Chapter 1

Introduction

1.1 Waves in the Sea

Engineers build various types of maritime structures. Breakwaters and quaywalls for ports and harbors, seawalls and jetties for shore protection, and platforms and rigs for the exploitation of oil beneath the seabed are some examples. These structures must perform their functions in the natural environment, being subjected to the hostile actions of winds, waves, tidal currents, earthquakes, etc. To ensure their designated performance, we must carry out comprehensive investigations in order to understand the environmental conditions. The investigations must be as accurate as possible so that we can rationally assess the effects of the environment on our structures.

Waves are the most important phenomenon to be considered among the environmental conditions affecting maritime structures, because they exercise the greatest influence. The presence of waves makes the design procedure for maritime structures quite different from that of structures on land. Since waves are one of the most complex and changeable phenomena in nature, it is not easy to achieve a full understanding of their fundamental character and behavior.

Waves have many aspects. They appear as the wind starts to blow, grow into mountainous waves amid storms and completely disappear after the wind ceases blowing. Such changeability is one aspect of the waves. An observer on a boat in the offshore region easily recognizes the pattern of wave forms as being made up of large and small waves moving in many directions. The irregularity of wave form is an important feature of waves in the sea. However, upon reaching the shore, an undulating swell breaks as individual waves, giving the impression of a regular repetition. Yuzo

Yamamoto, in his novel *Waves*, sees an analogy between successive waves and a son's succession to the father.

The generation of waves on a water surface by wind and their resultant propagation has been observed throughout history.[a] However, the mathematical formulation of the motion of water waves was only introduced in the 19th century. In 1802, Gerstner, a mathematician in Prague, published the trochoidal wave theory for waves in deep water, and in 1844, Airy in England developed a small amplitude wave theory covering the full range of water depth from deep to shallow water. Thereafter, in 1847, Stokes gave a theory of finite amplitude waves in deep water, which was later extended to waves in intermediate-depth water. This solution is now known as the Stokes wave theory. The existence of a solitary wave which has a single crest and propagates without change of form in shallow water was reported by Russell in 1844. Its theoretical description was given by Boussinesq in 1871 and Rayleigh in 1876. Later, in 1895, Korteweg and de Vries derived a theory of permanent periodic waves of finite amplitude in shallow water. This is now known as the cnoidal wave theory.

Thus, the fundamental theories of water waves were established by the end of the 19th century. Nevertheless, several decades had passed before civil engineers were able to make full use of these theories in engineering applications. An exception is the theory of standing wave pressure derived in 1928 by Sainflou,[2] an engineer at Marseille Port. Sainflou's work attracted the attention of harbor engineers soon after publication; his pressure formula was adopted in many countries for the design of vertical breakwaters. It should be mentioned, however, that it was during the Second World War when the mathematical theory and engineering practice was successfully combined together. This led to the formation of the discipline of coastal engineering, which can be said to have begun with the wave forecasting method introduced by Sverdrup and Munk;[3] this later evolved into the more sophisticated S–M–B method, the calculation of wave diffraction by a breakwater developed by Penney and Price,[4] and other milestone developments.

In proposing the foundation for the present S–M–B method, Sverdrup and Munk clearly understood that sea waves are composed of large and small waves. They introduced the concept of the *significant wave*, the height of which is equal to the mean of the heights of the highest one-third waves in a wave group, as representative of a particular sea state. Therefore, the

[a]The following historical overview of the study of water waves is based on the literature listed by Lamb.[1]

significant wave concept was based upon the understanding of sea waves as a random process. However, the significant wave, expressed in terms of a single wave height and wave period, is sometimes misunderstood by engineers to represent waves of constant height and period. The theory of monochromatic waves and experimental results obtained from a train of regular waves have been directly applied to prototype problems in the real sea on the belief that the regular waves correspond exactly to the significant wave.

As early as in 1952, a group of American oceanographers, headed by Pierson,[5] took the first step in recognizing the irregularity of ocean waves as a fundamental property and incorporating this fact in the design process. The so-called P–N–J method[6] of wave forecasting, often compared with the S–M–B method, introduced the concept of wave spectrum as the basic tool for describing wave irregularity. The generation and development of wind waves, the propagation of swell and wave transformation near the shore were all explained in detail via the concept of wave spectrum. Although the spectral concept was readily accepted by oceanographers at an early stage, coastal and harbor engineers with the exception of a few researchers considered it too complicated. Hence, the introduction of spectral computation techniques into the design process for coastal structures was much delayed. The situation began to change gradually since the 1970s. In the next subsection, an overview is given on the development of random wave concept and its engineering applications in the field of coastal engineering, which are discussed in detail in the chapters to follow in the present book.

1.2 Overview of Historical Development of Random Wave Applications

(A) *Statistical description of random seas*

At the early stage of coastal engineering, sea waves were measured with pressure gauges mounted on the seabed. The amplitudes of individual oscillations of wave pressure were converted to surface amplitudes with the small amplitude wave theory by using the respective wave periods. Statistical distributions of wave heights and periods thus obtained were investigated eagerly. For example, Putz[7] reported in 1952 that the mean ratio of the significant to the mean wave heights is 1.57, that of the one-tenth to the significant wave height is 1.29, and the wave height distribution can be approximated with the Pearson III type distribution. In the same year, however, Longuet-Higgins[8] verified the applicability of the Rayleigh distribution to sea waves and presented the theoretical relationships among various wave heights. Since then, the Rayleigh distribution of wave heights has been widely accepted.

The basic assumption of the Rayleigh distribution is that the wave spectrum is confined in a narrow frequency band. The spectrum of sea waves is broad-banded as evidenced through many field observations, however, and the question on the wave height distribution of broad-band spectrum arose soon. Most of field data yielded the wave height distribution close to the Rayleigh so long as individual waves are defined with the zero-upcrossing or zero-downcrossing method regardless of the spectral shape. The first approach to the question of wave height distribution under broad-band spectra was made by the author[9] in 1970 by means of the numerical simulation of wave profiles under various spectral shape. The simulation work confirmed the above finding through field observations.

The distribution of wave period is discussed in the form of the joint distribution with the wave height; the marginal distribution of wave period alone excites little interest from the practical viewpoint. Longuet-Higgins[10] presented the first theory of the joint distribution of wave height and period in 1975, followed by several researchers. Though the period distribution is affected by the spectral shape, the periods of individual large waves are clustered around the significant wave period.

(B) *Wave transformation analysis with wave spectrum*

Until the late 1960s, coastal engineers used to analyze wave transformations and actions by means of the regular waves, the height and period of which are equal to the significant wave height and period, respectively. Waves are thought to come from a single direction. Computer drawing of wave rays for wave refraction analysis was quite common. A breakthrough was made by Karlsson[11] in 1969, who introduced a numerical computation method for the spectral wave shoaling and refraction. The method was an application of the spectral wave forecasting method of the first generation, which had began to replace the S–M–B method in the late 1960s.

Karlsson's method, which is based on the energy balance equation, was immediately applied for an actual harbor planning by Nagai *et al.*[12] in 1974 and the method soon became a design tool among Japanese coastal engineers. Contrarily, American and European engineers did not seem to appreciate the spectral method at that time, probably because of unfamiliarity with the concept of directional wave spectrum. Measurement and analysis of directional wave spectra had been promoted by oceanographers since the late 1950s, and coastal engineers joined in the efforts from the 1970s.

In 1975, Mitsuyasu *et al.*[13] proposed a formulation of the directional spreading function based on a number of detailed ocean measurements. Goda and Suzuki[14] immediately adopted it as the standard functional form of the directional spectrum for engineering applications under the name of the Mitsuyasu-type directional spreading function. They made use of this directional spectrum for computing and presenting a full set of directional random diffraction diagrams. Goda *et al.*[15] presented several sets of these diagrams at the 16th International Conference on Coastal Engineering in 1978, but it did not awake the response of the audience.

The paper on the laboratory measurements of the refraction and diffraction of directional waves over an elliptical shoal by Vincent and Briggs[16] in 1989 seems to have arisen the interests of American and European engineers on directional spectral waves. A number of numerical schemes for directional spectral wave transformations have been presented since then. Holthuijsen *et al.*[17] developed the SWAN model in 1993 for the transformation of directional random waves in presence of currents. The model is a shallow water version of the spectral wave forecasting model of the third generation, WAM.

(C) *Random wave breaking and surf zone hydrodynamics*

While regular waves in a laboratory flume break at a fixed location with almost the same height, random waves break at various locations over quite a distance, which is called the surf zone. At a beach having sand bars offshore, the outer bar usually defines the edge of the surf zone. Otherwise, an observer needs to define the area of the surf zone somewhat subjectively.

Quantitative evaluation of wave decay within the surf zone was initiated by Collins[18] in 1970, who truncated the Rayleigh distribution of wave height beyond the height exceeding the breaking limit of regular waves. Truncation of the wave height distribution yielded a decrease of the total wave energy, thus approximating the wave decay process within the surf zone. Several modifications of this model followed, including the author's model[19] in 1975 to be discussed in Sec. 3.6.

In 1978, Battjes and Janssen[20] proposed another modeling approach with estimation of the percentage of breaking waves and evaluation of energy dissipation with the analogy of a bore in hydraulic jump. Though predictability of the model is limited to the variation of the root-mean-square wave height only, it can be applied to beaches of arbitrary bathymetry and thus it has been used as an engineering tool by European engineers.

The third approach to wave energy dissipation by breaking was proposed by Dally *et al.*[21] in 1985, who assumed the rate of energy dissipation being proportional to the difference between the energy level of the local waves and that of the stable waves after breaking when the former exceeds the latter. The model was initially developed for regular waves, but it has been applied to individual random waves by several researchers. One of such models will be presented in Sec. 12.4.

The surf zone is characterized with the spatial variation of local mean water level, called the *wave setup* and *wave setdown*, and the generation of nearshore currents, both of which are induced by the spatial gradients of radiation stresses associated with the wave momentum fluxes. Different behaviors of regular and random waves with respect to wave breaking bring forth quite different results of the local mean water level and nearshore currents. Although Battjes[22] demonstrated such differences in 1972, the majority of coastal engineers have been late in recognizing the difference. Even in the 2000s, there are some papers dealing with wave setup and nearshore currents induced by regular waves. This aspect will be discussed in Sec. 12.3.

(D) *Wave actions on structures*

It has been the established practice that offshore structures for oil exploitation are designed against the wave of maximum height among individual waves, because a single action of the largest wave loading may causes collapse of a structure. Composite breakwaters with upright sections are also designed against the maximum wave. The author's wave pressure formulas[23,24] presented in 1973 has been utilized for composite breakwater designs in Japan since the late 1970s. Interests in caisson breakwaters were arisen in Europe after the collapse of a deepwater mound breakwater at Sines Port in 1978, and a number of joint research projects on the design methodology of caisson breakwaters have been carried out under the EU science and technology sponsorship. Goda's formulas have served as a guiding reference in these studies.

Before the late 1970s, the stability of armor units of mound breakwaters had been tested by using regular waves. A question of how to convert the regular wave height in the tests to the representative height of random waves such as the significant height or the highest one-tenth height was hotly debated by researchers and practitioners. Wide spread of irregular wave machines in laboratory flumes since the late 1970s changed the situation by making irregular wave tests as the standard procedure for breakwater stability analysis. The test results are now presented in terms of the significant wave height. Sometimes the 2% exceedance wave height is used to describe the test results when there is deviation in the wave height distribution from the Rayleigh.

The crest elevation of a coastal dike is often designed such that it is equal to or higher than the 2% exceedance level of wave run-up. The practice was introduced by Dutch engineers when they designed the reclamation dike of the North East Polder in 1936 within the Ijsselmer that was created by closing the Zeidel Sea in 1932.[25] Use of the 2% run-up height for coastal dike design seems to be common in European countries.

Coastal dikes and seawalls are also designed against the mean discharge of overtopping under the design wave and water level conditions. In 1968, Tsuruta and Goda[26] introduced a concept of the expected wave overtopping rate, which synthesized the data of regular wave overtopping rate by averaging them with the weight of the probability density function of wave heights under the Rayleigh distribution. Then, Goda *et al.*[27] presented the design diagrams of the expected wave overtopping rate of vertical seawalls and sloped seawalls with concrete block mounds based on series of irregular wave tests in 1975. They were published in Japanese and remained unknown outside Japan until the publication of the first edition of the present book in 1985. In UK, Owen[28] presented the first overtopping formula in 1980 based on irregular wave tests. Since then, many studies on irregular wave overtopping have been carried out in various countries.

(E) *Methodology of laboratory tests using random waves*

Reproduction of random waves in laboratory basins requires a wave generator of the servo-control system that drives the wave paddle according to the control signal. Such irregular wave machines were first introduced in ship testing tanks in the early 1960s, and soon hydraulic laboratories in UK and Denmark equipped their test basins with such machines for harbor tranquility tests. Hydraulic laboratories in other countries also followed the trend, and irregular wave tests gradually became a standard procedure in coastal engineering studies.

One of the early problems encountered in irregular wave tests was the multireflection of waves between a model under test and the wave paddle. A regular wave test can be carried out by finishing measurements before the first wave reflected by the model and re-reflected by the paddle would reach the model again. An irregular wave test must take measurements of a few hundred waves and thus has to accept the multireflected waves in the measurement records. A solution was given by Goda and Suzuki[29] in 1976, who introduced the algorithm for resolving the incident and reflected waves from the wave records at two neighboring stations. The method was immediately accepted as one of the standard test procedures in hydraulic laboratories in the world.

Laboratory generation of multidirectional random waves was initiated by Salter[30] in the late 1970s, when he worked for development of a wave power device made of rotational floats. Many hydraulic laboratories adopted Salter's concept and developed their own multidirectional random wave generators for their test basins. By the early 1990s, such devices became the standard laboratory equipment in many countries. Advancement of random wave studies and progress in reliable design methods against random seas owe to the improvement in laboratory testing methods with random wave generators since the 1970s.

1.3 Outline of Design Procedures Against Random Sea Waves

1.3.1 *Wave Transformation*

A prerequisite for the reliable estimation of wave actions on maritime structures is a detailed understanding of how waves transform during their propagation toward the shore, after they have been generated and developed by the wind in the offshore region. The various types of wave transformations are schematically shown in Fig. 1.1.[31]

First, wind waves become swell when they move out of the generating area. The height of the swell gradually decreases with distance as it propagates (2 in Fig. 1.1). When wind waves and swell encounter an island or a headland during their propagation in deep water, they are diffracted and penetrate into the area behind the obstacle (4 in Fig. 1.1).

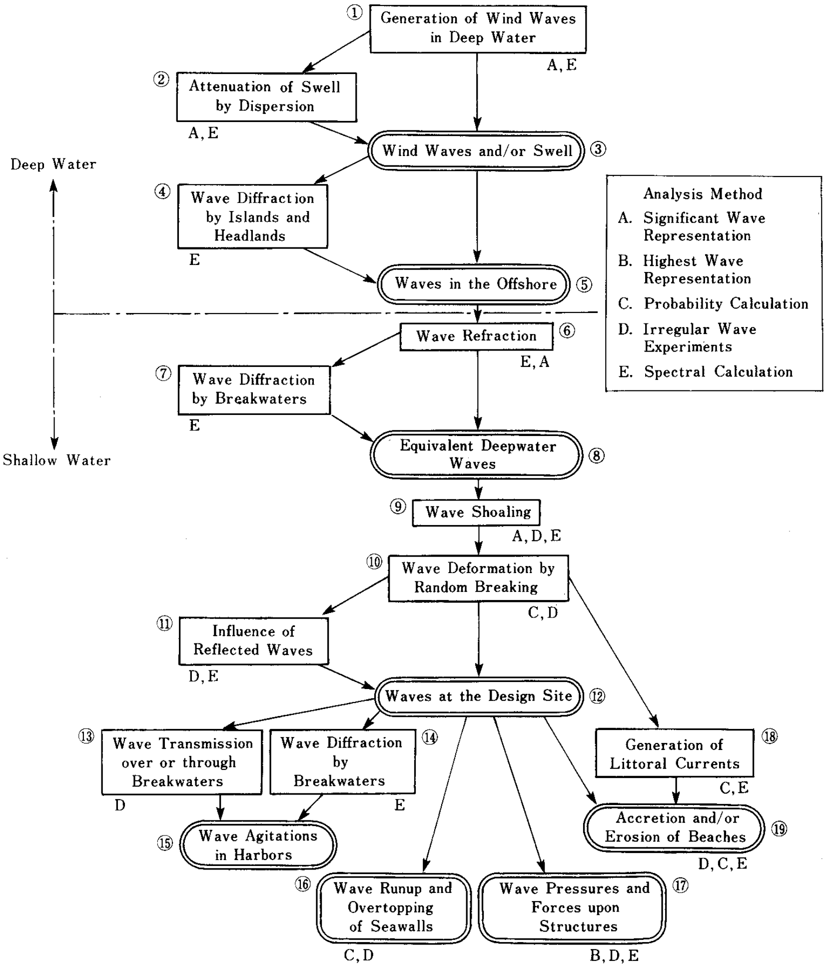

Fig. 1.1 Flow of the transformations and actions of sea waves with suggested methods for their calculation.[7]

When waves enter an area of depth less than about one-half of their wavelength, they are influenced by the seabed topography. These waves are called *intermediate-depth water waves*. Where the water is shallower than about one-twentieth of a wavelength, the waves are called *long waves*. For the sake of simplicity, both intermediate-depth water waves and long waves may be classified together and called *shallow water waves*; this terminology is employed in the present chapter. Waves in an area having a depth greater than about one-half of a wavelength are called *deepwater waves* (5 in Fig. 1.1). In a shallow bay or estuary, wind-generated waves may become shallow water waves during their process of development. In such a case, wave forecasting or hindcasting needs to take into account the effect of water depth on the wave development.

Having propagated into a shallow region, waves undergo *refraction* by which the direction of wave propagation, as well as the wave height, varies according to the seabed topography (6 in Fig. 1.1). Concerning waves inside a harbor, the phenomenon of *diffraction* by breakwaters is the governing process (7 in Fig. 1.1). Although not shown in Fig. 1.1, wave attenuation due to bottom friction and other factors may not be neglected in an area of relatively shallow water which extends over a great distance with a very gentle inclination in the sea bottom. For convenience, the change in wave height due to wave refraction, diffraction and attenuation is often incorporated into the concept of *equivalent deepwater waves* (8 in Fig. 1.1). Equivalent deepwater waves are assigned the same wave period as deepwater waves, but the wave height is adjusted to account for the spatial change due to wave refraction, diffraction and attenuation.

Waves propagating in a shallow region gradually change their height as a result of the change in the rate of energy flux due to the reduction in water depth, even if no refraction takes place. This is the phenomenon of *wave shoaling* (9 in Fig. 1.1). When waves reach an area of depth less than a few times the significant wave height, waves of greater height in a wave train begin to break one by one and the overall wave height decreases as the wave energy is dissipated. This is the wave deformation caused by *random wave breaking* (10 in Fig. 1.1).

Waves arriving at the site of a proposed structure (12 in Fig. 1.1) will experience the above transformations and deformations of wave refraction, diffraction, shoaling, breaking, etc. If a long extension of vertical breakwater is already located in the adjacent water area, or if the design site is within a harbor, the influence of waves reflected from neighboring structures must be added to that of the waves arriving directly from offshore (11 in Fig. 1.1). These wave transformations and deformations will be discussed in detail in Chapter 3. Once the characteristics of the waves at the site have been estimated, design calculations can be made according to the nature of the problem. For example, the problem of harbor tranquility requires the analysis of waves transmitted over or through breakwaters (13 in Fig. 1.1), waves diffracted through harbor entrances (14 in Fig. 1.1) and waves reflected within a harbor. These will be discussed in Chapter 7.

The planning of seawalls and coastal dikes to stand against storm waves requires the estimation of wave run-up and wave overtopping rate. Technical information for such problems is usually provided by conducting a scale model test in the laboratory or a set of design diagrams based on laboratory data. In such cases, the flow of calculation takes a jump from 8 to 16 in Fig. 1.1, because these data are prepared using the parameter of equivalent deepwater waves by directly incorporating the effects of transformations 9 and 10 into the data. Problems related to wave run-up and overtopping are discussed in Chapter 5.

In the design of breakwaters, the magnitude of the wave pressure is the focal point that requires an appropriate selection of calculation formulas. Chapter 4 discusses the formulas for wave pressure and their applications. It is remarked here that littoral drift, which is associated with beach erosion and accretion, is closely related to wave deformation by breaking and is induced by the resultant longshore currents (18 in Fig. 1.1). Prediction of longshore currents is discussed in Chapter 12, while beach morphology problems are dealt with in Chapters 14 and 15.

In the process of calculating the wave transformations and deformations described above, the significant wave height and period are used as indices of the magnitude of random sea waves. These parameters are converted to those of the highest waves or other descriptive waves whenever necessary, as in the case of a wave pressure calculation. Thus, at first sight, this procedure may look the same as the conventional design procedure, for which the significant wave is regarded as a train of regular waves. In the present treatise, however, the effects of wave randomness are accounted for the estimation of the respective wave transformations, and the resultant height of the significant wave after such transformation often takes a value considerably different from that obtained with the regular wave approach.

1.3.2 *Methods of Dealing with Random Sea Waves*

At present, the following five methods are available to deal with the transformation and action of random sea waves:

(A) Significant wave representation method
(B) Highest wave representation method

(C) Probability calculation method
(D) Irregular wave test method
(E) Spectral calculation method

The *significant wave representation method* makes use of a train of regular waves with height and period equal to those of the significant wave as representative of random sea waves. Transformations of sea waves are estimated with the data of regular waves on the basis of theoretical calculation and/or laboratory experiments. The method had has widely been employed in the field of coastal engineering since the introduction of the significant waves as the basis of the S–M–B method for wave forecasting. It has the merits of easy understanding and simple application, but it also has the demerit of containing a possibly large estimation error, depending on the type of wave phenomenon being analyzed. Diffraction, to be discussed in Sec. 3.3, is an example: the wave height behind a single breakwater may be estimated to be less than a third of the actual height if the diffraction diagram of regular waves is directly applied. The design of a steel structure in the sea is another example in which the maximum force exerted by individual waves is the governing factor. If the structure is designed against a regular wave equal to the significant wave, the structure will most probably fail under the attack of waves higher than the significant wave height when the design storm hits the site.

The danger of underestimating wave forces through the use of significant waves was well understood at the early stages of construction of offshore structures such as oil drilling rigs. It is an established practice to use a train of regular waves of height and period equal to those of the highest wave, and to design structures against this train of regular waves. This is called the *highest wave representation method* herein. The method is mainly used for structural designs.

In contrast to the above examples, the phenomenon of diffraction is quite sensitive to the characteristics of the wave spectra, especially to the directional spreading of wave energy. In the analysis of diffraction, refraction and wave forces upon a large isolated structure such as oil storage tank in the sea, calculation is made for individual components of the directional spectrum. The resultant total effect is estimated by summing the contributions from all spectral components. This is called the *spectral calculation method*.

The rate of wave overtopping of a seawall and the sliding of a concrete caisson of a vertical breakwater differ from the previous examples in the sense that the cumulative effect of the action of individual waves of random nature is important. The probability distribution of individual wave heights and periods is the governing factor in this type of problems. The phenomena of irregular wave run-up and wave deformation by random breaking belong to the same category. The calculation of these cumulative wave effects may be called the *probability calculation method*.

If a large wind–wave flume or a wave basin with a random wave generator is available, wave transformations and wave action on structures can be directly investigated by using simulated random water waves. This is the *irregular wave test method*. When the first edition of this book was published in 1985, the reproduction of directional random waves in model basins was possible only at a limited number of hydraulic laboratories in the world. As described in Sec. 1.2(E), many laboratories are now equipped with multidirectional random wave generators and become capable of carrying out model tests of various problems, including random wave refraction and diffraction. It may be said that the majority of hydraulic model tests related to sea waves are presently carried out with random waves, and wave tests with regular waves are mostly reserved for fundamental research purposes.

In Fig. 1.1, the symbols A to E indicate the analysis method appropriate to the respective phenomenon. As can be seen, the problems related to random sea waves must be solved by selecting the appropriate calculation method among the five, A to E, to obtain a safe and rational design. None of the five methods can be used alone to treat all problems concerning sea waves. This stems from the complicated nature of waves in the real sea. In the following chapters, the above methods of analyzing the various wave phenomena are presented and discussed.

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