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Coastal Engineering

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36.1 Wave Mechanics

Waves on the surface of a natural body of open water are the result of disturbing forces that create a deformation, which is restored to equilibrium by, gravitational and surface tension forces. Surface waves are characterized by their height, length, and the water depth over which they are traveling. Figure 36.1 shows a two-dimensional sketch of a sinusoidal surface wave propagating in the *x*-direction. The **wave height**, H, is the vertical distance between its crest and leading trough. Wavelength, L, is the horizontal distance between any two corresponding points on successive waves and wave period is the time required for two successive crests or troughs to pass a given point. The **celerity** of a wave C, is the speed of propagation of the waveform (phase speed), defined as C = L/T. Most ocean waves are progressive; their waveform appears to travel at celerity C relative to a background. Standing waves, their waveforms remains stationary relative to a background, occur from the interaction of progressive waves traveling in opposite directions and are often observed near reflective coastal features. Progressive deep ocean waves are oscillatory meaning that the water particles making up the wave do not exhibit a net motion in the direction of wave propagation. However, waves entering shallow-water begin to show a net displacement of water in the direction of propagation and are classified as translational. The equilibrium position used to reference surface wave motion, (Still Water Level SWL) is z = 0 and the bottom is located at z = –d (Fig. 36.1).

The free surface water elevation, η, for a natural water wave propagating over an irregular, permeable bottom may appear quite complex. However, by assuming that, viscous effects are negligible (concentrated near the bottom), flow is irrotational and incompressible, and wave height is small compared to wavelength,

* Professor William L. Wood, first author of this chapter in the first edition, died in 1997.

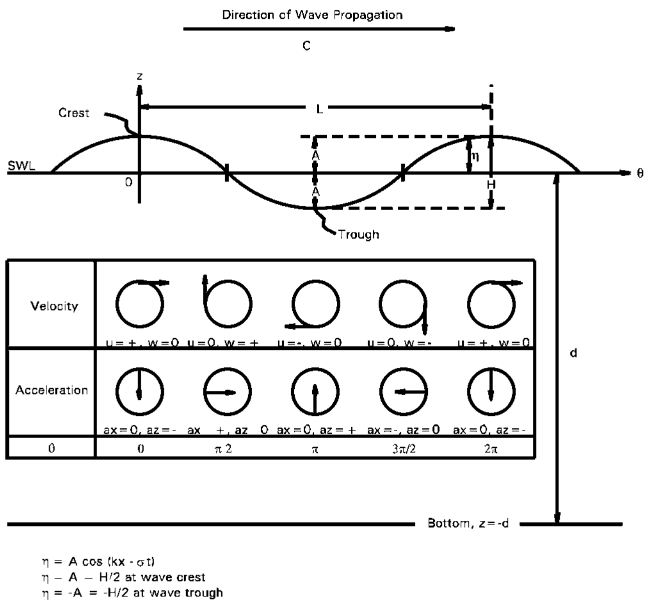

FIGURE 36.1 Definition sketch of free surface wave parameters for a linear progressive wave. Shown also are the fluid particle velocities and accelerations associated with each portion of the wave.

a remarkably simple solution can be obtained for the surface wave boundary value problem. This simplification, referred to as linear, small-amplitude wave theory, is extremely accurate and easy to use in many coastal engineering applications. Furthermore, the linear nature of this formulation allows for the free surface to be represented by superposition of sinusoids of different amplitudes and frequencies, which facilitates the application of Fourier decomposition and associated analysis techniques.

Progressive, Small-Amplitude Waves — Properties

The equation for the free surface displacement of a progressive wave is

$$\eta = A\cos(kx - \sigma t) \tag{36.1}$$

where amplitude, $A = H/2$
wave number, $k = 2\pi/L$
wave frequency $\sigma = 2\pi/T$

The expression relating individual wave properties and water depth, d, to the propagation behavior of these waves is the dispersion relation,

$$\sigma^2 = gk \tanh kd, \tag{36.2}$$

where g is the acceleration of gravity.

From Eq. (36.2) and the definition of celerity (C) it can be shown that

$$C = \frac{\sigma}{k} = \frac{gT}{2\pi} \tanh kd \tag{36.3}$$

and

$$L = \frac{g\,T^2}{2\pi} \tanh kd \tag{36.4}$$

The hyperbolic function *tanh kd* approaches useful simplifying limits of 1 for large values of *kd* (deep water) and *kd* for small values of *kd* (shallow water). Applying these limits to Eqs. (36.3) and (36.4) results in expressions for deep water of

$$C_o = \frac{gT}{2\pi} = 5.12T \quad \text{(English units, ft/s)}$$

or

$$C_o = 1.56T \quad \text{(SI units, m/s)}$$

and

$$L_o = \frac{gT^2}{2\pi} = 5.12\,T^2 \quad \text{(English units, ft)} \tag{36.5}$$

or

$$L_o = 1.56T^2 \quad \text{(SI units, m).}$$

A similar application for shallow water results in

$$C = \sqrt{g\,d} \tag{36.6}$$

which shows that wave speed in shallow water is dependent only on water depth. The normal limits for deep and shallow water are $kd > \pi$ and $kd < \pi/10$ ($d/L > 1/2$ and $d/L < 1/20$) respectively, although modification of these limits may be justified for specific applications. The region between these two limits ($\pi/10 < kd < \pi$) is defined as intermediate depth water and requires use of the full Eqs. (36.3) and (36.4).

Some useful functions for calculating wave properties at any water depth, from deep water wave properties, are

$$\frac{C}{C_o} = \frac{L}{L_o} = \tanh \frac{2\pi d}{L}. \tag{36.7}$$

Values of d/L can be calculated as a function of d/L_o by successive approximations using

$$\frac{d}{L} \tanh \frac{2\pi d}{L} = \frac{d}{L_o} \tag{36.8}$$

The term d/L has been tabulated as a function of d/L_o by Wiegel (1954) and is presented, along with many other useful functions of d/L in Appendix C of The Shore Protection Manual (U.S. Army Corps of Engineers, 1984).

Particle Motions

The horizontal component of particle velocity beneath a wave is

$$u = \frac{H}{2}\,\sigma\,\frac{\cosh k(d+z)}{\sinh kd} \cos(kx - \sigma t) \tag{36.9}$$

The corresponding acceleration is

$$a_x = \frac{\partial u}{\partial t} = \frac{H}{2}\sigma^2 \frac{\cosh k\,(d+z)}{\sinh\ kd}\sin\,(kx-\sigma t) \tag{36.10}$$

The vertical particle velocity and acceleration are respectively

$$w = \frac{H}{2}\sigma \frac{\sinh\ k(d+z)}{\sinh\ kd}\ \sin\,(kx-\sigma t) \tag{36.11}$$

and

$$a_z = \frac{\partial w}{\partial t} = -\frac{H}{2}\sigma^2 \frac{\sinh\ k(d+z)}{\sinh\ kd}\ \cos\,(kx-\sigma t) \tag{36.12}$$

It can be seen from Eqs. (36.9) and (36.11) that the horizontal and vertical particle velocities are 90° out of phase at any position along the wave profile. Extreme values of horizontal velocity occur in the crest (+, in the direction of wave propagation) and trough (–, in the direction opposite to the direction of wave propagation) while extreme vertical velocities occur mid-way between the crest and trough, where water displacement is zero. The u and w velocity components are at a minimum at the bottom and both increase as distance upward in the water column increases. Maximum vertical accelerations correspond to maximum in horizontal velocity and maximum horizontal accelerations correspond to maximum in vertical velocity. Figure 36.1 provides a graphic summary of these relationships.

The particle displacements can be obtained by integrating the velocity with respect to time and simplified by using the dispersion relationship (Eq. [36.2]) to give a horizontal displacement

$$\xi = -\frac{H}{2}\frac{\cosh k(d+z_o)}{\sinh kd}\ \sin\,(kx_o-\sigma t) \tag{36.13}$$

and vertical displacement

$$\zeta = \frac{H}{2}\frac{\sinh k(d+z_o)}{\sinh kd}\ \cos\,(kx_o-\sigma t) \tag{36.14}$$

where (x_o, z_o) is the mean position of an individual particle.

It can be shown by squaring and adding the horizontal and vertical displacements that the general form of a water particle trajectory beneath a wave is elliptical. In deep water, particle paths are circular and in shallow-water they are highly elliptical as shown in Fig. 36.2.

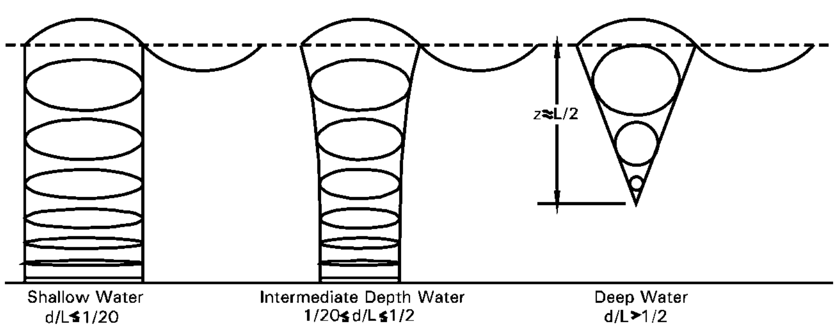

FIGURE 36.2 Fluid particle paths for linear progressive waves in different relative water depths.

Pressure Field

The pressure distribution beneath a progressive water wave is given by the following form of the Bernoulli equation

$$p = -\rho g z + \rho g \eta K_p(z) \tag{36.15}$$

where ρ is fluid density and K_p, the pressure response coefficient, is

$$K_p = \frac{\cosh k(d+z)}{\cosh kd} \tag{36.16}$$

which will always be less than 1, below mean still water level. The first term in Eq. (36.15) is the *hydrostatic pressure* and the second is the *dynamic pressure* term. This dynamic pressure term accounts for two factors that influence pressure, the free surface displacement η and the vertical component of acceleration.

A frequently used method for measuring waves at the coast is to record pressure fluctuations from a bottom-mounted pressure gage. Isolating the dynamic pressure (P_D) from the recorded signal by subtracting out the hydrostatic pressure gives the relative free surface displacement

$$\eta = \frac{P_D}{\rho g K_p(-d)} \tag{36.17}$$

where $K_p(-d) = 1/\cosh kd$.

It is necessary, therefore, when determining wave height from pressure records to apply the dispersion relationship (Eq. [36.2]) to obtain K_p from the frequency of the measured waves. It is important to note that K_p for short period waves is very small at the bottom (–d), which means that very short period waves may not be measured by a pressure gage.

A summary of the formulations for calculating linear wave theory wave characteristics in deep, intermediate, and shallow water is presented in Table 36.1. A much more comprehensive presentation of linear, small-amplitude theory is found in the classic reference Oceanographical Engineering (Weigel, 1954) The Shore Protection Manual (U.S. Army Corps of Engineers, 1984), and Water Wave Mechanics for Engineers and Scientists (Dean and Dalrymple, 1991).

Wave Energy

Progressive surface water waves possess potential energy from the free surface displacement and kinetic energy from the water particle motions. From linear wave theory it can be shown that the average potential energy per unit surface area for a free surface sinusoidal displacement, restored by gravity, is

$$\bar{E}_p = \frac{\rho g H^2}{16} \tag{36.18}$$

Likewise the average kinetic energy per unit surface area is

$$\bar{E}_k = \frac{\rho g H^2}{16} \tag{36.19}$$

and the total average energy per unit surface area is

$$\bar{E} = \bar{E}_p + \bar{E}_k = \frac{\rho g H^2}{8} \tag{36.20}$$

TABLE 36.1 Summary of Linear Wave Theory, Wave Characteristics, $\theta = (kx - \sigma t)$

Relative Depth	Shallow Water $d/L < 1/20$	Transitional Water $1/20 < d/L < 1/2$	Deep Water $d/L > 1/2$
1. Wave profile	Same as "Transitional Water" →	$\eta = \frac{H}{2}\cos[2kx - \sigma t T] = \frac{H}{2}\cos\theta$	Same as "Transitional Water" ←
2. Wave celerity	$C = \frac{L}{T} = \sqrt{gd}$	$C = \frac{L}{T} = \frac{gT}{2\pi}\tanh(kd)$	$C = C_o = \frac{L}{T} = \frac{gT}{2\pi}$
3. Wavelength	$L = T\sqrt{gd} = CT$	$L = \frac{gT^2}{2\pi}\tanh(kd)$	$L = L_o = \frac{gT^2}{2\pi} = C_o T$
4. Group velocity	$C_g = C = \sqrt{gd}$	$C_g = nC = \frac{1}{2}\left[1 + \frac{2kd}{\sinh(2kd)}\right]\cdot C$	$C_g = \frac{1}{2}C = \frac{gT}{4\pi}$
5. Water particle velocity (a) Horizontal	$u = \frac{H}{2}\sqrt{\frac{g}{d}}\cos\theta$	$u = \frac{H}{2}\sigma\frac{\cosh k(z+d)}{\sinh kd}\cos\theta$	$u = \frac{\pi H}{T}e^{kz}\cos\theta$
(b) Vertical	$w = \frac{H\pi}{T}\left(1 + \frac{z}{d}\right)\sin\theta$	$w = \frac{H}{2}\sigma\frac{\sinh k(z+d)}{\sinh kd}\sin\theta$	$w = \frac{\pi H}{T}e^{kz}\sin\theta$
6. Water particle accelerations (a) Horizontal	$a_x = \frac{H\pi}{T}\sqrt{\frac{g}{d}}\sin\theta$	$a_x = \frac{H}{2}\sigma^2\frac{\cosh k(z+d)}{\sinh kd}\sin\theta$	$a_x = 2H\left(\frac{\pi}{T}\right)^2 e^{kz}\sin\theta$
(b) Vertical	$a_z = -2H\left(\frac{\pi}{T}\right)^2\left(1 + \frac{z}{d}\right)\cos\theta$	$a_z = -\frac{H}{2}\sigma^2\frac{\sinh k(z+d)}{\sinh kd}\cos\theta$	$a_z = -2H\left(\frac{\pi}{T}\right)^2 e^{kz}\cos\theta$
7. Water particle displacements (a) Horizontal	$\xi = -\frac{HT}{4\pi}\sqrt{\frac{g}{d}}\sin\theta$	$\xi = -\frac{H}{2}\frac{\cosh k(z+d)}{\sinh kd}\sin\theta$	$\xi = -\frac{H}{2}e^{kz}\sin\theta$
(b) Vertical	$\zeta = \frac{H}{2}\left(1 + \frac{z}{d}\right)\cos\theta$	$\zeta = \frac{H}{2}\frac{\sinh k(z+d)}{\sinh kd}\cos\theta$	$\zeta = \frac{H}{2}e^{kz}\cos\theta$
8. Subsurface pressure	$p = \rho g(\eta - z)$	$p = \rho g\eta\frac{\cosh k(z+d)}{\cosh kd} - \rho g z$	$p = \rho g\eta e^{kz} - \rho g z$

The unit surface area considered is a unit width times the wavelength L so that the total energy per unit width is

$$E_T = \frac{1}{8}\rho g H^2 L \tag{36.21}$$

The total energy per unit surface area in a linear progressive wave is always equipartitioned as one half potential and one half kinetic energy.

Energy flux is the rate of energy transfer across the sea surface in the direction of wave propagation. The average energy flux per wave is

$$F_E = E\,Cn \tag{36.22}$$

where

$$n = \frac{C_g}{C} = \frac{1}{2}\left(1 + \frac{2kd}{\sinh 2kd}\right) \tag{36.23}$$

and C_g is the *group speed* defined as the speed of energy propagation.

In deep water $n = 1/2$ and in shallow water $n = 1$ indicating that energy in deep water travels at half the speed of the wave while in shallow water energy propagates at the same speed as the wave.

Wave Shoaling

Waves entering shallow water conserve period and, with the exception of minor losses, up to breaking, conserve energy. However, wave celerity decreases as a function of depth and correspondingly wavelength shortens. Therefore, the easiest conservative quantity to follow is the energy flux (given in Eq. [36.22]), which remains constant as a wave shoals. Equating energy flux in deep water (H_o, C_o) to energy flux at any shallow water location (H_x, C_x) results in the general shoaling relation

$$\frac{H_x}{H_o} = \left(\frac{1}{2n}\frac{C_o}{C_x}\right)^{1/2} \tag{36.24}$$

where n is calculated from Eq. (36.23) and C_o/C_x can be obtained from Eq. (36.7).

Therefore, by knowing the deep water wave height and period (H_o, T_o)1 and the bathymetry of a coastal region, the shoaling wave characteristics (H_x, C_x, L_x) can be calculated at any point, x, prior to breaking. A limitation to Eq. (36.24) is that it does not directly incorporate the effect of deep-water angle of approach to the coast.

Wave Refraction

It can be shown that a deep water wave approaching a coast at an angle α_o and passing over a coastal bathymetry characterized by straight, parallel contours refracts according to Snell's law:

$$\frac{\sin \alpha_o}{C_o} = \frac{\sin \alpha}{C} \tag{36.25}$$

Since waves in shallow water slow down as depth decreases, application of Snell's law to a plane parallel bathymetry indicates that wave crests tend to turn to align with the bathymetric contours. Unfortunately, most offshore bathymetry is both irregular and variable along a coast and the applicable **refraction** techniques involve a non-linear partial differential equation, which can be solved approximately by various computer techniques (Noda, 1974; RCPWAVE, and others). However, there are more easily applied ray tracing methods that use Snell's law applied to idealized bathymetry (bathymetry that has been "smoothed" to eliminate abrupt turns and steep gradients). The U.S. Army Corps of Engineers distributes an easy to use set of PC computational programs, Automated Coastal Engineering System (ACES), which include a Snell's law ray tracing program.

Considering two or more wave rays propagating shoreward over plane parallel bathymetry, Fig. 36.3, it is possible to have the rays either converge or diverge. Under these conditions, the energy per unit area may increase (convergence) or decrease (divergence) as a function of the perpendicular distance of separation between wave rays b_o and b_x. Using the geometric relationships shown in Fig. 36.3, Eq. (36.24) is modified to account for convergence and divergence of wave rays as

$$\frac{H_x}{H_o} = \left(\frac{1}{2n}\frac{C_o}{C_x}\right)^{1/2}\left(\frac{b_o}{b_x}\right)^{1/2} \tag{36.26}$$

also written as

$$H_n = H_o K_s K_R \tag{36.27}$$

where K_s = the shoaling coefficient
K_R = the refraction coefficient.

This expression is equally valid between any two points along a wave ray in shallow water.

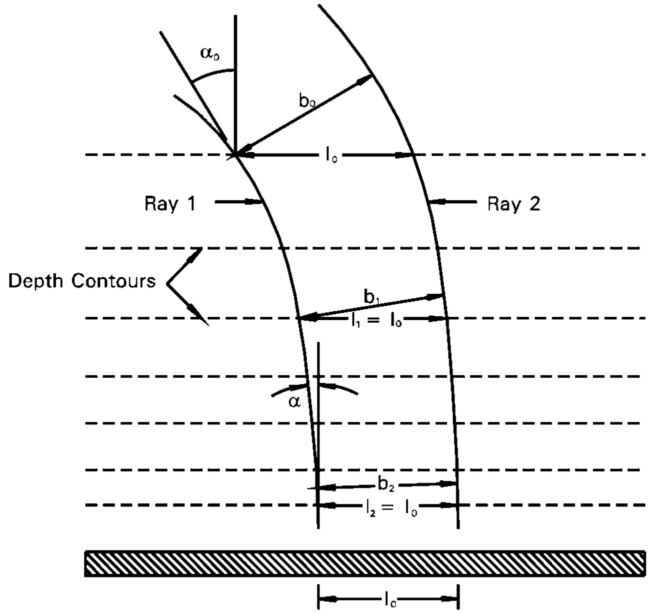

FIGURE 36.3 Definition sketch for wave rays refracting over idealized plane parallel bathymetry.

Wave Diffraction

Wave diffraction is a process by which energy is transferred along the crest of a wave from an area of high energy density to an area of low energy density. There are two important coastal engineering applications of diffraction. First, as wave rays converge and diverge in response to natural changes in bathymetry the K_R term in Eq. (36.27) will increase and decrease respectively. As a result, energy will move along the wave crest from areas of convergence to areas of divergence. It is, therefore, necessary to consider the effects of both refraction and diffraction when calculating wave height transformation due to shoaling. The U.S. Army Corps of Engineers RCPWAVE is a PC compatible program capable of doing these calculations for "smoothed" bathymetry.

The fundamental equations used to carry out diffraction calculations are based on the classical Sommerfield relation

$$\eta = \frac{AkC}{g} \cosh kd \left| F(r, \Psi) \right| e^{ikCt} \tag{36.28}$$

where

$$K' = \frac{H_o}{H_i} = \left| F(r, \Psi) \right| \tag{36.29}$$

The second, and perhaps most important, application of wave diffraction is that due to wave-structure interaction. For this class of problems, wave diffraction calculations are essential for obtaining the distribution of wave height in harbors or behind engineered structures. There are three primary types of wave-structure diffraction, important to coastal engineering (Fig. 36.4a-c): (a) diffraction at the end of a single breakwater (semi-infinite); (b) diffraction through a harbor entrance (gap diffraction); and (c) diffraction around an offshore breakwater.

The methods of solution for all three of these wave-structure interactions are similar, but are restricted by some important assumptions. For each case there is a *geometric shadow zone* on the sheltered side of

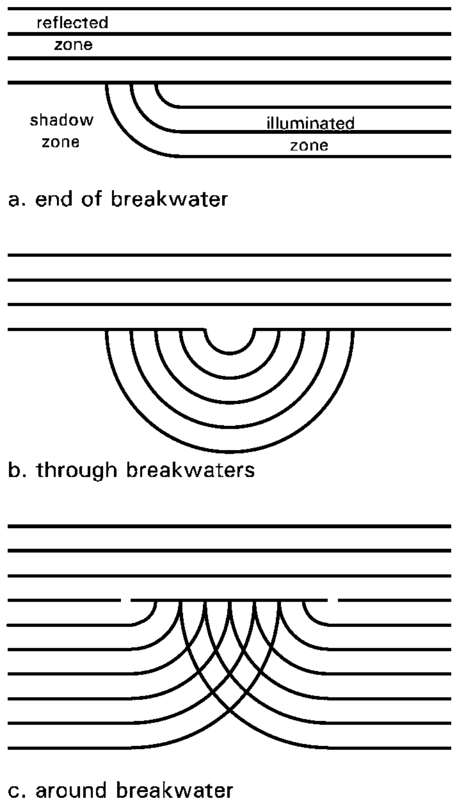

FIGURE 36.4 Wave diffraction patterns around breakwaters.

the structure, a *reflected wave zone* on the front or incident wave side of the structure, and an "*illuminated*" *zone* in the area of direct wave propagation (Fig. 36.4a). The solution to $F(r, \Psi)$ is complicated, however, The Shore Protection Manual (1984) provides a series of templates for determining diffraction coefficients K', defined as the ratio of wave height in the zone affected by refraction to the unaffected incident wave height, for semi-infinite breakwaters and for breakwater gaps between 1 and 5 wave lengths (L) wide. For breakwater gaps greater than 5L the semi-infinite templates are used independently and for gaps 1L or less, a separate set of templates are provided (The Shore Protection Manual (U.S. Army Corps Of Engineers, 1984)). A basic diffraction-reflection calculation program is also provided in the Automated Coastal Engineering System (ACES, 1992).

Wave Breaking

Waves propagating into shallow water tend to experience an increase in wave height to a point of instability at which the wave breaks, dissipating energy in the form of turbulence and work done on the bottom. Breaking waves are classified as**: spilling breakers** generally associated with low sloping bottoms and a gradual dissipation of energy; **plunging breakers** generally associated with steeper sloping bottoms and a rapid, often spectacular, "explosive" dissipation of energy; and *surging breakers* generally associated with very steep bottoms and a rapid narrow region of energy dissipation. A widely used classic criteria (McCowan, 1894) applied to shoaling waves relates breaker height H_b to depth of breaking d_b through the relation

$$H_b = 0.78\, d_b \tag{36.30}$$

However, this useful estimate neglects important shoaling parameters such as bottom slope (m) and deepwater wave angle of approach (α_o). Dean and Dalrymple (1991) used Eq. (36.26) and McCowan's breaking criteria to solve for breaker depth (d_b), distance from the shoreline to the breaker line (x_b) and breaker height (H_b) as

$$d_b = \frac{1}{g^{1/5}\kappa^{4/5}}\left(\frac{H_0^2\, C_0 \cos\alpha_0}{2}\right)^{2/5} \tag{36.31}$$

$$x_b = \frac{d_b}{m} \tag{36.32}$$

and

$$H_b = \kappa\, d_b = \kappa m x_b = \left(\frac{\kappa}{g}\right)^{1/5}\left(\frac{H_0^2\, C_0 \cos\alpha_0}{2}\right)^{2/5} \tag{36.33}$$

where m = beach slope
$\kappa = H_b/d_b$

Dalrymple et al. (1977) compared the results of a number of laboratory experiments with Eq. (36.32) and found it under predicts breaker height by approximately 12% (with $\kappa = 0.8$). Wave breaking is still not well understood and caution is urged when dealing with engineering design in the active breaker zone.

36.2 Ocean Wave Climate

The Nature of the Sea Surface

As the wind blows across the surface of the sea, a large lake, or a bay, momentum is imparted from the wind to the sea surface. Of this momentum, approximately 97% is used in generating the general circulation (currents) of the water body and the remainder supplies the development of the surface wave field. Although this surface wave momentum represents a small percentage of the total momentum, it results in an enormous quantity of wave energy.

Within the region of active wind-wave generation, the sea surface becomes very irregular in size, shape, and direction of propagation of individual waves. This disorderly surface is referred to as **sea**. As waves propagate from their site of generation, they tend to sort themselves out into a more orderly pattern. This phenomenon, known as dispersion, is due to the fact that longer period waves tend to travel faster, while short period waves lag behind (see Eq. [36.2]). Therefore, **swell** is a term applied to waves, which have propagated outside the region of active wind wave generation and are characterized by a narrow distribution of periods, and regular shape and a narrow direction of travel.

Given these distinctions between sea and swell it is reasonable to expect that the statistical description of the sea surface would be very different for each case. The wave spectrum is a plot of the energy associated with each frequency ($f = 1/T$) component of the sea surface. The difference between sea and swell spectra is shown schematically in Fig. 36.5. Note that the sea spectrum is typically broadly distributed in frequency while the swell spectrum is narrowly distributed in frequency (tending toward monochromatic waves).

The directional distribution of wave energy propagation is given by the two-dimensional directional wave energy spectrum. Just as in the case of the one-dimensional wave energy spectrum, the two-dimensional spectrum provides a plot of wave energy versus frequency; however, the spectrum is further defined by the direction of wave propagation.

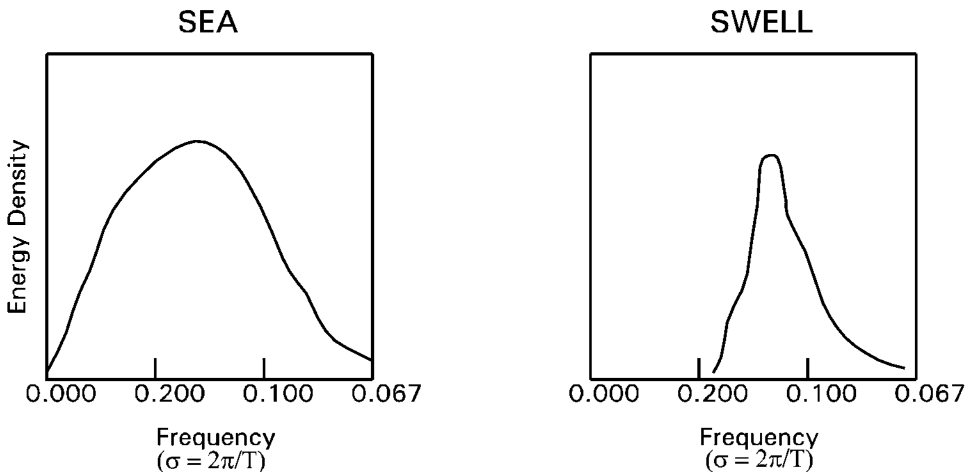

FIGURE 36.5 Characteristic energy diagrams showing the difference between sea and swell.

TABLE 36.2 One-Dimensional Wave Prediction Formulae

SMB	$H_s = 0.283\, g^{-1}\, U^2 \tanh\left[0.0125\left(\frac{gx}{U_{10}^2}\right)^{0.42}\right]$
	$T = 7.54\, g^{-1}\, U \tanh\left[0.077\left(\frac{gx}{U_{10}^2}\right)^{0.25}\right]$
JONSWAP	$H_s = 0.0016\, g^{0.5}\, U_x^{0.5}$
	$T = 0.286\, g^{0.62}\, U_{10}^{0.33}\, x^{0.33}$
Donelan	$H_s = 0.00366\, g^{-0.62}\, U_{10}^{1.24} \times x^{0.38} (\cos\phi)^{1.24}$
	$T = 0.54\, g^{-0.77}\, U_{10}^{0.54}\, x^{0.23} (\cos\phi)^{0.54}$
where:	H_s = significant wave height (in meters) T = peak energy wave period (in seconds) U_{10} = wind speed at 10m height (in meters per second) x = fetch length (in wave direction for Donelan formulas) φ = angle between wind and waves g = 9.8 m/s^2

Wave Prediction

The wave height and associated energy contained in the sea surface is generally dependent on three parameters: the speed of the wind, U, measured at 10 m above the sea surface, the open water distance over which the wind blows, **fetch** length, x and the length of time the wind does work on the sea surface, duration, t. The growth of a wind driven sea surface may be limited by either the fetch or duration, producing a sea state less than "fully arisen" (maximum energy) for a given wind speed.

One-dimensional wave prediction models generally consist of equations, which estimate wave height and wave period at a particular location and time as a function of fetch length and wind speed. Three examples of one-dimensional wave prediction formulas are provided in Table 36.2. It should be noted that the wind speed utilized in these wave prediction models must be obtained from, or corrected to, a height of 10 m above the water surface. A widely used approximation for correcting a wind speed, measured at height z over the open ocean, to 10 m is

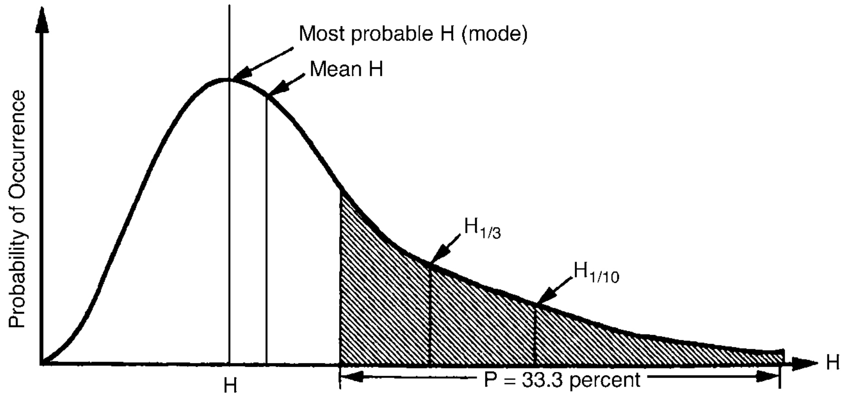

FIGURE 36.6 Statistical distribution of wave heights.

$$U_{10} = U_z \left(\frac{10}{z}\right)^{1/7} \tag{36.34}$$

If the wind speed is measured near the coast, the exponent used for this correction is 2/7. In the event that over water winds are not available, over land winds may be utilized, but need to be corrected for frictional resistance. This is due to the fact that the increased roughness typically present over land sites serves to modify the wind field. A concise description of this methodology is presented in the Shore Protection Manual (1984).

Since the natural sea surface is statistically complex, the wave height is usually expressed in terms of the average of the one-third largest waves or the **"significant wave height,"** H_s. The significant wave period corresponds to the energy peak in the predicted wave spectrum. Other expressions for wave height which are commonly used in design computations are H_{max}, the maximum wave height, H_{rms}, the root mean square wave height, $\bar{H}$, the average wave height, H_{10}, the average of highest 10 percent of all waves, H_1, the average of the highest 1 percent of all waves (Fig. 36.6). The energy-based parameter commonly used to represent wave height is H_{10}, which is an estimate of the significant wave height fundamentally related to the energy distribution of a wave train. Table 36.3 summarizes the relationship between these various wave height parameters.

When predicting wave generation by hurricanes, the determination of fetch and duration is much more difficult due to large changes in wind speed and direction over short time frames and distances. Typically, the wave field associated with the onset of a hurricane or large storm will consist of a locally generated sea superimposed on swell components from other regions of the storm (see The Shore Protection Manual [U.S. Army Corps Of Engineers, 1984]).

TABLE 36.3 Summary of Approximate Statistical Wave Height Relations

	$\bar{H}3$	H_{rms}	H_s	H_{10}	H_1	H_{max}
$\bar{H}2$		.89	.63	.49	.38	0.33
H_{rms}	1.13		.71	.56	.42	0.38
H_s	1.60	1.42		.79	.60	0.53
H_{10}	2.03	1.80	1.27		.76	0.68
H_1	2.67	2.37	1.67	1.31		0.89
H_{max}	2.99	2.65	1.87	1.47	1.12	

See accompanying text for explanations of various wave height designations.

Wave Data Information Sources

Several sources of wave data exist in both statistical and time series form. The National Climatic Data Center (NCDC) has compiled summaries of ship observations, over many open water areas, into the series: Summary of Synoptic Meteorological Observations (SSMO). These publications present statistical summaries of numerous years of shipboard observations of wind, wave and other environmental conditions. These publications may be purchased through NCDC/NOAA Asheville, North Carolina.

The National Data Buoy Center (NDBC) is responsible for the archiving of wave and weather data collected by their network of moored, satellite reporting buoys. These buoys report hourly conditions of wave height and period, as well as wind speed and direction, air and sea temperature and other meteorological data. This information can be obtained in time series form (usually hourly observations) from the NDBC office.

Another statistical summary of wind and wave data is available from the U.S. Army Corps of Engineers, Waterways Experiment Station, Coastal and Hydraulics Laboratory. The primary purpose of the Wave Information Study (WIS) is to provide an accurate and comprehensive database of information of the long-term wave climate. The WIS generally uses a complete series of yearly wind records, which varies in length from 20 to 40 years. The study considers the effects of ice cover where applicable and reflects advances in the understanding of the physics involved in wave generation, propagation, and dissipation, employing currently developed techniques to model these processes. The summary tables generated from the WIS hindcast include: percent occurrence of wave height and period by direction, a wave "rose" diagram, the mean significant wave height by month and year, the largest significant wave height by month and year and total summary statistics for all of the years at each station. In addition, the study also provides return period tables for the 2-, 5-, 10-, 20-, and 50-year design waves. WIS reports for ocean coastal areas of the U.S. and the Great Lakes can be obtained from CERC.

36.3 Water Level Fluctuations

Long period variations of water level occur over a broad range of time scales, greater than those of sea waves and swell. These types of fluctuations include astronomical tides, **seiches**, **tsunamis**, wave setup, and storm surge as well as very long period (months to years) variations related to climatologic and eustatic processes.

Tides

Tides are periodic variations in mean sea level caused by gravitational attraction between the earth, moon and sun and by the centrifugal force balance of the three-body earth, moon, sun system. Although complicated, the resultant upward or downward variation in mean sea level at a point on the earth's surface can be predicted quite accurately. Complete discussions of tidal dynamics are given in Defant (1961), Neumann and Pierson (1966), Apel (1990). The specific computational approach currently being used for official tide prediction in the United States is described in Pore and Cummings (1967). Tide tables for the coastlines of the United States can be obtained for the U.S. Department of Commerce, National Ocean Service, Rockville, MD.

Tides tend to follow a lunar (moon) cycle and thus show a recurrence pattern of approximately 1 month. During this one month cycle there will be two periods of maximum high and low water level variation, called **spring tides** and two periods of minimum high and low water level variation, called **neap tides** (Fig. 36.7). Figure 36.7 also illustrates the different types of tides that may occur at the coast. These tides may be: diurnal, high and low tide occur once daily; semi-diurnal, high and low tides occur twice daily; or mixed, two highly unequal high and low tides occur daily. Diurnal tides occur on a lunar period of 24.84 hours and semi-diurnal tides occur on a half lunar period of 12.42 hours. Therefore, the time of occurrence of each successive high of low tide advances approximately 50 (diurnal) or 25 (semi-diurnal) minutes.

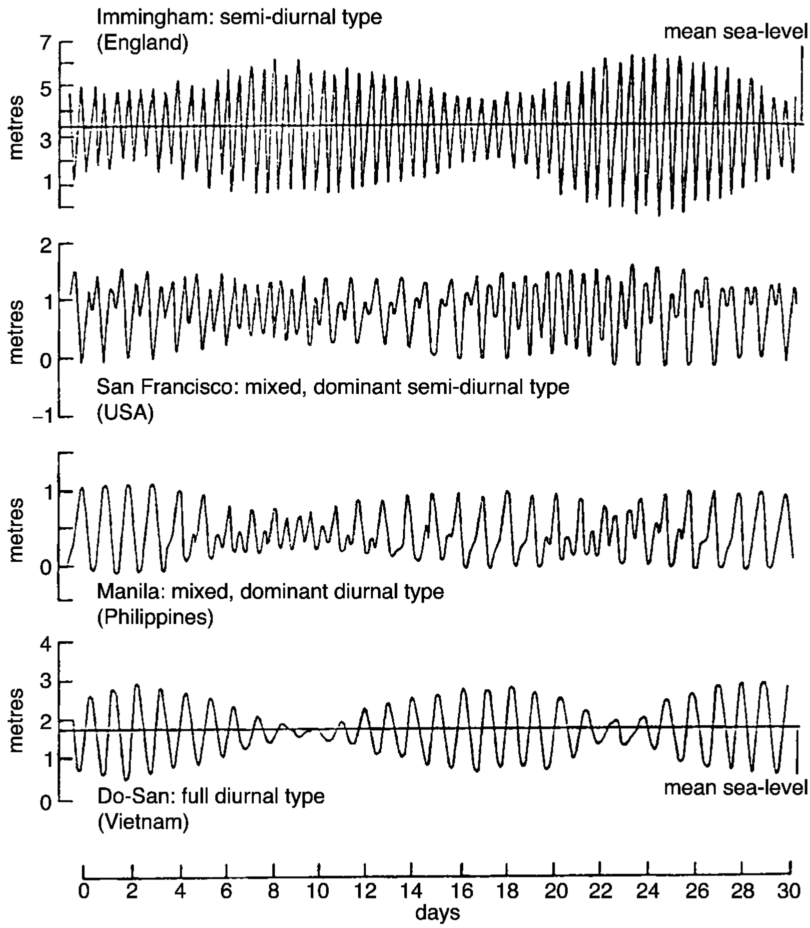

FIGURE 36.7 Different types of tides that may occur at a coast.

Seiches

Seiches are long period standing waves formed in enclosed or semi-enclosed basins such as lakes, bays, and harbors. Seiches are usually generated by abrupt rapid changes in pressure or wind stress. The natural free oscillation period, T_n, of a seiche in an *enclosed* rectangular basin of constant depth is

$$T_n = \frac{2l_b}{n\sqrt{g\,d}} \tag{36.35}$$

where l_b = the basin length in the direction of travel
n = the number of nodes along the basin length

The maximum period occurs at the fundamental, where $n = 1$. The natural free oscillation period for an *open* rectangular basin (analogous to a bay or harbor) is

$$T'_{n'} = \frac{4l_{b'}}{\left(1+2n'\right)\sqrt{g\,d}} \tag{36.36}$$

where n' is the number of nodes between the node at the opening and the antinode at the opposite end.

A complete discussion of seiches is presented in Huthinson (1957).

Tsunami

Tsunamis are long period progressive gravity waves generated by sudden violent disturbances such as earthquakes, volcanic eruptions, or massive landslides. These long waves usually travel across the open seas at shallow water waves speeds, $c = \sqrt{g\,d}$, and thus can obtain speeds of hundreds of kilometers per hour. These relatively low waves (10s cm) on the open ocean are greatly amplified at the coast and have been recorded at heights in excess of 30 m.

Wave Setup

Wave setup is defined as the superelevation of mean water level caused by wave action at the coast (The Shore Protection Manual, U.S. Army Corps Of Engineers, 1984). This increase in mean water level occurs between the breaking point and the shore. The Shore Protection Manual (1984) gives the following formula for calculating wave setup

$$S_w = 0.15\, d_b - \frac{g^{0.5} \left(H_{o'}\right)^2 T}{64\, \pi\, d_b^{0.66}} \tag{36.37}$$

where d_b = the depth of breaking
$H_{o'}$ = the unrefracted deep water wave height

Wave setup is typically of the order of centimeters.

Storm Surge

Storm conditions often produce major changes in water level as a result of the interaction of wind and atmospheric pressure on the water surface. Severe storms and hurricanes have produced surge heights in excess of 8 m on the open coast and can produce even higher surges in bays and estuaries. Although prediction of storm surge heights is dependent upon many factors (see The Shore Protection Manual, U.S. Army Corps Of Engineers, 1984), an estimate of sea surface slope at the coast, caused by wind-stress, τ_s, effects, can be calculated as

$$\frac{dz}{dx} = \lambda\, \frac{\tau_s}{\gamma d} \tag{36.38}$$

where λ, an experimentally determined variable, ranges from 0.7 to 1.8 (average value of 1.27) and

$$\tau_s = 0.58\, U_{10}^{2.22} \tag{36.39}$$

where τ_s = kg-m^2
U_{10} = the wind speed in m/s measured at 10 m above the sea surface

A similar simple estimate of pressure setup (P_s) in meters at the storm center for a stationary storm can be made using the relation

$$P_s = 0.0136\, \Delta P \tag{36.40}$$

where ΔP is the difference between the normal pressure and the central pressure of the storm measured in millimeters of mercury.

Climatologic Effects

For most coastal engineering on ocean coasts, climatologic effects on sea level change are small and may be neglected. However, on large lakes such as the Great Lakes water levels may vary tens of centimeters per year and meters per decade. The National Oceanic and Atmospheric Administration provide Great

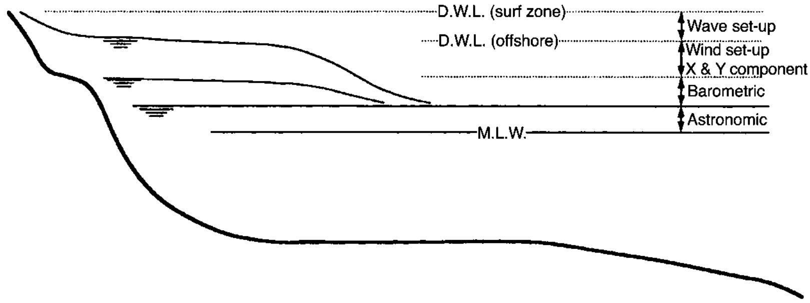

FIGURE 36.8 Schematic of the design water level components.

Lakes water level information monthly and local large lake water level information is usually available from state agencies. Accurate knowledge of sea and lake level change is essential to successful coastal engineering design.

Design Water Level

Design of coastal structures usually requires the determination of a maximum and minimum design water level (D.W.L.). Design water level is computed as an addition of the various water level fluctuation components as follows:

$$D.W.L. = d + A_s + S_w + P_s + W_s \tag{36.41}$$

where d = chart depth referenced to mean low water
A_s = the astronomical tide
S_w = wave set-up defined in Eq. (36.37)
P_s = the pressure set-up as defined in Eq. (36.40)
W_s = the wind set-up calculated using Eq. (36.38)

It is important to recognize that all of these components may not occur in phase and, therefore, Eq. (36.41) will tend to result in extreme maximum and minimum DWL values. It should also be recognized that some of these component effects are amplified at the shore. Figure 36.8 shows a schematic illustration of the design water level components and their relative magnitude.

36.4 Coastal Processes

Coastal region can take a wide variety of forms. Of the total U.S. shoreline, 41% is exposed to open water wave activity the remainder is protected. Outside of Alaska, about 30% is rocky and approximately 14% of the shoreline has beaches. Approximately 24% of the shoreline of the United States is eroding.

Beach Profiles

Nearshore profiles oriented perpendicular to the shoreline have characteristic features, which emulate the influence of local littoral processes. A typical beach profile possesses a sloping nearshore bottom, one or more sand bars, one or more flat beach berms and a bluff or escarpment. As waves move towards shore, they first encounter the beach profile in the form of a sloping nearshore bottom. When waves reach a water depth of approximately 1.3 times the wave height, the wave will break. Breaking results in the dissipation of wave energy by the generation of turbulence and the transport of sediment. As a result, waves suspend sediment and transport it to regions of lower energy, where it is deposited. In many cases, this region is a sand bar. Therefore, the beach profile is constantly adjusting to the incident wave conditions.

The Equilibrium Beach

Although beaches seldom reach a "steady state" profile, it is convenient to consider them as responding to variable incident wave conditions by approaching an "ideal" long-term configuration dependent upon steady incident conditions and sediment characteristics. Bruun (1954) and Dean (1977), in examining natural beach profiles, found that the "typical" profile was well defined by the relationship:

$$d(x) = Ax^q \tag{36.42}$$

where A = a scale coefficient dependent upon the sediment characteristics
q = a shape factor, found both theoretically and experimentally to be approximately 2/3
x = the distance offshore
d = the water depth

It is this concept of the equilibrium profile, which serves as a basis for many models of coastal sediment movement and bathymetric change. When a profile is disturbed from equilibrium by a short-term disturbance, such as storm-induced erosion, the profile is then hypothesized to adjust accordingly towards a new equilibrium state defined by the long-term incident wave conditions and sediment characteristics of the site.

Beach Sediments

Beach sediments range from fine sands to cobbles. The size and character of sediments and the slope of the beach are related to the forces to which the beach is exposed and the type of material available on the coast. The origin of coastal materials can be from inland or offshore sources or from erosion of coastal features. Coastal transport processes and riverine transport bring these materials to the beach and nearshore zone for disbursement through wave and current activity. Beaches arising from erosion of coastal features tend to be composed of inorganic materials while beaches in tropical latitudes can be composed of shell and coral reef fragments. Finer particles are typically kept in suspension in the nearshore zone and transported away from beaches to more quiescent waters such as lagoons and estuaries or deeper offshore regions.

Longshore Currents

As waves approach the shoreline at an oblique angle, a proportion of the wave orbital motion is directed in the longshore direction. This movement gives rise to longshore currents. These currents flow parallel to the shoreline and are restricted primarily between the breaker zone and the shoreline. Longshore current velocities vary considerably across the surf zone, but a typical mean value is approximately 0.3 m/s. Expressions for calculating longshore current velocity range from theoretically based to completely empirical expressions. However, all of the expressions are calibrated using measured field data. Two accepted formulations for calculating mean longshore current velocities $(\bar{V})$ are: the theoretically based formulation of Longuet-Higgins (1970)

$$\bar{V} = 20.7\, m \left(g\, H_b\right)^{1/2} \sin 2\alpha_b \tag{36.43}$$

where m = beach slope
H_b = wave height at breaking
H_b = wave crest angle at breaking
g = gravitational acceleration

and the empirical formulation of Komar and Inman (1970)

$$\bar{V} = 2.7\, u_{\max} \sin \alpha_b \cos \alpha_b \tag{36.44}$$

where u_{max} is the maximum particle velocity at breaking.

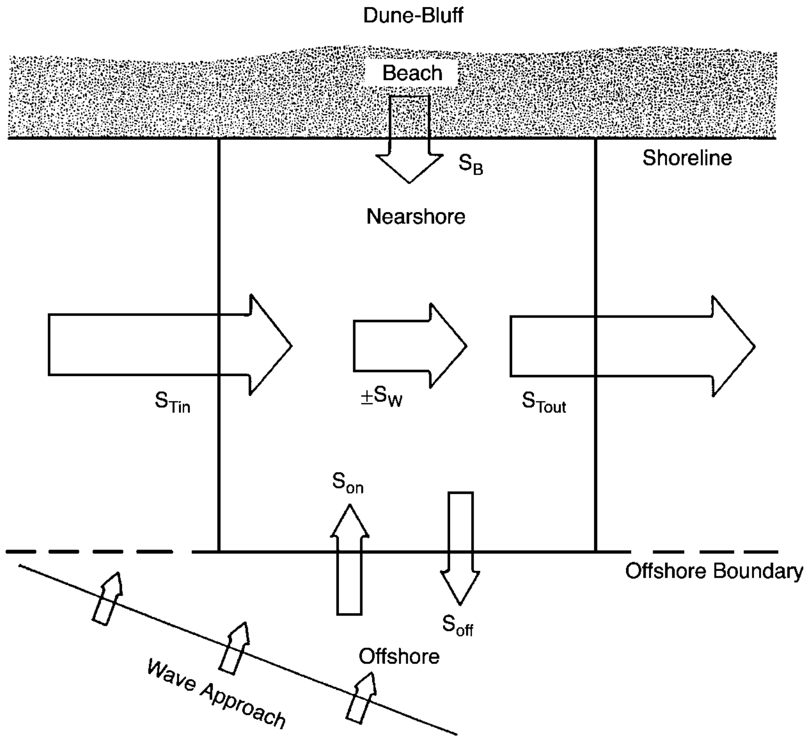

FIGURE 36.9 Box diagram of sediment transport components at the coast.

Cross-shore Currents

Cross-shore currents and resultant transport can be caused by: (a) mass transport in shoaling waves, (b) wind-induced surface drift, (c) wave-induced setup, (d) irregularities on the bottom, (e) density gradients. These factors can produce cross-shore currents ranging in intensity from diffuse return flows, visible as turbid water seaward of the surf zone, to strong, highly organized, rip currents. Rip currents are concentrated flows, which carry water seaward through the breaker zone. There is very little information on the calculation of cross-shore current velocities in the breaker zone.

Sediment Transport

A schematic drawing of the sediment transport (littoral transport) along a coast is shown in Fig. 36.9. This diagram depicts the distribution of sediments for an uninterrupted length (no shoreline structures) of natural coastline. When there is sufficient wave energy, a system of sediment erosion, deposition, and transport is established as shown in Fig. 36.9.

A finite amount of sediment S_{Tin} is transported by longshore currents into a section of the coast (nearshore zone). Wave action at the shore erodes the beach and dune-bluff sediment (S_B) and carries it into the longshore current. This same wave action lifts sediment from the bottom (S_w) for potential transport by the longshore current. Finally, wave action and cross-shore currents at the offshore boundary move sediment onshore or offshore (S_{on} and S_{off}) depending on wave and bottom conditions. This transport at the outer limit of the nearshore zone can usually be assumed negligible with respect to the other transports. The summation of these various transports over time provide a measure of the net sediment budget for a section of coast.

An estimate of the longshore sediment transport rate can be obtained by the energy flux method using deep water wave characteristics applied to calculate the longshore energy flux factor entering the surf zone (P_{ls})

$$P_{ls} = 0.05\,\rho\, g^{3/2}\, H_{so}^{5/2}\left(\cos\alpha_o\right)^{1/4}\sin 2\,\alpha_o \tag{36.45}$$

where ρ = the density of water
H_{so} = the deep water significant wave height
α_o = the deep water wave angle of approach

The actual quantity of sediment transported can be calculated directly from P_{ls} as

$$Q\left(\frac{m^3}{yr}\right) = 1290\, P_{ls}\left(\frac{J}{m-s}\right) \tag{36.46}$$

or

$$Q\left(\frac{yd^3}{yr}\right) = 7500\, P_{ls}\left(\frac{ft-lb}{ft-s}\right). \tag{36.47}$$

These calculations should be carried out using an offshore wave climatology distributed in wave height and angle. A complete description of this methodology is given in Chapter 4 of the The Shore Protection Manual (U.S. Army Corps Of Engineers, 1984).

36.5 Coastal Structures and Design

There are four general categories of coastal engineering problems that may require structural solutions: shoreline stabilization, backshore (dune-bluff) protection, inlet stabilization, and harbor protection (Shore Protection Manual, 1984). Figure 36.10 shows the types of structures or protective works in each of these four coastal engineering problem areas. A listing of factors that should be considered in evaluating

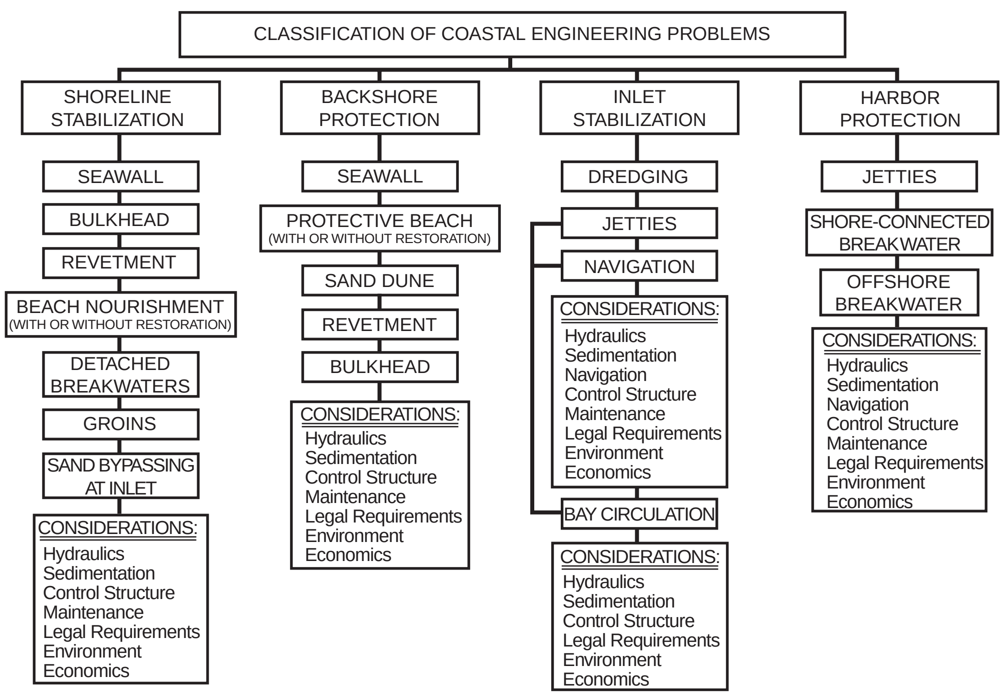

FIGURE 36.10 Classification of coastal engineering problems at the coast (Shore Protection Manual, 1984).

each of these problem areas is also given in Fig. 36.10. Hydraulic considerations include wind, waves, currents, storm surge or wind set-up, water-level variation, and bathymetry. Sedimentation considerations include: sediment classification, distribution properties, and characteristics, direction and rate of littoral transport; *net* versus *gross* littoral transport; and shoreline trend and alignment. Control structure considerations include selection of the protective works with respect to type, use, effectiveness, economics and environmental impact (The Shore Protection Manual, U.S. Army Corps of Engineers, 1984). The other factors listed in Fig. 36.10 are more generally understood and will not be elaborated upon further. It is important to remember that a "no action" alternative should also be considered as a possible solution for any one of these categories of coastal problems.

Structural Selection Criteria

There are a diverse set of criteria that need to be considered in the selection and design of coastal structures. Structural stability criteria and functional performance criteria encompass two areas of primary concern for selection and evaluation of coastal structures.

Structural stability criteria are usually associated with extreme environmental conditions, which may cause severe damage to, or failure of a coastal structure. These stability criteria are, therefore, related to episodic events in the environmental (severe storms, hurricanes, earthquakes) and are often evaluated on the basis of risk of encounter probabilities. A simple method for evaluating the likelihood of encountering an extreme environmental event is to calculate the encounter probability (E_p) as

$$E_p = 1 - \left(1 - \frac{1}{T_R}\right)^{\Lambda} \tag{36.48}$$

where T_R = the return period
Λ = the design life of the structure (see Borgman, 1963)

The greatest limitation to structural stability criteria selection is the need for a long-term data base on critical environmental variables sufficient enough to determine reasonable return periods for extreme events. For example coastal wave data for U.S. coasts is geographically sparse and in most locations where it exists the period of collection is in the order of 10 years. Since most coastal structures have a design life well in excess of 10 years, stability criteria selection often relies on extrapolation of time limited data or statistical modeling of environmental processes.

Functional performance criteria are generally related to the desired effects of a coastal structure. These criteria are usually provided as specifications for design such as the maximum acceptable wave height inside a harbor breakwater system or minimum number of years for the protective lifetime of a beach nourishment fill project. Functional performance criteria are most often subject to compromise because of initial costs.

The U.S. Army Shore Protection Manual (1984) provides a complete discussion of coastal structures, their use, design and limitation. A P-C based support system entitled Automated Coastal Engineering System (ACES) is also available through the USAE Waterways Experiment Station, Coastal and Hydraulics Laboratory, Vicksburg, MS 39180–6199.

Environmental Impacts of Coastal Structures

The placement of engineered structures on or near the coastline must be contemplated with extreme care. In general, alteration of the natural coastline comes with an associated environmental penalty. Hard (structures made of stone, steel, concrete, etc.) or soft (beach nourishment, sediment filled bags, etc.) engineering structures can alter many physical properties of the beach to often induce undesired effects. These alterations of natural processes can take the form of increased reflectivity to incident waves,

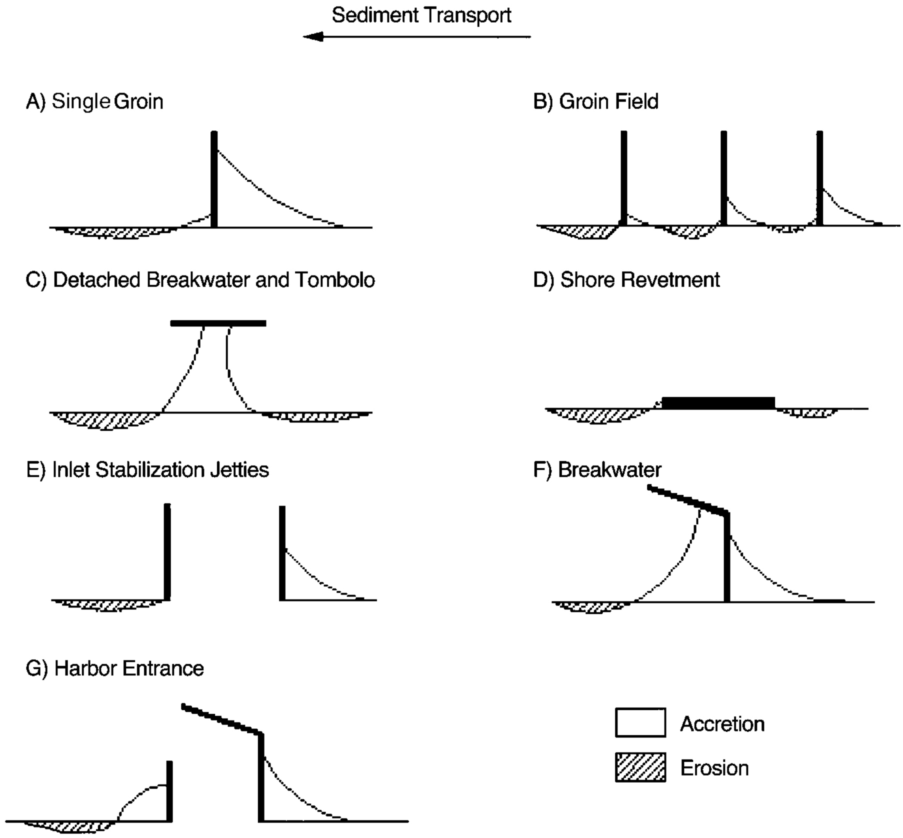

FIGURE 36.11 Classification of "typical" coastal engineering structures and associated regions of shoreline impact.

interruption of longshore currents and resulting longshore sediment transport, alteration of incident wave patterns and the generation of abnormal underwater topography.

Figure 36.11 presents a suite of "typical" coastal engineering structures found along the world's coastlines. These coastal engineering structures range from simple, single component shore protection structures to complex harbor entrance structures. In each case, the primary direction of wave induced, longshore sediment transport is from right to left. These coastal engineering structures increase in scope and complexity, moving down the page. Anticipated regions of shoreline sediment accretion and erosion are also indicated for each type of structure. A significant body of recent research has indicated that these regions of structural impact along the shoreline extend between five and ten times the length of the structure. Hence, for a structure protruding from the undisturbed shoreline a distance of 100 m, the anticipated region of impact should be expected to extend from 500 to 1000 m either side of the structure.

Further Information

The field of coastal engineering is far from a mature science. This is a time of rapid and significant advances in our understanding of the physical processes, which control the response of the nearshore region to wind, waves and water level changes. Furthermore, advances in the design, implementation and in predicting the response of coastal structures and fortifications are made almost daily. Hence, it is nearly impossible to provide a comprehensive review of the most current material.

To further aid the reader, following are some of the most reliable and current, electronic sources of information for the coastal engineer. Within the United States, several government agencies have at least partial responsibility for engineering along the coastline. These include, within the Department of Defense, U.S. Army Corps of Engineers, Coastal and Hydraulics Laboratory, Coastal Engineering Research Center (CERC), (http://chl.wes.army.mil/research/centers/cerc/) and within the Department of Commerce, the National Oceanic and Atmospheric Administration's, Office of Oceanic and Atmospheric Research (http://www.oar.noaa.gov/) and the Office of the National Ocean Service (http://www.nos.noaa.gov/). CERC has published the Automated Coastal Engineering System (ACES, 1992), which provides an integrated software package to solve a variety of coastal engineering problems.

In addition, within the public sector, most major coastal and Great Lakes Universities offer advanced study in Coastal Engineering. Published studies can be found, for example, in the Journal of Waterways, Ports, Coastal and Ocean Engineering, of the American Society of Civil Engineers (ASCE) (http://www.pubs.asce.org/journals/ww.html) and in the Coastal Engineering Journal. Professor Dalrymple of the University of Delaware has provided a very useful and fully tested set of coastal engineering applications for common use. These may be found at: Dalrymple's Coastal Engineering Java Page (http://rad.coastal.udel.edu/faculty/rad/). Similarly, the Coastal Guide provides a quick reference to coastal engineering software and can be found at: (http://www.coastal-guide.com/). Finally, a simple web search of the key words "coastal engineering" will provide the reader with several hundred relevant sources of current information.

Defining Terms

Celerity — Speed that a wave appears to travel relative to its background.

Fetch — The distance over the water that wind transfers energy to the water surface.

Group speed — Speed of energy propagation in a wave.

Neap tide — A tide occurring when the sun and moon are at right angles relative to the earth. A period of minimal tidal range.

Plunging breaker — A wave that breaks by curling over and forming a large air pocket, usually with a violet explosion of water and air.

Sea — A term used to describe an irregular wind generated wave surface made up of multiple frequencies.

Seiche — A stationary oscillation within an enclosed body of water initially caused by external forcing of wind or pressure.

Significant wave height (H_s) — The average height of the one-third highest waves in a wave height distribution.

Spilling breaker — A wave that breaks by gradually entraining air along the leading face and slowly decays as it moves shoreward.

Spring tide — A tide occurring when the sun and moon are inline relative to the earth. A period of maximum tidal range.

Surging breaker — A wave that breaks by surging up the beach face instead of breaking in the conventional manner.

Swell — A term used to describe a very regular series of wind generated surface waves made up of a single dominant frequency.

Tsunami — A long period, freely traveling wave usually caused by a violent disturbance such as an underwater earthquake, volcanic eruption, or landslide.

Wave diffraction — The spread of energy laterally along a wave crest usually due to the interaction of a wave with a barrier.

Wave height — The vertical distance from a wave crest to the trough of the preceding wave.

Wave refraction — The bending of a wave crest as it enters shallow-water caused by a differential in speed between the deeper and shallower portions of the crest.

Wave ray — A line drawn perpendicular to a wave crest indicating the direction of propagation of the wave.

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