolphins may be of the flexible or rigid type. Wood-pile clusters are examples of the former type. These are driven in clusters of 3, 7, 19, etc., piles, which are wrapped with galvanized cable (Fig. 23.16). The center pile of each cluster is usually permitted to extend about 3 ft above the other piles to provide a means of attaching a ship's mooring lines. A modification of this type of dolphin arranges the piles symmetrically and on a slight batter. They are bolted to wood cross members located just above low-water level, with wood framing at the top. Large steel cylinders and groups of steel-pipe piles have also been used to provide flexible dolphins.

In general, dolphins of the flexible type have been used for mooring small vessels, not exceeding 5000 DWT (deadweight tonnage), as an outer defense for protection of docks, or for breasting off somewhat larger vessels from loading platforms and structures not designed to take the impact of ships. Bottom soil conditions must be suitable for a flexible-type installation. If the soil is too soft, the dolphins or pile clusters will not rebound to their original positions after being struck by a vessel,

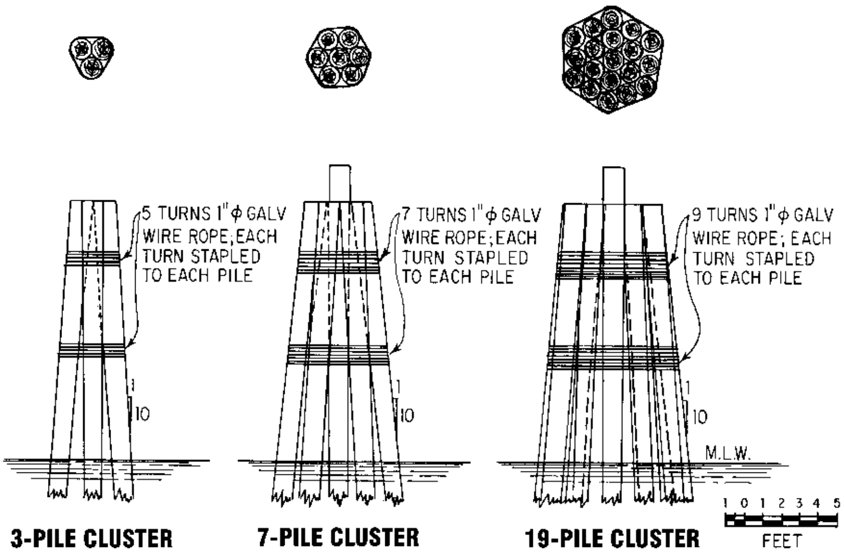

Fig. 23.16 Typical wood-pile dolphins in clusters of 3, 7, and 19 piles.

and their energy-absorbing ability, which depends on deflection, will be gradually dissipated.

For larger cargo ships and tankers of 9000 to 17,000 DWT, a wood-platform type of rigid dolphin, utilizing wood batter piles, may be used for mooring and breasting. Since the wood platform is relatively lightweight, its lateral stability depends to a large extent on the pullout value of the wood piles. In general, a lateral force of about 40 to 50 tons is about the most that a dolphin of this type can resist without becoming too large and unwieldy.

If bottom soil conditions are suitable, sheetpile cells make excellent dolphins. They can be designed to withstand the forces from the largest ships, if provided with adequate fenders. Cells, because of their circular shape, are well suited for **turning dolphins**, for warping or turning a ship around at the end of a dock. Cellular dolphins are usually capped with a heavy concrete slab, to which the mooring post or bollard is anchored. When large ships are to be handled, a powered capstan should be provided to draw in the heavy wire-rope mooring lines.

For big ships, dolphins may be designed with heavy concrete platform slabs supported by vertical and batter piles, usually of steel or precast concrete. This type of dolphin with low-reaction-force, high-energy-absorption rubber fenders can take the docking and mooring forces from the largest supertankers. For this purpose, a large number of batter piles are required. The uplift from these piles, in turn, makes it necessary to have a considerable amount of deadweight since the vertical piles will in general resist only a relatively small portion of the uplift. This deadweight is supplied by the concrete slab, which may be 5 to 6 ft thick. A sufficient number of vertical piles must be provided to support this deadweight. In addition, the vertical piles must not be spaced too far apart; otherwise, it will be difficult and expensive to provide forms for the concrete. When the depth of slab exceeds 4 ft 6 in, it is usually economical to cast the slab in two lifts, with horizontal shear keys at the construction joint. This greatly reduces the cost of the forms.

23.12 Marina Layout and Design

The layout of marinas varies according to local markets, customs, and site geometry. Marinas provide berthing space in calm water, access to utilities for boats, parking, fueling and sometimes repair service, boat ramps, stores, launch hoists, and storage sheds or areas. Marinas should be designed to meet the needs of the physically impaired as required by the Americans with Disabilities Act.

Selection of slip length is based on economics and the local boating community's typical lengths. Fairway aisles widths are 1.5 to 2 times the slip length (Fig. 23.17), depending on whether or not over-length boats are allowed in the slips. The width of single slips is about 4 ft wider than the boat beam or about half the slip length for slips less than 30 ft long. Piers can be either fixed or floating with ramps down from the surrounding grade level. Fixed-elevation piers are typically used in areas with a tide range of less than 3 ft. The width of piers gives at least 5 or 6 ft of clearance (inside of boat and utility boxes). Finger-pier widths are less. Wider piers are used for long piers that have more traffic.

Channel-depth requirements are often dictated by local physical conditions and boat fleet requirements. Dredging of marinas to provide adequate depth is a planning challenge because of the permit requirements related to the dredging and disposal of dredged material. (See J. B. Herbich, "Handbook of Coastal and Ocean Engineering," Gulf Publishing Company, Houston, Tex (www.gulfpub.com).)

23.12.1 Design Considerations for Megayachts

Recreational vessels typically 80 feet in length or greater are referred to as megayachts. Requirements for berthing facilities for these larger vessels differ from the requirements for berthing the smaller recreational boats in the 25 to 50 foot length.

Dock systems can be either fixed or floating, but require higher freeboard than normal to allow access from the vessel. The dock width should also be wider since most megayachts have their own gangway or steps that extend onto the dock. The fendering system along the dock to protect the expensive finishes on the vessels is more substantial than a typical marina dock. The wind loads exerted on the dock from a megayacht are also higher.

Utility requirements for megayachts are substantial. Electrical requirements may range from single phase, 50-amp to three phase, 400-amp. The

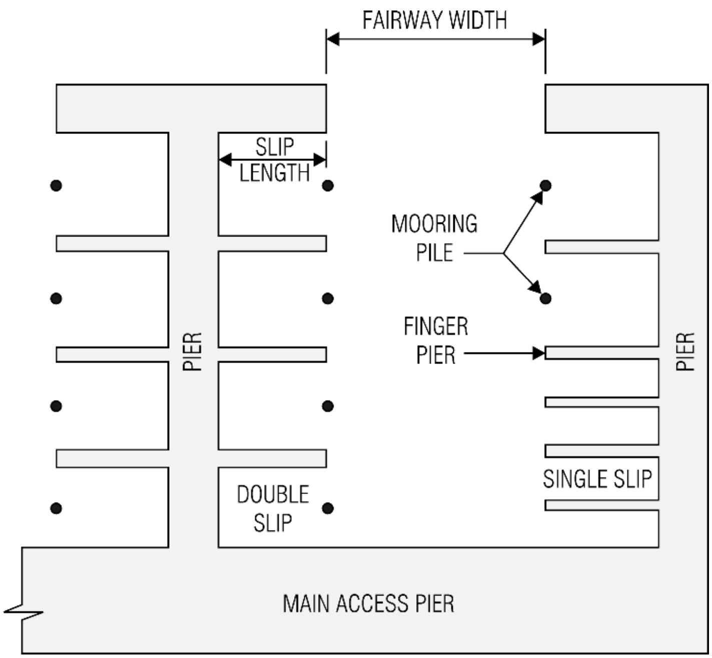

Fig. 23.17 Layout of marina with single and double slips.

larger water holding tanks on the yachts may require 1- to 3-inch water mains to fill the tanks within a reasonable time.

23.12.2 Breakwaters for Marinas

Some form of breakwater may be required to reduce wave heights to an acceptable level, usually less than 1 ft. (Although they may be necessary, breakwaters usually do not directly produce income for the marina.) Breakwater layout should provide adequate protection as well as allow safe movement into and out of the marina. The width of the entrance channel is often dictated by the local boat fleet. Enough room should be provided for safe passage in two-way traffic. This leads to use of typical entrance widths of 75 to 125 ft of navigable depth. Alternative breakwater types include rubble-mound, caissons, floating breakwaters, vertical walls, or wave fences. (See R. R. Bottin et al., "Maryland Guidebook for Marina Owners and Operators on Alternatives Available for the Protection of Small Craft against Vessel Generated Waves," U.S. Army Engineers Coastal Engineering Research Center, Washington, D.C.)

Wave protection of a breakwater is expressed in terms of the wave transmission coefficient k_t:

$$k_t = \frac{H_t}{H_i} \tag{23.6}$$

where H_i = incident wave height

H_t = transmitted wave height in the lee of the breakwater

Transmission through breakwaters is caused by water moving through the interstices of the rocks and by splash over the top. The wave energy that rounds the edges of breakwaters is diffraction energy. The wave heights in the lee of a structure will be the result of a combination of transmission and diffraction. The "Shore Protection Manual," U.S. Army Coastal Engineering Research Center, Government Printing Office, Washington, D.C., provides design guidance for rubble-mound structures used primarily for protection against large waves.

Floating breakwaters are an attractive alternative for use in deep water and sites with short fetches for wind that allow only short-period seas. Transmission coefficients k_t for floating breakwaters depend on the ratio of the width of the float to the incident wavelength; the ratio of the depth of penetration below water level to the water depth; and the stiffness of the mooring system. For relatively deep water, k_t ranges from 0.2 to 0.4 when the ratio of float width to wavelength is greater than 0.5. For example, for a float width of 15 ft, this ratio gives a wavelength $L = 30$ ft. This L corresponds to a wave period T of about 2.4 s [Eq. (23.2)]. For a windspeed of 40 knots, this corresponds to a fetch distance of about 1 to 2 miles (Table 23.3). Thus, for fetches of about 1.5 miles and design wind conditions of 40 knots, the incident wave height of 2.2 ft would be reduced to less than 1 ft by a 15-ft-wide floating breakwater. Transmission coefficients can be much higher for narrower widths or longer waves. (See J. W. Gaythwaite, "Design of Marine Facilities for the Berthing, Mooring, and Repair of Vessels," Van Nostrand Reinhold, New York, and "Planning and Design Guidelines for Small-Craft Harbors," Manual 50, American Society of Civil Engineers.

Transmission at complete, impermeable, vertical walls is zero. However, diffraction and wave reflection allow energy to penetrate breakwater gaps into lee areas.

Wave fencelike structures are built to allow some water flow through them. These structures also are most effective with short-period wind-generated seas from limited fetches. For significant reduction of wave energy, the gap spacing between fence boards should be such that the total gap area is less than 10% of the total area. These small gaps, however, may close because of biofouling and defeat the original purpose.

23.12.3 Design of Docks for Marinas

Dock design includes design of the dock structure and the related, desired utilities, such as electrical, water (domestic and fire protection), telephone, cable television, and sanitary sewer hookups or pumpouts.

Fixed piers may be constructed of timber, steel, concrete, aluminum, or plastic. Manufactured floating dockage systems are available. Flotation is provided by either an airtight compartment, foam, or wood. Reliable anchorage is essential; many failures of floating dockage systems have been attributed to inadequate anchorage.

Horizontal live loads are the sum of current and wave loads and wind loads due to the exposed cross section of the dock and boats.

Wood decking on piers is often 2 × 6-in pressure-treated lumber with galvanized screws. Bolting of cleats to the structural frame of the deck is recommended (ASCE Manual 50 (www.asce.org).)

23.12.4 Boat Ramps

Boat ramps are an inclined paved surface that extends into the water that allows trailerable boats to be launched and retrieved. The top elevation of the ramp should be a least 2 feet above the highest expected water level the ramp is designed to operate and at least 4 feet below the lowest expected water level that the ramp is designed to operate. The number of lanes is dependent upon the use and demand. The lane width should be 12 feet minimum; 15 feet is preferred. Studies have indicated that a single lane can handle up to 50 launches and 50 retrievals per day under average conditions. The grade of the ramp should be uniform and between 12.5% and 15%. A smooth transition with a vertical curve should be made between the top of the ramp and the approach to the ramp.

Ramps are typically constructed of reinforced concrete or precast reinforced concrete slabs placed over an aggregate base. It is important to provide a non-skid surface to allow for maximum traction for vehicles launching and retrieving boats. Concrete ramps should be finished with a V-groove surface with the grooves placed at an angle of 60 degrees from the axis of the ramp.

Boarding docks, fixed or floating, are provided for access to the boats after launching and before retrieving.

(B. O. Tobiasson and R. C. Kollmeyer, "Marinas and Small-Craft Harbors," Van Nostrand Reinhold, New York; "Planning and Design Guide for Small-Craft Harbors," ASCE Manual 50 (www.asce.org).)

23.12.5 Ice in Marinas

Ice is a significant marina engineering concern, including horizontal and vertical ice loadings on structures and boats. The most common adverse effect is the vertical jacking of piles as ice grips them and then rises with the tide. The severity of the condition depends on site location and seasonal use of the marina. (See C. A. Wortley, "Ice Engineering Manual for Design of Small-Craft Harbors and Structures," University of Wisconsin Sea Grant Institute; ASCE Manual 50 (www.asce.org).)

23.12.6 Marina Flushing Analysis

Marina facilities may pose a threat to the health of aquatic systems if they are poorly planned or managed. The water in a marina must meet applicable water quality standards. Thus, water quality considerations are a part of the planning and permit application process for marina construction and modification. Pollutants at marinas include boat grease, oil, and gas, head discharges, bilge discharges, and land runoff. Runoff can introduce to the marine water typical nonpoint-source runoff pollutants, such as automobile oil and grease from parking lots and fertilizers, pesticides, and herbicides from adjacent grass areas. Proper runoff management practices can be critical to protection of marina water quality.

A simple tidal-prism analysis tool for estimating the flushing of a proposed marina is described in "Coastal Marinas Assessment Handbook," Environmental Protection Agency (EPA). The fundamental concept is the same as that for estuary or bay flushing: A pollutant concentration in a marina will decrease with each tidal dilution but will never completely flush. A typical analysis goal is to estimate the time needed to reduce the concentration of a pollutant to 10% of its initial concentration. The tidal-prism analysis can be extended to a dissolved oxygen or nutrient balance to address the question of whether or not a significant reduction will occur in dissolved oxygen or nutrient level over a tidal cycle.

The EPA methodology assumes that tidal exchange is the dominant flushing mechanism. It can be used with fresh-water inflow. It does not, however, specifically account for other water exchanges, such as wind- and current-generated circulation. A more limiting assumption is that the water is well mixed inside the marina.

There are several commercially available numerical modeling software systems capable of marina flushing analysis. Examples of such programs are the RMA2/RMA4 programs in the Surface-Water Modeling Systems (SMS) created by Brigham Young University or the Water Quality or Spill Analysis modules in MIKE21 developed by DHI. Typical input parameter for numerical models include bathymetry and hydrodynamic data, boundary conditions and extents, pollutant concentrations and type, and simulation length. There are also many input variables available for each model to customize simulations to specific criteria.

23.13 Beach Nourishment

Also called beachfill, this refers to mechanically or hydraulically placed sand on a beach. The arguments for beach nourishment are threefold:

1. Enhanced recreation and increased property value. A sandy beach is a desirable recreation area. This is a very important consideration for developments along many coastlines and is probably the primary reason for most beachfills. The **strand**, the sand (and maybe also the dunes) between the water and hard structures such as houses, has economic and aesthetic value, although it may not be easily documented.
2. Increased property protection. Beach nourishment can provide a volume of sand that protects properties behind the beach from the sea. This protection may be in the form of long, high sand dunes that are designed to erode somewhat during storms without breaching or low, wide beaches, which reduce wave heights, or both.
3. Beneficial disposal of dredged material. Beachfills can serve as disposal sites for sediment dredged from nearby coastal construction projects. Sometimes, the purpose of the placement of dredged material on adjacent beaches is to continue the natural littoral drift or *river of sand* by keeping dredged inlet sands in the littoral system. Sometimes, the beachfill is for convenience in obtaining construction permits or to avoid more expensive disposal options.

With respect to the first two contentions, beach nourishment is justified if the annualized economic benefits exceed the annualized cost of the beachfill. Note that beach nourishment should be perceived as an ongoing or maintenance requirement. **Periodic renourishment** is a requirement of many nourishment projects.

Many very large beachfills (several million cubic yards each) have been placed on open ocean beaches as well as in more sheltered locations, such as bay shorelines. The purposes of some of these beachfills have been wetlands habitat protection and creation as well as recreation and property protection.

Beachfill Behavior ▪ One of the fundamental design criteria for beachfills is estimated project life. There is not at present a consensus on what this value should be.

One of the best ways to estimate the behavior of a planned beachfill is to evaluate the behavior of the last beachfill at that site. This, however, is not always possible, but it is justification for implementation of a monitoring system. If a beach is now being nourished, it probably will need nourishing in the near future. If a beachfill was previously placed at that site, the result with more beachfill, however, will not be the same, even if the beachfill design is exactly the same as before. Fill response depends on time variations in waves and longshore transport. Alterations in the volume, length, width, configuration, or size of the sand will also change the backfill behavior. However, qualitative and quantitative estimates based on previous fill behavior are valuable if the designs are similar enough.

The behavior of beachfills follows some general patterns. These can be viewed as the constructed beach moving into dynamic equilibrium with the wave and water-level climate. The volume of sand per unit length of beach is the sum of the following:

1. Volume of sand required to create an equilibrated beach of the desired width (profile equilibration)
2. Volume expected to move out of the project limits in the longshore direction (planform equilibration) during the design life

This sum should be multiplied by an overfill factor for winnowing due to source-native sand-size difference (size equilibration).

Cross-shore or profile equilibration is the most rapid adjustment of a beachfill. Beaches change shape constantly in response to changes in incident waves and water levels. Natural beach slopes are mild, from 1:10 to 1:100. Often, the constructed beach has a much steeper beach face than that which occurs naturally (Fig. 23.18).

Sand is pulled offshore from the dry beach onto the portions of the beach that are below the water surface. Since the dry beach area rapidly decreases, this change is often incorrectly perceived as a loss of the beachfill. Although the public may be unimpressed by the unseen, underwater portions of a beachfill, they are no less important than the

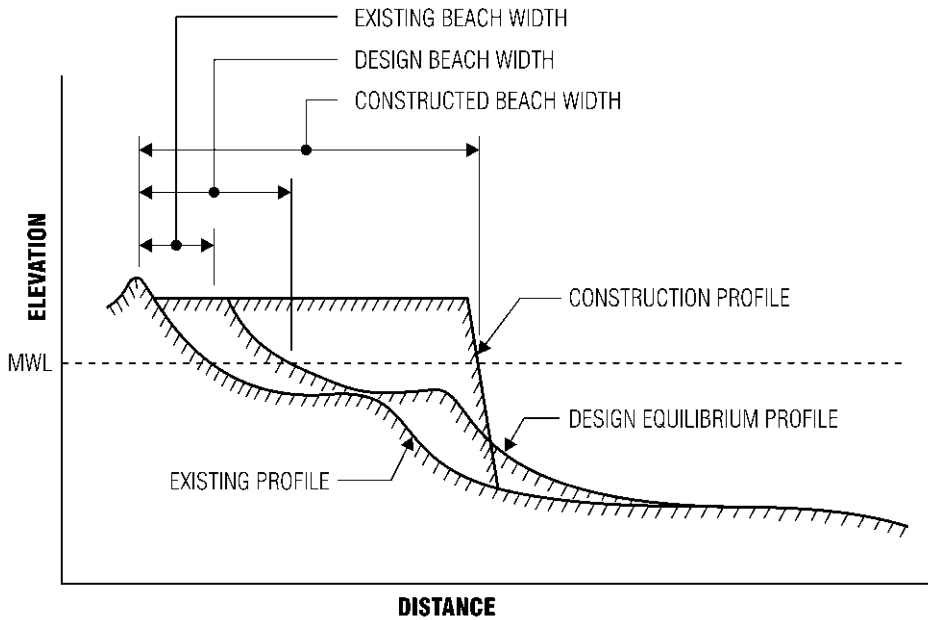

Fig. 23.18 Vertical cross section through a beach showing the existing beach profile, beachfill construction profile, and the effects of the cross-shore equilibration process.

underground foundation of a building. The full volume of sand required to widen the dry beach some distance must account for the underwater portion of the profile. Given that the dry beach is often a primary reason for benefits, however, there does not seem to be much reason for attempting to construct a more natural, milder beach slope. This would be difficult anyway. Public education, including construction-site signs and public meetings, are often used to inform the public that the initially constructed beach width is much greater than the intended design beach width.

A common technique for estimating the required volume of sand to build a desired beach width is based on the assumption that the post-construction beach slopes will match the native beach slopes but will be offset seaward by the design beach width (Fig. 23.18). There is often some *ideal* or *healthy* **composite beach profile** that represents the features, primarily slopes, of the beaches along a coast. This is essentially an estimated profile based on site-specific data on the equilibrium beach profile. The data include the actual wave climate and grain-size distribution. Often, a number of profiles estimated for several locations along the beach and based on a common point (HWL, MWL, toe of dunes) can be overlain to develop the composite profile. Care should be taken to account for overstarved profiles that have been previously armored and are not representative of healthy profiles.

When the source material has a different size or size distribution from that of the native sand, a theoretical equilibrium approach based on an equilibrium-beach-slope concept can be used. The time required to reach cross-shore equilibrium of a beachfill can be roughly estimated with some form of a cross-shore transport model.

Longshore transport or **planform equilibration** moves sand from the constructed project limits in the alongshore direction. Natural longshore sand transport processes will move sand from the constructed location to adjacent beaches or inlets. Although the sand is not lost from the littoral system, it is lost from the project. There are three general ways to estimate the rate of planform equilibration:

1. Use of some historical or *background* erosion rate. Historical profile and shoreline change data can be used to estimate volumetric erosion rates. Vital to this estimate is a clear understanding of the local coastal processes, particularly the cause of erosion at the site. This approach assumes that the beachfill sand will leave the project area at the same rate as sand has been leaving the existing beach. However, constructed beaches may change faster than the existing beach has been, because the constructed beaches are not in equilibrium with adjacent beaches and shoals and the wave climate.

2. Use of a *diffusion-type* model for planform equilibration. The spread of sand away from a location may be computed from the same differential equation as that for the spread of heat in a bar, the classic heat or diffusion equation. One theoretical result of this approach is that if the length of a beachfill is doubled, the beachfill longevity will be quadrupled. This type of approach is most appropriate for long, straight coastlines away from the influence of inlets. It suffers from the constraint that it is based on some single, representative-wave-height parameter and does not directly account for the time variability of longshore sand transport.

3. Use of a time-dependent shoreline change model could be applied to estimate the fate of the fill under assumed incident-wave conditions.

Winnowing of fines or **sand-size equilibration** is the wave-induced sorting and winnowing of any fines that may be in the fill material. Many dredging projects include large portions of fine material that will not remain in the active beach environment. The type of dredging used influences this process. In general, hydraulic dredging removes much of the fines during the washing involved in the construction process itself. This can create a suspended sediment plume from the pipe outfall area. The alternative, mechanical placement, probably reduces the construction plume and leaves more room for fill deflation due to winnowing. Alternative sources of beachfill sand are nearby inlets, offshore shoals, upland quarries, back-bay dredging, and sand hauled from a distant source. The amount of fine-sand winnowing can be estimated with the overfill ratio method described in the "Shore Protection Manual," 4th ed., U.S. Army Coastal Engineering Research Center, Washington, D.C.

The duration of each portion of the equilibration process can range from months to years, depending on the size of the beachfill. Larger beachfills in milder wave climates take longer to respond fully.

Because of the public nature of most beachfills and the confusion concerning the *success* or *failures* of beachfills, postconstruction monitoring should be a prearranged, funded component of the project. A well-designed monitoring program will allow a more complete and rational evaluation of beachfill behavior. Also, monitoring results will be valuable for design of future beachfills at the site.

On some coastlines, structures can be built to extend the life of beachfill projects. Offshore segmented breakwaters with a beach nourishment or marsh construction project in the lee are one alternative for constraining beach erosion that can provide environmental habitat benefits. Rosati and Weggel provide design guidance for offshore segmented breakwaters. Bender outlines a headland breakwater concept that may be used to construct pocket beaches with beachfill.

[T. Bender, "An Overview of Segmented Offshore/Headland Breakwater Projects Constructed by the Buffalo District," in "Coastal Engineering Practice '92," Proceedings of a Specialty Conference on the Planning, Design, Construction, and Performance of Coastal Engineering Projects, American Society of Civil Engineers (ASCE). J. D. Rosati, "Functional Design of Breakwaters for Shore Protection: Empirical Methods," Technical Report CERC-90-15, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss.; D. K. Stauble and N. C. Kraus, "Beach Nourishment Engineering and Management Considerations," ASCE. L. S. Tait, Proceedings of the Annual National Conference on Beach Preservation Technology, The Florida Shore & Beach Preservation Association, Tallahassee, Fla. *Shore Protection Manual*, 4th ed., U.S. Army Coastal Engineering Research Center, Government Printing Office, Washington, D.C.; J. R. Weggel, "Coastal Groins and Nearshore Breakwaters," Engineer Manual EM 1110-2-1617, U.S. Army Corps of Engineers, Washington, DC 20314-1000.]

23.14 Monitoring Programs for Coastal Engineering Projects

Postconstruction monitoring of coastal engineering projects should be standard coastal engineering practice. In addition to engineering benefits from such practice, there are public relations and scientific reasons for monitoring.

A formal monitoring program consists of response measurements, such as surveys, and forcing-function monitoring, such as wave climatology. If well designed, the program facilitates a verifiable, fact-based evaluation of project functioning.

Monitoring is especially important for projects that have a potential for adverse impact on nearby beaches, waterways, coastal structures, or dredging. Beachfill projects also should be monitored to obtain information on how fast the sand is leaving project boundaries.

The primary purpose of monitoring programs is to obtain data for future management decisions relating to the project site. Though present ability to model quantitatively the complex natural processes of the coastal zone are limited, project-specific monitoring is a proven method for development of cost-effective engineering solutions. Monitoring provides the data for an improved understanding of how a beach responds to engineering and also why it responds that way. A monitoring program is often funded with project construction.

Coastal Structures

23.15 Effects of Coastal Structures on Beaches

One of the recurring problems with engineered structure on beaches is evaluation of their impact on adjacent beaches. Because the littoral system is interconnected, what is done on one stretch of beach can have significant impacts on nearby beaches. The impact is related to the coastal processes of the area, in particular the longshore sand-transport rate. Longshore sand transport along beaches can extend for many miles. Inlet shoals are part of the same littoral system as the adjacent beaches if sand can move from the beaches to the shoals and vice versa.

Coastal structures by themselves can only function to redistribute the sand that is in the littoral systems. They do not create sand.

Structures such as groins that are perpendicular to the shore cause a buildup of sand on the updrift side by reducing the sand supply to the downdrift side. A field of groins will essentially realign the shoreline toward the dominant wave climate. Information on groin fields is limited but there is some indication that they are probably beneficial in stabilizing some stretches of coast. They probably provide wave protection and slow the local longshore transport rate because of the local realignment in shoreline angle. Groins, however, reduce the sand movement to the downdrift side. Also, they can induce nearshore circulation cells with rip currents capable of removing sand from the beach face into the sand-bar system. The net result is probably a beach system with a more stable shoreline. The beach neither comes nor goes as much as it would without the groins.

Inlet jetties can have impacts on adjacent beaches as do groins because of trapping of sand on the updrift side. They also can realign and change the volume of sand that is stored in the tidal shoal system.

Mechanical bypassing of sand is used to maintain the littoral drift system at some engineered inlets. The bypassing can consist of either periodic dredging with disposal on downdrift beaches or construction of a fixed bypassing plant. Design of a bypassing scheme is based on estimates of both the rate and variability of longshore sand transport at the site.

Dredging of navigation channels, even without inlet jetties, can contribute to beach erosion through two mechanisms:

1. The sand dredged from the ebb-tidal shoal is often disposed offshore in deeper water and thus removed from the littoral system.
2. The dredging can realign the main ebb-tidal channel, which results in realignment of the ebb-tidal shoals and thus affects the sheltering of the adjacent beaches provided by those shoals.

Seawalls, bulkheads, and revetments protect the land behind the wall, not the beach in front of the wall. There is little clear evidence that seawalls actually cause erosion, except for the impact of the reduction in sediment available for the littoral system if the shoreline erodes to the seawall. However, a seawall constructed on a beach that is receding for other reasons, the usual site for a seawall, will contribute to loss of beach. ("Shore Protection Manual," 4th ed., U.S. Army Coastal Engineering Research Center, Government Printing Office, Washington, D.C., "Coastal Engineering

Manual," (www.USACE.mil/net/asce-docs/eng-manuals/em-htm).)

23.16 Revetment and Seawall Design

A revetment, or seawall, is built to protect property from erosion. Many revetments are constructed as a rubble mound, because the technique has proved to be effective in the harsh and dynamic wave environment. One particularly attractive feature of rubble-mound structures is that, when designed correctly, they continue to function even after damage due to storms larger than the design storm.

There are four typical failure mechanisms for revetments: inadequate armoring, flanking, toe scour, and splash. All are related to inadequate protection of the underlying soil on the downstream side of the structure. Armor, or front-face, design includes selection of the armor unit size and layer thickness plus any underlayers required. For rubble-mound structures (Fig. 23.19), the armor-unit rock size can be calculated from Hudson's equation:

$$W = \frac{w_r H^3}{K_D (S_r - 1)^3 \cot \theta} \tag{23.7}$$

where W = median weight of armor units

H = design wave height

S_r = specific weight of armor unit material

w_r = unit weight of armor unit

θ = slope of structure face

K_D = empirical coefficient that includes shape, roughness, and interlocking ability of the armor units

Hudson's equation is based on a 5% damage level observed in laboratory tests with monochromatic waves. There is no factor of safety inherent in Hudson's equation.

For rough, angular, randomly placed quarry-stone placed in at least two layers, K_d varies from 1.5 to 4, depending on the structure slope, whether the stones are exposed to breaking or nonbreaking waves, and whether the stone is at the head of a pointed structure or in the main body of a long revetment. For breaking waves on the main body of a revetment of 1:3 slope, $K_D = 2.0$ when the gradation of the individual units is within the fairly narrow range $0.75W < W < 1.25W$. For small design wave heights ($H < 5$ ft), a wider range of stone size, riprap with a weight range of $0.25W_{50} < W < 4W_{50}$, can be used with a stability coefficient of 2.2 to 2.5. W_{50} is the median stone weight. The stability coefficient can be higher for concrete armor units, such as tetrapods, dolosse, or quadrapods.

The design wave height H used in Eq. (23.7) should be the average of the heights of the highest 10% H_{10} or 5% H_5 of the waves. The wave height is often depth-limited.

Slopes of 1:2 (vertical to horizontal) or 1:3, for heavy wave action, are recommended. Although

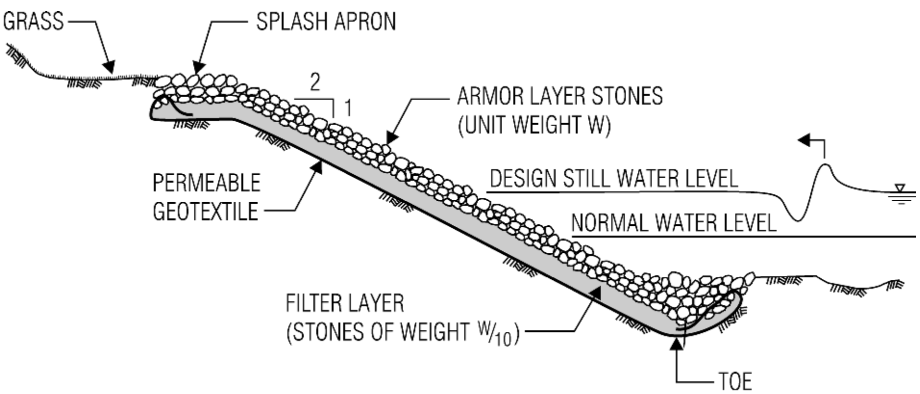

Fig. 23.19 Rubble-mound revetment with armor layer stones, underlain by a stone filter layer and permeable geosynthetic membrane.

slopes as high as 1:1.5 have been tested with Hudson's equation and much steeper slopes can be built by careful crane operators, more failures have occurred on these steeper slopes. Alternatives for steeper slopes include vertical sheetpile bulkheads or gabions. **Gabions**, galvanized wire baskets with rocks, are used in very mild ($H < 1$ ft) wave environments.

Underlayers should be sized so that the stones do not get pulled through the gaps in the overlaying layer. To meet this requirement, underlayer stones should have a minimum median weight of 10% of the overlying median weight. Permeable geotextiles may be installed between the revetment layers and the underlying soil. The toe design shown in Fig. 23.19 allows armor units to roll down into any scour hole that may form.

An alternative to the type of design in Fig. 23.19 with its layers and large armor units is a **dynamic revetment** with a larger volume of stones of a smaller size and wider gradation. The objective of the dynamic revetment is to allow the revetment to move in response to storm waves, much as a cobble beach responds.

The splash of storm waves over a revetment can cause failure by removing underlying soils. The level of wave runup above the design still-water level can exceed the equivalent of one to two full wave heights, depending on the structure slope and roughness and the incident wave period. Runup elevations of irregular waves vary following a form of a Rayleigh distribution, much as wave heights do. Revetments are commonly designed for a certain level of runup and are provided with a splash apron of stones or grass, or both, for protection against the waves that overtop the structures. Estimating the amount of water washing over a seawall or bulkhead is difficult because of limited test data and the extreme sensitivity of overtopping to still-water level.

Flanking due to shoreline recession at the ends of a revetment can be avoided by either tying it into another adjacent seawall or revetment or constructing return walls perpendicular to the shore. Return walls should be long enough to protect against long-term shoreline recession, any storm recession, and excess storm erosion due to the presence of the seawall. Since a structurally sound seawall protects the underlying sediment from erosion during a storm, excess erosion may occur adjacent to walls during storms. This excess erosion is roughly 20% of the length of the wall.

Bulkhead Design ▪ Design of vertical bulkheads is controlled primarily by geotechnical considerations. Two coastal engineering considerations are the potential for scour at the wall base (due to wave action) and wave-induced pumping action at improperly joined seams. Scour at the toe of a vertical wall can approach 1 to 1.5 times the wave height. Inasmuch as waves are often depth limited and, on a horizontal bottom, the maximum wave height is about equal to the water depth, the scour elevation below the mudline is about 1 to 1.5 times the depth of the water above the original mudline. Scour can be accounted for by designing the wall for a lowered mudline.

Another alternative is to design a rubble-mound toe. The size of the armor units at the base of the rubble mound can be determined from Eq. (23.7) for toes that will be exposed to breaking waves. For submerged toes, the design median weight of the rocks should be

$$W = \frac{w_r H^3}{(S_r - 1)^3 N_s^3} \tag{23.8}$$

where N_s, the stability number, varies from $1.8 < N_s < 5$. H is depth limited or H_{10} or H_5. Other terms are defined as given for Eq. (23.7).

(Y. Goda, "Random Seas and Design of Maritime Structures," University of Tokyo Press, Tokyo. J. B. Herbich, "Handbook of Coastal and Ocean Engineering," Gulf Publishing Company, Houston, Tex (www.gulfpub.com). "Seawall, Bulkheads, and Revetment Design," Engineer Manual EM-1110-2-1614, U.S. Army Corps of Engineers, Washington, DC 20314-1000 (www.USACE.army.mil/inet/USACE-docs/eng-manuals/em-htm).)

23.17 Use of Physical and Numerical Models in Design

Physical and numerical models are used in coastal engineering for a variety of reasons. The traditional physical model is used to account for turbulent processes that limit the ability of equations to predict phenomena. With small-scale physical models, engineers have the ability to address problems with site-specific and design-specific geometry, but the models have some scaling constraints that limit their usefulness. Numerical models, or systems of equations, have been developed to address a wide variety of coastal engineering related problems. Use of any model should consist of two distinct phases, calibration and verification, before the application.

(M. V. Cialone, "The Coastal Modeling System (CMS): A Coastal Processes Software Package," *Journal of Coastal Research*, vol. 10, no. 3, pp. 576–587; S. A. Hughes, "Physical Models and Laboratory Techniques in Coastal Engineering," World Scientific Publishing Co., River Edge, Nev.)