# Wave Runup and Overtopping on Beaches and Coastal Structures

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# WAVE RUNUP AND OVERTOPPING ON BEACHES AND COASTAL STRUCTURES

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**Abstract**

Wave runup and overtopping on inclined coastal structures and wave runup on beaches are reviewed together to examine the ranges of wave runup processes occurring on slopes of different inclinations. Laboratory experiments on regular wave runup and overtopping on coastal structures are reviewed first to provide historical perspective. More recent laboratory experiments on irregular wave runup and overtopping on coastal structures are summarized to show the improved quantitative understanding due to the improved capabilities for irregular wave experiments. Field experiments on wave runup on beaches are then reviewed to discuss the possible dominance and causes of low-frequency shoreline oscillations on gently sloping beaches. The recent development of time-dependent numerical models is reviewed to indicate the rapid progress of the numerical capabilities of predicting irregular wave runup on inclined coastal structures and beaches. This review indicates that the improved quantitative understanding of irregular wave runup and overtopping on inclined coastal structures and irregular wave runup on beaches has essentially been limited to normally incident waves on coastal structures and beaches of alongshore uniformity. Future experimental and numerical studies are suggested in this review.

## INTRODUCTION

The population of the world is concentrated near coasts. Tropical and extratropical storms can cause severe damage due to extremely high wind, storm surge and waves. For example, Hurricane Hugo in 1989 caused damage exceeding 7 billion dollars on the U.S. mainland where more than 15,000 homes were destroyed and over 40 lives lost (Finkl and Pilkey 1991). Hurricane Opal in 1995 caused damage mostly in the form of storm surge, wave attack and overwash in contrast to Hurricane Andrew in 1992 whose principal agent of destruction was wind (Webb et al. 1997). Furthermore, there is considerable public concern over beach erosion because most developed beaches are experiencing long-term erosional trends (National Research Council 1990). Storm damage and beach erosion will accelerate if the mean sea-level rise increases due to the greenhouse effect (National Research Council 1987). On the other hand, tsunamis generated by submarine earthquakes and landslides can cause severe coastal damage and loss of life (e.g., Wiegel and Saville 1996).

In the U.S., the Army Corps of Engineers' shore protection program covers only 8 percent of the nation's 4,300 km of critically eroding shoreline and has shifted from primarily coastal structures to primarily beach restoration and nourishment through placement of sand (Hillyer et al. 1997). This program has also shifted from primarily recreation oriented to one of protection for storm damage reduction. The performance of beach nourishment and protection projects is presently predicted by extrapolating historical shoreline changes because

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there is no satisfactory model available for predicting complex nearshore waves, circulation and sediment transport processes (National Research Council 1995). On the other hand, the maintenance and repair of coastal structures such as jetties and breakwaters is an important element in the nation's rehabilitation of deteriorating infrastructure. Most of these structures in the U.S. are constructed of locally available stone and exposed to depth-limited breaking design waves. Jetties constructed to stabilize navigation inlets and breakwaters constructed to protect harbors have often interrupted sand drift and caused downdrift erosion (e.g., Dean 1987). This review paper deals with inclined structures, whereas vertical structures are more common in Japan as discussed by Goda (1985).

Wave runup is normally defined as the upper limit of wave uprush above the still water level in the field of coastal engineering. Wave runup on a beach determines the landward boundary of the area affected by wave action. Wave runup is hence important in delineating the area affected by storm waves and tsunamis. A quantitative understanding of the swash dynamics associated with wave uprush and downrush is also essential for predicting sediment transport in the swash zone and establishing the landward boundary conditions of beach and dune erosion models (e.g., Kriebel 1990). Moreover, field and laboratory measurements indicate that the longshore sediment transport rate in the swash zone can be as large as that in the breaker zone (Bodge and Dean 1987; Kamphuis 1991). On the other hand, wave overtopping of dunes causes landward sediment transport due to overwash (Kobayashi et al. 1996).

The prediction of wave runup on a coastal structure is necessary in determining the crest height of the structure required for no overtopping of design waves (e.g., Shore Protection Manual 1984). Wave uprush and downrush on the seaward slope of the structure affect the wave forces acting on armor units and the stability and movement of armor units (e.g., Kobayashi and Otta 1987). If wave overtopping is allowed, the stability of armor units on the crest and landward side of the structure needs to be examined as well (Vidal et al. 1992). Furthermore, the amount of wave overtopping determines the severity of flooding landward of a structure protecting a shoreline (e.g., Kobayashi and Reece 1983). For a structure protecting a harbor, wave overtopping affects wave transmission landward of the structure (Seelig 1980).

This paper reviews our understanding of wave runup and overtopping on both beaches and coastal structures where coastal structures generally have steeper and rougher slopes than beaches. Wave runup and overtopping on coastal structures have traditionally been investigated by coastal engineers in laboratories, whereas wave runup on beaches has been studied by nearshore oceanographers using field measurements. On the other hand, solitary wave runup has been studied in relation to coastal flooding and damages caused by tsunamis, which are transient waves with much shorter durations than wind waves. Solitary wave runup is also reviewed here to examine the effect of the incident wave duration on wave runup.

Storm surge and tides are important in determining the still water level in the absence of waves. The numerical modeling of storm surge for the estimated atmospheric pressure and wind field has matured considerably over the past 30 years as reviewed by Bode and Hardy (1997). Storm surge models generally neglect wave effects such as wave setup and wave-induced currents. Coupled surge-wave models on a continental shelf are being developed to include wave effects on wind and bottom shear stresses acting on currents (e.g., Mastenbroek et al. 1993). For example, wind waves increase the bottom shear stress felt by wind-induced currents through nonlinear interactions in the bottom boundary layer (Grant and Madsen 1986). To extend such a coupled surge-wave model into very shallow water, wave breaking, wave setup and wave-induced currents will need to be taken into account.

Concurrently, third-generation wave prediction models for arbitrary directional random waves on a continental shelf have been developed and implemented over the last decade as summarized by Komen et al. (1994). Van Veldder et al. (1994) and Booij et al. (1996) attempted to extend full spectral third-generation wave prediction models into shallower water by including the triad wave-wave interactions and surf breaking. The triad interactions of waves traveling in different directions were observed to be important for directional spectra of shoaling waves on a natural beach (Freilich et al. 1990) and in a laboratory (Elgar et al. 1993). The spectral energy dissipation rate as a function of frequency is not well understood for breaking waves in the surf zone and different empirical formulas have been proposed (Mase and Kirby 1992; Eldeberky and Battjes 1996; Elgar et al. 1997). Furthermore, these spectral models neglect wave reflection and do not predict wave setup and runup on beaches.

In the following, the still water level in the absence of waves and the incident waves in relatively shallow water outside the surf zone are assumed to be known, although the existing numerical models may not be able to predict these quantities within errors and uncertainties of about 10%.

This review paper is organized as follows. First, experimental studies on wave runup and overtopping are reviewed because most of the existing knowledge is based on laboratory experiments and field observations. Second, various numerical models for predicting wave runup are reviewed to indicate the capabilities and limitations of available models based on different assumptions. Third, the time-dependent numerical model based on the finite-amplitude, shallow-water equations is explained in more detail because it is the simplest model among the available numerical models and has already been compared with various laboratory and field data. Finally, conclusions and recommendations are given to summarize past progress and suggest future research directions.

## LABORATORY AND FIELD EXPERIMENTS

Wave runup and overtopping on coastal structures have historically been investigated using hydraulic models in laboratories probably because storm waves occur infrequently and field measurements are expensive and difficult during storms. On the other hand, wave runup on beaches has mostly been studied on natural beaches partly because of the difficulties in simulating incident low frequency waves and reproducing actual beach profiles in laboratories. Most laboratory experiments were conducted in wave flumes for normally incident waves on straight structures and beaches. Experiments in directional wave basins are becoming more common throughout the world. Wave runup on a beach was normally measured along a single cross-shore line normal to a long straight shoreline. However, the field experiment DUCK94 has revealed that the nearshore dynamics is far less uniform alongshore than had previously been assumed (Birkemeier and Hathaway 1996). This may also be true for the wave dynamics on coastal structures. The alongshore variability is not addressed in this review paper for lack of data. In the following, normally incident waves on straight structures and beaches are assumed unless stated otherwise.

### Wave Runup and Overtopping on Coastal Structures

Early laboratory experiments were conducted using solitary waves (Hall and Watts 1953) and monochromatic waves (Saville 1955). This historical background of the early U.S. experiments was given by Wiegel and Saville (1996). Various data on monochromatic wave runup and overtopping were later summarized in TAW (1974), Stoa (1978), Shore Protection Manual (1984), and Bruun (1985). The runup height is generally normalized by the wave height because the runup and wave height are on the same order of magnitude. The normalized runup depends on many dimensionless parameters including the seaward slope angle, wave steepness, normalized toe depth, slope roughness and permeability. As a result, many figures were required to present various data sets.

Attempts to develop simple empirical formulas and theories were generally limited to smooth uniform slopes in relatively deep water so that the normalized runup could be assumed to depend on the slope angle and wave steepness only. Moreover, the critical wave steepness for the onset of wave breaking may be assumed to be expressed in terms of the slope angle. The need for a criterion for wave breaking is obvious because potential flow theories with no energy dissipation may be applied to nonbreaking waves only. The relationship proposed by Iribarren and Nogales (1949) is still widely used to estimate whether waves break on the slope or not. Galvin (1968) separated the gradual transition of nonbreaking to breaking waves into surging, collapsing, plunging and spilling breakers. On the other hand, Miche (1951) hypothesized that wave runup results from standing waves formed by the reflection of wave energy that is not dissipated by wave breaking. This hypothesis has been used successfully to interpret wave runup data on natural beaches (e.g., Holland et al. 1995; Raubenheimer et al. 1995).

Earlier theories for the prediction of wave runup on smooth slopes were synthesized by LeMéhauté et al. (1968) and are normally included in textbooks (e.g., Whitham 1974; Mei 1989). These theories for regular waves shed light on the wave mechanics involved in wave runup but are not accurate enough for practical applications. The prediction of wave runup is very difficult because the wave runup processes are nonlinear and involve the moving shoreline. Furthermore, waves typically break on the slope of a coastal structure unless the slope is sufficiently steep, whereas the steep slope is generally covered with armor units whose effects on wave runup must be accounted for. On the other hand, laboratory data on regular wave runup were presented by various deign curves based on dimensionless parameters as explained above. The only exception was the simple empirical formula proposed by Hunt (1959) for breaking wave runup on smooth uniform slopes in relatively deep water.

Various overall properties of regular waves breaking on smooth uniform slopes were measured and presented in different dimensionless forms until Battjes (1974) showed the utility of the surf similarity parameter or Iribarren number in expressing these overall properties in a synthesized manner. It may be noted that Iribarren's research is little known outside of Spain and that his contributions to coastal engineering are summarized by Losada et al. (1996). The surf similarity parameter combines the effects of the slope angle and incident wave steepness and reduces the number of dimensionless parameters required in describing the overall properties such as the breaker criterion, breaker type, wave runup and reflection. The use of the surf similarity parameter simplified the breaker criterion of Iribarren and Nogales (1949), the wave reflection formula of Miche (1951), and the wave runup formula of Hunt (1959). The surf similarity parameter was also shown to be effective in describing the stability of armor units as a function of the wave period and breaker type (e.g., Ahrens and McCartney 1975; Bruun and Johannesson 1976; Gunbak and Bruun 1979; Losada and Giménez-Curto 1979). Empirical formulas were developed to predict the normalized wave runup and run-down on various rough and permeable slopes as a function of the surf similarity parameter (e.g., Ahrens and McCartney 1975; Seelig 1980; Losada and Giménez-Curto 1981). On the other hand, Ahrens and Martin (1985) developed an empirical formula for the normalized runup of nonbreaking waves on smooth uniform slopes which can not be expressed as a function of the surf similarity parameter only.

If wave runup exceeds the crest height of a coastal structure, wave overtopping occurs. Only the volume of overtopped water during a specified time interval was measured in typical hydraulic model tests (e.g., Saville 1955; Jensen and Sorensen 1979). Accordingly, available empirical formulas based on the measured volume of overtopped water such as the formula proposed by Weggel (1976) predict only the average rate of regular wave overtopping and do not give any information on the temporal variations of the water velocity and depth during wave overtopping which are required to assess the severity of the damage caused by wave overtopping. The measured average overtopping rate was typically normalized by the incident wave height and gravitational acceleration. The normalized overtopping rate was then expressed in terms of various dimensionless parameters including the crest height normalized by the wave height or the hypothetical wave runup in the absence of wave overtopping. The overtopping rate is much more difficult to predict than wave runup partly because its order of magnitude varies considerably and partly because it is very sensitive to the ratio between the hypothetical wave runup and the crest height (e.g., Weggel 1976). Consequently, empirical formulas typically predict only order-of-magnitude estimates.

Solitary wave runup has been studied separately in relation to coastal flooding and damage caused by tsunamis. Available laboratory data on solitary wave runup are limited in comparison to regular wave runup data (e.g., Synolakis 1987). Since solitary waves do not have specific wave periods, the overall properties of solitary and regular waves on smooth uniform slopes were not compared until Kobayashi and Karjadi (1994a) introduced the representative solitary wave period and associated surf similarity parameter. Breaking solitary wave runup, normalized by its incident wave height, was shown to be predominantly dependent on the surf similarity parameter and larger than breaking regular wave runup affected by interaction between regular wave uprush and downrush on the slope. The characteristics of solitary wave breaking, decay and reflection as a function of the surf similarity parameter, were found to be qualitatively similar to those of regular waves (Battjes 1974).

Wind-generated waves are irregular with respect to their height and period. For lack of extensive irregular wave data, earlier attempts to predict irregular wave runup and overtopping hypothesized that runup and overtopping relationships for regular waves could be applied to the individual waves in an irregular wave train (Saville 1962). This hypothesis has been shown to yield fair agreement with experiments (Tsurata and Goda 1968; Van Oorschot and D'Angremond 1968; Battjes 1971; Gunbak and Bruun 1979), although it neglects wave group formation among storm waves and wave interactions on the slope. To apply this hypothesis, the joint distribution of wave heights and periods (e.g., Longuet-Higgins 1983a) is required but not well established (Goda 1985). Kobayashi and Reece (1983) applied this hypothesis to predict irregular wave overtopping on a circular gravel island for lack of any data apart from the data of Tautenhain et al. (1982) on obliquely incident wave runup on straight sea dikes. Their example computation indicated that the probability of wave overtopping and the amount of overtopped water would be sensitive to the spectral width parameter; thus, the correlation coefficient between wave heights and periods.

An alternative and much simpler approach is to assume that the probability distribution of individual runup heights follows the Rayleigh distribution (Battjes 1971; Ahrens 1977; Losada and Giménez-Curto 1981). The Shore Protection Manual (1984) adopted the Rayleigh runup distribution and estimated the significant runup (the average of the highest one-third of the runups) as the regular wave runup based on the significant wave height and period. Kobayashi et al. (1990a) showed that this method adopted in the Shore Protection Manual (1984) could be used for a preliminary prediction of the runup distribution on a rough permeable uniform slope because of its simplicity rather than its accuracy. However, the experiment by Kobayashi and Raichle (1994) on irregular wave overtopping of a revetment situated well inside a surf zone indicated that the Rayleigh runup distribution overpredicted the overtopping probability because of wave breaking seaward of the revetment. The empirical procedure in the Shore Protection Manual (1984) based on the Rayleigh runup distribution and the overtopping relationship developed for regular wave runup was found to underpredict the average overtopping rate in spite of the overprediction of the overtopping probability.

Since irregular wave experiments in wave flumes have lately become standard, it is more straightforward to develop empirical formulas directly from irregular wave data. Recent advances in the laboratory simulation of irregular waves were reviewed by Mansard and Miles (1995). Furthermore, recent empirical formulas based on extensive irregular wave data include several dimensionless parameters to account for various important effects as reviewed by Van der Meer (1994). These formulas were developed for the stability of armor units (Van der Meer 1987, 1988), irregular wave runup and overtopping (Van der Meer and Stam 1992; De Waal and Van der Meer 1992; Van der Meer and Janssen 1995), and irregular wave reflection (Seelig and Ahrens 1995; Davidson et al. 1996).

These empirical formulas predict the important quantities for the design of coastal structures but do not yield any information on the spatial and temporal (or spectral) variations. Moreover, the incident irregular waves are normally represented only by the significant wave height and the spectral peak or mean period measured at the toe of the structure. The specification of the incident waves at the toe of the structure is standard nowadays because the irregular wave transformation on the beach seaward of the structure is not simulated in typical laboratory experiments. However, this creates difficulties when the incident waves break on the beach seaward of the structure. The separation of the incident and reflected waves using spaced wave gages (e.g., Thornton and Calhoun 1972; Goda and Suzuki 1976; Kobayashi et al. 1990) or collocated gages (e.g., Guza et al. 1984; Hughes 1993) is based on linear wave theory and the accuracy of the available methods is uncertain for breaking waves. Furthermore, no simple model is presently available to predict the incident significant wave height inside the surf zone in the presence of waves reflected from the structure. Since the design waves for most coastal structures are depth-limited breaking waves, it will be necessary to develop an accurate method for separating the incident and reflected waves inside the surf zone. Alternatively, the incident and reflected waves may be separated immediately outside the surf zone using linear wave theory as was done by Kobayashi and Raichle (1994) and Kobayashi et al. (1996). However, this approach will require the simulation or modeling of the irregular wave transformation in the surf zone and the subsequent wave runup on the structure.

Wind-generated irregular waves are also directional. Experiments on coastal structures conducted in directional wave basins are becoming more common throughout the world. These experiments were conducted for straight structures on the horizontal bottom to include the effects of incident wave angles and directionality in empirical formulas for wave runup and overtopping (DeWall and Van der Meer 1992; Juhl and Sloth 1994), wave reflection (Isaacson et al. 1996), armor stability on the breakwater trunk (Galland 1994), and armor stability on the breakwater head (Van der Meer and Veldman 1992; Matsumi et al. 1994; Vidal et al. 1995). Available data are still limited partly because directional wave basin experiments include more design parameters and are much more time-consuming than unidirectional wave flume experiments. Furthermore, measurements are normally limited to free surface oscillations and slope profiles, but laboratory velocity measurements have become easier owing to acoustic doppler velocimeters (e.g., Kraus et al. 1994). On the other hand, field data associated with coastal structures are very limited and include only a few measuring points (e.g., Melo and Guza 1991; Dickson et al. 1995).

### Wave Runup on Beaches

The foreshore slope of a beach is generally much gentler than the seaward slope of a coastal structure. Incident waves normally break on the beach before they uprush on the foreshore slope. The time-varying shoreline elevation above the still water shoreline is called runup among nearshore oceanographers (e.g., Guza and Thornton 1982; Holman and Sallenger 1985), whereas wave runup is defined as the maximum elevation reached by the uprushing water in this paper and among coastal engineers (e.g., Shore Protection Manual 1984). The time-varying shoreline elevation is separated into wave setup (mean shoreline elevation above the still water level) and swash (fluctuations about the setup level).

For regular waves breaking on smooth uniform slopes, wave setup normalized by the incident wave height is on the order of 0.2 (e.g., Bowen et al. 1968), whereas wave runup normalized by the incident wave height is approximately proportional to the surf similarity parameter (Hunt 1959; Battjes 1974) which decreases as the slope is decreased for the given wave steepness. As a result, wave setup becomes dominant in comparison to swash on a very gentle slope for the surf similarity parameter on the order of 0.1 or less. In other words, almost all the incident wave energy is dissipated by wave breaking in a wide surf zone on the very gentle slope. Consequently, swash and wave reflection are negligible for regular waves breaking on the very gentle slope (e.g., Kobayashi et al. 1989). On the other hand, wave setup is small relative to swash on a steep slope that causes appreciable wave reflection (Battjes 1974).

Irregular wave setup and swash on beaches are more complicated because of appreciable swash fluctuations with periods substantially longer than the incident waves. The low frequency swash oscillations are typically dominant on gently sloping beaches (e.g., Huntley et al. 1977; Guza and Thornton 1982; Raubenheimer et al. 1995; Raubenheimer and Guza 1996). These low frequency swash fluctuations are also present but negligible on the steep slope of a coastal structure (e.g., Kobayashi et al. 1990). The low frequency swash fluctuations are related to surf beat or infragravity waves on beaches. In the following, recent research on irregular wave reflection, setup and swash on beaches are reviewed. Guza and Thornton (1982) presented a comprehensive summary of earlier research on swash oscillations, whereas Guza and Thornton (1985b) summarized earlier observations of surf beat.

Incident wind waves and swells whose periods are less than about 20 s are normally assumed to be dissipated completely on beaches. This assumption is appropriate on gentle dissipative beaches and allows the local application of linear progressive wave theory even inside the surf zone (Guza and Thornton 1980). However, wave reflection from steep reflective beaches is not negligible as discussed in Kobayashi et al. (1989). On the other hand, infragravity waves whose periods are in the range of about 20 to 200 s are generally assumed to be reflected completely from beaches, although wave reflection varies more gradually with respect to the wave period or frequency (Kobayashi and Wurjanto 1992a; Raubenheimer et al. 1995). This assumption allows the use of linear shallow water theory with no dissipation to compute edge waves, which are long waves trapped in the nearshore by reflection and refraction (Holman and Bowen 1979) and leaky (untrapped) waves, which are standing in the cross-shore direction (Guza and Thornton 1985b).

Elgar et al. (1994) estimated the energy of seaward and shoreward propagating waves on a natural beach using extensive data from an array of 24 pressure sensors in 13 m water depth, 2 km from the North Carolina coast. The observed ratio of seaward to shoreward propagating energy in the swell-sea frequency band decreased with increasing wave frequency and wave energy and increased with increasing beach slope, qualitatively consistent with a regular wave formula by Miche (1951). Most incident swell-sea energy dissipated in the surf zone but reflection was up to 18% of the incident swell-sea energy when the beach face was steep at high tide and the wave field was dominated by low-energy, low-frequency swell. In contrast, there was usually more seaward than shoreward propagating energy in the infragravity frequency band. This trend increased with increasing swell energy, suggesting the generation of infragravity waves in very shallow water. On the other hand, Baquerizo et al. (1997) examined the cross-shore variation of the local reflection coefficient of normally incident wind waves which was shown to increase shoreward with the increased percentage of breaking waves. The incident wave energy is dissipated due to wave breaking in the surf zone but the energy reflected, presumably from the shoreline, seems to be affected little by wave breaking.

Irregular wave setup on natural beaches was estimated as the time-varying shoreline elevation measured using resistance wires and films. Guza and Thornton (1981) used a resistance wire positioned 3 cm above and parallel to a gently sloping beach face. The measured setup was about 17 percent of the deep water significant wave height but the data consisting of 11 estimates showed considerable scatter. Holman and Sallenger (1985) measured wave setup on a moderately steep beach with a nearshore bar using time-lapse photography where the manually digitized shoreline for 154 time series was estimated to correspond to the water depth on the order of 0.5 cm. The wave setup data also exhibited considerable scatter and showed some influence of the nearshore bar at low tide. As a whole, the measured setup was much larger than 17% of the significant wave height in 20 m water depth.

Holman and Guza (1984) compared the shoreline elevations measured using the time-lapse photography technique and resistance wires elevated either 3 or 5 cm above the bed. More recently, Holland et al. (1995) compared the time-varying shoreline elevations measured using video images and five resistance wires at elevations of 5, 10, 15, 20 and 25 cm above the beach face. These comparisons indicated the sensitivity of the shoreline elevation to the wire elevation owing to thin runup tongues. The wave setup and the swash standard deviation increased with the decrease of the wire elevation. The video-based estimate corresponded to a very near-bed (less than a few centimeters elevation) wire measurement. These comparisons imply that wave setup and swash depend on the definition of the time-varying shoreline. Wave setup defined by the mean water depth on an impermeable slope becomes tangential to the beach face and approaches the upper limit of wave runup because the mean water depth is positive in the region wetted by uprushing water (Bowen et al. 1968; Nielsen 1989; Kobayashi and Karjadi 1996). On the other hand, Nielsen (1988,1989) measured the mean water level on a natural beach and the mean water table inside the beach using manometer tubes. He defined the shoreline setup as the elevation of the intersection between the beach face and the straight line connecting the measured mean water level and water table. This definition neglects the possible formation of a seepage face when the water table inside the beach outcrops on the beach face above the mean water surface in the ocean (Nielsen 1990; Turner 1993).

The considerable scatter of available data on wave setup on beaches appears to be caused mainly by different methods used to measure wave setup. In addition, wave setup may be sensitive to the spatial and temporal variability of the beach topography where edge waves appear to play an important role in the generation of rhythmic beach morphology (e.g., Holman and Bowen 1982). Lippmann and Holman (1990) used daily time exposure images of incident wave breaking on an open coast sandy beach to infer the spatial and temporal variability of the nearshore sand bar morphology for 2 years at the site where Holman and

Sallenger (1985) measured wave setup. The bar morphology was found to be complex and change rapidly during storms (on time scales of less than 1 day). The change of the bar morphology may cause the corresponding changes of the mean water level and circulation pattern including rip currents where Dalrymple (1978) reviewed various theories proposed for rip current generation. The recent measurements by Smith and Largier (1995) using a sector-scanning Doppler sonar indicated that the observed rip currents were episodical and aperiodical. If the mean water level is sensitive to the beach topography, accurate prediction of wave setup on the beach face will require sufficient data on the beach topography. The dynamics of the beach topography can not be predicted at present, although efforts were made to predict the shoreward movement of a linear bar outside the surf zone (Trowbridge and Young 1989) and the seaward movement of a linear bar inside the surf zone (Thornton et al. 1996).

Measured swash on beaches has been analyzed using spectral methods to examine the variations of the shoreline oscillations with respect to the frequency $f$. The measured swash spectra in the high frequency band were found to be approximately proportional to $f^{-4}$ (e.g., Huntley et al. 1977; Raubenheimer and Guza 1996) or $f^{-3}$ (Guza and Thornton 1982) and almost independent of the incident wave height. This high frequency band approximately corresponded to the wind wave frequency band. This was interpreted as the saturation of the shoreline oscillations caused by breaking wind waves. Huntley et al. (1977) explained the $f^{-4}$ dependence of the saturated swash spectra in the high frequency band using the breaking criteria of Miche (1951) and Carrier and Greenspan (1958) for regular waves on uniform slopes, assuming that each frequency component behaves like regular waves without interactions among frequency components. On the other hand, the observed swash spectra in the low frequency band did not show any clear frequency-dependence and were not saturated. Swash energy in the low frequency band on gently sloping beaches tended to increase linearly with increasing incident wave energy and become more dominant with increasing incident wave height (Guza and Thornton 1982; Raubenheimer and Guza 1996; Ruggiero et al. 1996).

Low frequency swash oscillations can be generated in various manners. For a steep reflective slope, individual wind waves break and uprush on the slope and swash at wind wave frequencies is dominant (e.g., Kobayashi et al. 1990). Grouped incident waves were observed to runup and overtop as an amplified group, resulting in the increase of low-frequency components (Kobayashi et al. 1989; Kobayashi and Raichle 1994). Furthermore, wave uprush and downrush caused by individual waves were observed to interact and reduce the number of individual runup events, resulting in the increase of swash oscillations periods (Carlson 1984; Mase and Kobayashi 1993). However, these low frequency swash oscillations on the steep slope are generally small in comparison to the swash oscillations associated with individual waves. If the slope is gentle enough to cause the dissipation of individual waves before they reach the shoreline, low frequency swash oscillations on such a gentle dissipative slope can not be explained by the mechanisms based on individual waves. Large bores were observed to overtake and capture smaller ones (e.g., Raubenheimer et al. 1995) but this is not the dominant mechanism for energy transfer to low frequencies as explained by Guza and Thornton (1982).

Infragravity waves have been observed by many researchers to be substantial in very shallow water and important in inner surf zones as reviewed by Guza and Thornton (1985b). The measured fields of the cross-shore velocity and free surface elevation were shown to be consistent with high model edge waves or cross-shore standing waves by many researchers as also reviewed by Guza and Thornton (1985b) where the cross-shore velocity and elevation fields for high mode edge waves and standing waves are too similar to differentiate these waves. Low mode edge waves were observed in alongshore velocity fields (Huntley et al. 1981; Oltman-Shay and Guza 1987). The observed low frequency swash oscillations on dissipative beaches have been shown to be consistent with linear standing waves by many researchers (e.g., Guza and Thornton 1985b; Cox et al. 1992; Holland et al. 1995). Oltman-Shay and Guza (1987) estimated that low mode edge waves contributed significantly to the low frequency swash spectra observed on two California beaches. It may be noted that shear waves with periods and alongshore wavelengths of the order of 100 seconds and meters, respectively, are generated by the shear instability of the longshore current and cause negligible free surface variations because shear waves are not surface gravity waves (Bowen and Holman 1989; Oltman-Shay et al. 1989).

To predict the low frequency swash oscillations on dissipative beaches caused by infragravity waves, it is necessary to predict infragravity waves in the nearshore. Several models have been proposed for the generation of infragravity waves. Longuet-Higgins and Stewart (1962) showed the existence of a second-order bound wave under normally incident wave groups outside the surf zone that could subsequently be reflected from the shoreline and escape out to deep water as free waves. Gallagher (1971) extended their cross-shore group forcing model to obliquely incident waves and showed the possibility of resonant second-order forcing of edge waves outside the surf zone. The laboratory experiment by Bowen and Guza (1978) provided empirical evidence of this resonant interaction model even when the incoming waves broke. Alternatively, Symonds et al. (1982) developed a cross-shore model for the generation of infragravity waves by the time-varying breakpoint and the accompanying variation in wave setup inside the surf zone. This model can not generate edge waves but may possibly be extended to obliquely incident waves. List (1992) developed a cross-shore model by combining the generation mechanisms proposed by Longuet-Higgins and Steward (1962) and Symonds et al. (1982). Bryan and Bowen (1996) showed that edge waves could be trapped and amplified on a nearshore bar. Elgar et al. (1992) observed infragravity waves for about 1 year in 8 m water depth in the Pacific and in 8 and 13 m depths in the Atlantic. The observed infragravity wave energy was well correlated with energy in the swell frequency band of 7 to 20 s periods, suggesting that the infragravity waves were generated locally by the swell. Infragravity waves were separated into free waves (edge waves or leaky waves radiating to or from deep water) and bound waves (second-order waves coupled to groups of incident waves). Bound wave contributions were significant only for energetic incident waves (significant wave heights greater than about 2 m). In summary, these studies indicate difficulties in predicting infragravity waves in the nearshore where no model is presently available to predict edge waves and leaky waves radiating from deep water.

Since infragravity waves are not predictable, empirical attempts have been made to relate swash statistics on foreshore slopes directly to incident wind waves in relatively deep water.

Holman and Sallenger (1985) defined the significant swash height as $4\sigma$ where $\sigma$ was the standard deviation of the measured swash oscillation on a moderately steep beach. The incident waves were characterized by the significant wave height and spectral peak period. The significant swash height normalized by the significant wave height was plotted as a function of the surf similarity parameter where the foreshore slope was used to represent the beach slope effect on swash. The normalized swash height increased with the increase of the surf similarity parameter in a manner similar to Hunt's formula (Battjes 1974), although the data consisting of 154 points showed considerable scatter. The swash height in the wind wave frequency band appeared to be saturated only when the surf similarity parameter was sufficiently small. Holman (1986) analyzed the same data and obtained the maximum runup height during each data run of 35 minutes, the 2% exceedance level of shoreline elevation, the 2% exceedance level for individual runup peaks, and the 2% exceedance level for swash height as determined by a zero upcrossing method. These extreme values were normalized by the incident significant wave height and plotted against the surf similarity parameter. The normalized extreme values also increased with the increase of the surf similarity parameter but exhibited considerable scatter. For the surf similarity parameter greater than about 1.5, the runup was dominated by the incident wave frequencies but for smaller surf similarity parameter longer period motions dominated the swash. This demarcation may be regarded as a crude estimate for separating steep (reflective) and gentle (dissipative) beaches for the purpose of wave runup predictions.

Nielsen and Hanslow (1991) measured wave runup probabilities using a cross-shore array of stakes placed on a wide range of sandy beaches in Australia. They counted the number of individual waves running up past each stake whose elevation was measured on each beach. The exceedance probability for the given elevation above the still water level was estimated as the ratio between the counted number of individual waves and the total number of individual waves expected during each data run of 20 minutes. The estimated exceedance probabilities for the cross-shore stake array were shown to follow the Rayleigh distribution reasonably well where the vertical scale involved in the Raliegh distribution was obtained using a linear regression analysis. The vertical scale for steep beaches was shown to be consistent with Hunt's formula in which the root-mean-square wave height and significant wave period were used. However, the vertical scale for gentle (flat) beaches was approximately independent of the beach slope. The foreshore slope of about 0.1 was the demarcation between the steep and gentle beaches for their data. The proposed empirical formulas for the vertical scales for the steep and gentle beaches showed considerable scatter.

Available field data on wave runup on beaches indicate difficulties in developing general empirical formulas for different beaches and various incident wave conditions. This is probably because various wave transformation processes occur on actual beach profiles between the shoreline and the offshore site where the incident waves are specified. Laboratory experiments on irregular wave runup on coastal structures in relatively deep water involve less variables. Moreover, the horizontal distance between the toe and shoreline (waterline) on coastal structures is too short to allow the development of wave motions of different time and spatial scales. However, most coastal structures are located well inside surf zones during design storm waves and various wave transformation processes on fronting beaches will affect irregular wave runup on coastal structures (e.g., Kobayashi and Raichle 1994).

The statistical distributions of the swash oscillations measured on beaches were also compared with statistical models based on linear (Gaussian) random waves (Huntley et al. 1977; Holland and Holman 1993). If the swash oscillation is Gaussian, the probability distribution of the time-varying shoreline elevation can be described only by its mean (wave setup) and standard deviation (degree of the shoreline oscillation). The Gaussian assumption was found to be satisfactory apart from certain discrepancies related to the skewness and kurtosis of the measured swash distributions. This conclusion is similar to that of Guza and Thornton (1985a) who computed the various moments of the fluid velocity field measured on a gently sloping beach in estimating nearshore sediment transport rates. The Gaussian model with the measured mean and standard deviation predicted the even moments fairly accurately but could not predict the odd moments associated with the skewness and nonlinearities of the velocity field. On the other hand, Kobayashi et al. (1997) measured the free surface elevations using vertical gages placed at fixed locations on a smooth impermeable slope. In their laboratory experiment, the free surface elevation could not be lower than the slope elevation. The probability distribution of the measured free surface elevation in the lower swash zone was shown to be approximately exponential with the skewness being equal to two. Future studies will be required to establish the relationship between the shoreline elevation on the beach face and the free surface elevation on a fixed location in the swash zone. Statistical approaches may not reveal the swash dynamics but allow one to describe time-varying variables using a few parameters such as the mean, standard deviation and skewness.

Wave overtopping on beaches and dunes is important in predicting sediment overwash but has been studied very little in the past. Kobayashi et al. (1996) conducted laboratory experiments to measure wave reflection, overtopping, and overwash of dunes. The measured reflection coefficient and overtopping rates were compared with the empirical formulas of Seelig and Ahrens (1995) and Van der Meer and Janssen (1995), respectively, developed for coastal structures. The equivalent uniform slope for overtopping was assumed to be the overall slope between the dune crest and the point where the water depth equaled the significant wave height. The toe depth of the coastal structure was assumed to correspond to the water depth immediately seaward of the breaker zone on the beach. The formulas with these adjustments were then shown to predict the order of magnitude of the measured reflection coefficients and overtopping rates. Furthermore, the average volumetric sand concentration in the overwash flow was measured to be about 0.04 for the small-scale experiments. Additional laboratory experiments will be required to asses the validity of these empirical results because the ranges of parameters varied in these experiments were limited. Field experiments will be very difficult during storms that are severe enough to cause wave overtopping and overwash.

## NUMERICAL MODELS

Various models on nearshore wave dynamics have been reviewed recently. These reviews include wave propagation in the nearshore (Dalrymple 1992; Mei and Liu 1993; Liu 1994;

Kirby 1997), wave breaking on beaches (Peregrine 1983), surf zone dynamics (Battjes 1988; Svendsen and Putrevu 1996), wave impact on vertical walls (Peregrine 1995), wave motions on inclined coastal structures (Kobayashi 1995), and long wave runup (Yeh et al. 1996). Numerical models related to the prediction of wave runup and overtopping on beaches and coastal structures for given wave conditions in relatively deep water on the order of 10 m or less are reviewed in the following.

Regular wave setup and currents induced by monochromatic waves breaking on beaches are normally predicted using the time-averaged continuity and momentum equations including the radiation stresses associated with the time-averaged momentum fluxes due to waves (e.g., Bowen et al. 1968; Longuet-Higgins 1970. Wu and Liu 1985). The radiation stresses for monochromatic progressive linear waves over a slowly varying water depth was given by Longuet-Higgins and Stewart (1964). The breaker height inside the surf zone has been estimated assuming that the ratio $\gamma$ between the local height and mean water depth is constant where $\gamma$ is in the range of about 0.7 to 1.2 depending on the beach slope and steepness as summarized by Raubenheimer and Guza (1996). The assumption of constant $\gamma$ is not appropriate in the trough region of a barred beach. Dally et al. (1985) used a time-averaged energy equation with an empirical dissipation rate due to wave breaking to predict the wave height variation across a beach of arbitrary profile.

Irregular wave setup and currents induced by wind waves breaking on beaches are predicted by adjusting the time-averaged equations for regular waves in different ways. The simplest approach is to represent irregular waves by the root-mean-square wave height $H_{rms}$ with a representative wave period such as the spectral peak period or mean period (e.g., Battjes and Janssen 1978; Thornton and Guza 1983; Battjes and Stive 1985) where no time-averaged equation is available to predict the spatial variation of the representative period. The representative wave direction for directional random waves is normally selected to reproduce the radiation stresses (e.g., Guza and Thornton 1985a; Wu et al. 1985; Thornton and Guza 1986) where the spatial variation of the wave direction is based on unidirectional monochromatic wave theory such as Snell's law for beaches of alongshore uniformity and the conservation of wave number for arbitrary bathymetry. The spatial variation of $H_{rms}$ is predicted using the time-averaged energy equation with the dissipation rate estimated using the formula for a hydraulic jump adjusted for irregular wave breaking (e.g., Battjes and Janssen 1978; Thornton and Guza 1983). Alternatively, the individual waves in the incident irregular wave train may be assumed to be approximated as a sum of regular waves for the given distribution of wave heights and periods (Mase and Iwagaki 1982; Dally and Dean 1986; Dally 1992). This approach is presently limited to normally incident irregular waves only and is similar to the hypothesis of equivalence used to predict irregular wave runup using regular wave runup data (Saville 1962). These approaches based on the time-averaged equations adjusted for irregular waves neglect wave reflection and infragravity waves and have never been applied to coastal structures.

The existing time-averaged models for normally incident irregular waves, which have been calibrated using surf zone data, may considerably underpredict the wave setup and the root-mean-square wave height $H_{rms}$ in the swash zone on a beach in light of the comparison made by Cox et al. (1994). This is partly because the ratio $\gamma$ between the wave height (significant wave height or $H_{rms}$) and mean water depth can be considerably larger in the swash zone than in the surf zone (Kriebel 1994; Kobayashi et al. 1997). Furthermore, the extensive field data of Raubenheimer and Guza (1996) indicate that the measured values of $\gamma$ in the inner surf zones are more variable than expected from earlier data and depend on the fractional change in water depth over a wavelength. As a result, the time-averaged models will need to be improved by adopting more accurate criteria for irregular wave breaking in the inner surf and swash zones. Time-averaged models are much more efficient computationally than time-dependent models and are suited for engineering applications. If the wave setup and $H_{rms}$ in the swash zone can be predicted accurately and the relationship between $H_{rms}$ and the swash standard deviation exists, it will be possible to predict the mean and standard deviation of the shoreline elevation whose probability distribution may simply be assumed to be Gaussian (Huntley et al. 1977). This will then allow one to estimate the exceedance probability as a function of the shoreline elevation in an efficient manner. Future studies will be necessary to develop such a statistical model.

Time-dependent models are required to predict the time-varying shoreline elevations on beaches and coastal structures. Numerical models based on the finite amplitude shallow water equations are presently the only models that have been verified fairly extensively using laboratory and field data as will be discussed later. These models for nondispersive waves can not predict wave shoaling without wave breaking over a long distance unlike the Boussinesq equations for dispersive waves (e.g., Kobayashi et al. 1989). Consequently, the computation domain of such a numerical model needs to be limited to the region with a relatively short distance from the shoreline. The small computation domain allows the use of small grid spacings to resolve breaking waves and runup sufficiently. Relatedly, the incident waves required as input to the model need to be specified in shallow water.

Spectral models such as third-generation wave prediction models may eventually be extended to shallow water (Van Vledder et al. 1994; Booij et al. 1994) and provide the incident waves required for the numerical model for predicting wave runup and overtopping. For practical problems, only the gross characteristics of incident wind waves such as the significant wave height, spectral peak period and pre-dominant wave direction may be available in relatively deep water. Use may then be made of a standard directional spectrum for wind waves such as the TMA frequency spectrum (Bouws et al. 1985) and the Mitsuyasu-type directional spreading function (Goda 1985). Shoaling and refraction of the assumed directional spectrum may be computed using linear finite-depth theory for parallel bottom contours (LeMéhauté and Wang 1982) unless the bathymetry is know to be complex. Linear theories have been shown to predict the gross characteristics of shoaled and refracted wind waves outside the surf zone reasonably well (e.g., Guza and Thornton 1980; Elgar and Guza 1985a). The frequency spectrum of the computed directional spectrum may be used to numerically generate the corresponding incident wave train in shallower water depth using a random phase scheme (e.g., Elgar et al. 1985) as was done by Kobayashi and Wurjanto (1992a), and Kobayashi and Karjadi (1996). However, this procedure does not account for incident infragravity waves and the wind waves in shallower water depth may show marked departures from this linear simulation due to nonlinearities (e.g., Elgar et al. 1984). Irregular wave runup on beaches and coastal structures may not be extremely sensitive to the details of the incident wind waves outside the surf zone (Kobayashi et al. 1987).

Numerical models based on the Boussinesq equations for weakly dispersive waves for a sloping bottom (Peregrine 1967) were developed to predict the weakly-nonlinear wave transformation outside the surf zone. Abbott et al. (1978, 1984) solved the two-dimensional Boussinesq equations in the time domain. Their numerical model was verified against analytical and experimental results for shoaling, refraction, diffraction, and partial reflection processes (Madsen and Warren 1984). Freilich and Guza (1984) developed a frequency domain model based on the one-dimensional Boussinesq equations to predict the nonlinear evolution of the wave field's Fourier amplitudes and phases. Elgar and Guza (1985a, 1985b, 1986) showed utility of the frequency domain model coupled with bispectral techniques for predicting and analyzing the observed nonlinear evolution of shoaling random waves. Yoon and Liu (1989) adopted the parabolic approximation of the Boussinesq equations developed by Liu et al. (1985) to predict the development of stem waves (Wiegel 1964) along a vertical wall numerically. The Boussinesq equations were recently modified to improve their linear dispersion properties in deeper water and extend their applicability to shorter waves (Madsen and Sorensen 1992; Nwogu 1993; Chen and Liu 1995; Wei and Kirby 1995). The Boussinesq equations for weakly-nonlinear waves were also extended to fully-nonlinear waves in order to predict shoaling waves up to the point of wave breaking more accurately (Wei et al. 1995).

The original and extended Boussinesq equations are based on the assumption of inviscid irrotational flow. This assumption has been used successfully to predict when and how waves break but is not valid after wave breaking (e.g., Peregrine 1983). Nevertheless, the Boussinesq equations have been extended to the surf zone in semi-empirical manners because no rigorous model is presently available to predict the detailed wave characteristics in the surf zone. The simplest approach is to include the horizontal momentum diffusion terms with an eddy viscosity in the Boussinesq equations after wave breaking (Zelt 1991; Karambas and Koutitas 1992; Sato and Kabiling 1994) where the eddy viscosity and the onset of wave breaking were expressed empirically in different ways. Zelt (1991) used a Lagrangian model to facilitate the prediction of the moving shoreline due to solitary waves. Nwogu (1996) estimated the eddy viscosity using a semi- empirical transport equation for turbulent kinetic energy produced by wave breaking. Sato (1996) applied the numerical model of Sato and Kabiling (1994) to simulate and interpret the observed tsunami propagation and focusing on the lee side of an island.

Alternatively, Schäffer et al. (1992) included the additional momentum fluxes due to a surface roller to account for wave breaking in the Boussinesq equations where an empirical geometric method was used to estimate the shape and location of the surface roller. A similar roller model was proposed by Brocchini et al. (1992). Madsen et al. (1994) extended the roler model of Schäffer et al (1992) to predict the swash oscillations on a gentle slope due to monochromatic and bichromatic waves. The moving shoreline was treated using a slot technique with the dispersive terms switched off at the still water shoreline. but the computed shoreline oscillations were dependent on the assumed width of an artificial slot. In short, the extended Boussinesq models have been applied successfully to predict wave propagation from relatively deep water to the surf zone but the detailed shoreline oscillations appear to have been predicted satisfactorily only for solitary waves (Zelt 1991). It is easier to treat the moving shoreline using the finite amplitude shallow water equations which do not contain the dispersive terms of third-order derivatives in the Boussinesq equations.

The vertical structure of the flow field is assumed in the shallow water equations including the extended Boussinesq equations so that the governing equations do not depend on the vertical coordinate. Numerical models based on vertically two-dimensional equations have been developed to predict the vertical and cross-shore variations of the flow field as explained in the following. These numerical models require more computational efforts and have not yet been expanded to the three dimensions. Furthermore, the computations are typically limited to relatively short durations even for two-dimensional problems.

For nonbreaking waves on steep smooth slopes, boundary integral equation methods for a potential flow with nonlinear free-surface boundary conditions (e.g., Grilli 1996) will probably predict wave runup more accurately than one-dimensional models based on the shallow water equations. Chian and Gerritsen (1990) computed solitary wave runup on smooth slopes and predicted the stability of armor units using the armor stability model of Kobayashi et al. (1986). The predicted stability number was in fair agreement with the data by Ahrens (1975) who tested riprap stability under regular wave action. The problem of their comparison is that solitary wave runup on smooth slopes is very different from regular wave runup on the corresponding riprap slopes. Liu and Cho (1994) included the effect of bottom friction via a boundary-layer approximation in their computations of solitary and cnoidal wave runup on smooth uniform slopes. Their computed results have confirmed that the effects of bottom friction on wave runup are important when the slope angle is less than 20° (1:2.7 or gentler slope). In short, the numerical models based on potential flow theory are of limited practical use because steep slopes are normally protected with armor units, whereas waves on gentle slopes tend to break. These models are more useful in predicting nonbreaking wave impact on vertical walls (e.g., Peregrine 1995).

On the other hand, to simulate plunging waves on impermeable slopes, the Navier-Stokes equations were solved by Sakai et al. (1986) using a marker and cell method and by Van der Meer et al. (1992) using a volume of fluid method. Pedersen et al. (1992) applied a discrete vortex model to simulate the motion of vortices generated by a jet of water and advected by the ambient potential flow where a semi-empirical procedure was used to generate vortices. Liu and Lin (1997) developed a numerical model to compute the evolution of a breaking wave. The incompressible Reynolds equations for the mean flow field and the $k-\epsilon$ equations for the turbulent field (e.g., Rodi 1980) were solved using finite difference methods where the free surface locations were represented by a volume of fluid method. This numerical model was shown to be in good agreement with available data on the runup and rundown of nonbreaking and breaking solitary waves.

The turbulence and vortices generated by breaking waves may be important for the suspension of sediment in the surf zone (e.g., Deigaard et al. 1986; Pedersen et al. 1995) but their effects on wave runup appear to be secondary. Laboratory measurements of turbulence using Laser-Doppler velocimeters (e.g., Stive 1980; Nadaoka and Kondoh 1982; Cox et al.

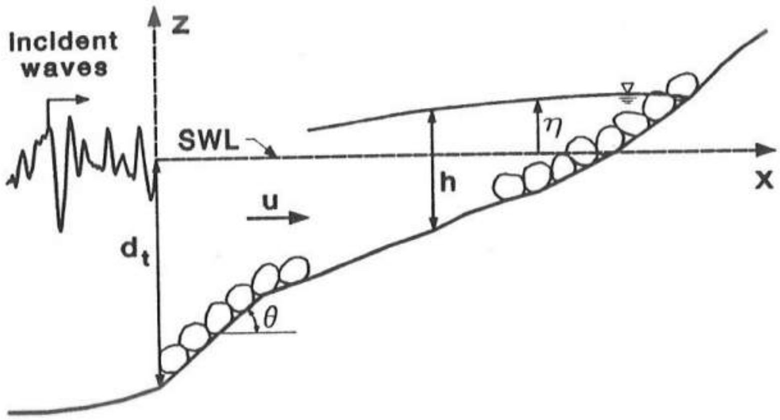


Figure 1: Wave Runup on Rough Impermeable Slope

1994) and field measurements of turbulence using hot film anemometers (George et al. 1994) have indicated that turbulent velocities are on the order of 10% or less of cross-shore velocities below wave trough level in surf zones. Chang and Liu (1996) measured two-dimensional instantaneous velocity fields under deep-water spilling waves using particle image velocimetry (Greated et al. 1992). The mean and turbulent velocities were obtained from the ensemble average of 24 repeated runs. The turbulent velocity was up to about 30% of the mean velocity but highly concentrated air bubbles near the surface made the velocity measurement very difficult. At present, no attempt has been made to measure the velocity and turbulence fields in the swash zone on a beach (Kobayashi and Karjadi 1996).

For nonbreaking waves on steep porous breakwaters, Sakakiyama and Kajima (1992) applied the porous body model developed originally for the flow in heat exchangers and fuel-rod bundles in a nuclear reactor. Sun et al. (1992) used a boundary element method for a potential flow outside of a rubble mound breakwater and a finite element method for a seepage flow inside the porous breakwater. On the other hand, Fischer et al. (1992) solved the Reynolds equations including laminar and turbulent flow friction in the porous media which were solved using a finite difference method. These numerical models have not been used widely probably because they require significant computation time in spite of uncertain empirical parameters included in the governing equations for the flow inside porous structures.

In the following, numerical models based on the finite amplitude shallow water equations are reviewed in more detail because they have been used fairly extensively to predict regular and irregular wave runup on inclined structures and beaches. These models may not be very accurate but are relatively simple and applicable to both breaking and nonbreaking waves on slopes of arbitrary geometry and reflectance.

### Wave Runup and Overtopping on Impermeable Structures

Numerical models were developed to predict the wave motion and runup on a rough or smooth impermeable slope for specified normally-incident waves as shown in Fig. 1 for the case of a rough slope where $x$ = horizontal coordinate taken to be positive landward with $x = 0$ at the seaward boundary of the computation domain; $z$ = vertical coordinate taken to be positive upward with $z = 0$ at the still water level (SWL); $d_t$ = water depth below SWL at the seaward boundary which is normally taken at the toe of the slope unless the incident waves break seaward of the toe. $\theta$ = local angle of the slope which is allowed to vary along the slope; $\eta$ = free surface elevation above SWL; $h$ = instantaneous water depth above the impermeable slope; and $u$ = depth-averaged horizontal velocity. The theoretical bottom level for the flow on the rough slope is difficult to pinpoint as is the case with oscillatory rough turbulent boundary layers (Jonsson 1980). The finite amplitude shallow water equations including bottom friction are the vertically-integrated equations of mass and horizontal momentum for shallow water waves on the impermeable slope

$$\frac{\partial h}{\partial t} + \frac{\partial}{\partial x}(hu) = 0 \tag{1}$$

$$\frac{\partial}{\partial t}(hu) + \frac{\partial}{\partial x}\left(hu^2 + \frac{1}{2}gh^2\right) = -gh\tan\theta - \frac{1}{2}f_b \mid u \mid u \tag{2}$$

where $t$ = time; $g$ = gravitational acceleration; and $f_b$ = bottom friction factor which may vary due to the spatial variation of bottom roughness (e.g., Kobayashi and Raichle 1994).

Eqs. (1) and (2) with $f_b = 0$ have been derived assuming potential flow in textbooks for wave theories (e.g., Mei 1989). However, these equations can be derived from the two-dimensional Reynolds equations under the assumption of $\sigma^2 \gg 1$ with $\sigma$ = ratio between the horizontal and vertical length scales (Kobayashi and Wurjanto 1992a). Eq. (1) is exact, whereas Eq. (2) is approximate because it neglects the additional momentum flux $m$ due to the vertical variation of horizontal velocity relative to the depth-averaged velocity $u$ (Kobayashi et al. 1997). The vertical momentum equation under the assumption of $\sigma^2 \gg 1$ yields approximately hydrostatic pressure below the instantaneous free surface. This numerical model for inclined structures under the assumption of $\sigma^2 \gg 1$ does not predict wave runup on vertical walls well (Kobayashi and Tega 1996). The one-dimensional energy equation corresponding to (1) and (2) can be derived from the two-dimensional horizontal momentum equation used to derive (2) as shown by Kobayashi and Wurjanto (1992a). The energy dissipation rate due to wave breaking in this energy equation involves the vertical variations of the horizontal velocity and shear stress which are unknown in the one-dimensional model. As a result, the flow field represented by $h$ and $u$ is computed using (1) and (2) and the energy dissipation rate due to wave breaking is then estimated using the energy equation. This procedure is similar to the conventional analysis of a steady hydraulic jump but does not simulate the wave breaking and associated energy dissipation explicitly (Kobayashi and Wurjanto 1992a).

Eqs. (1) and (2) are written in the conservation-law form of the mass and horizontal momentum equations except for the two terms related to the bottom slope and friction on the right hand side of (2). This form is normally used in numerical methods (e.g., Richtmyer and Morton 1967). Kobayashi et al. (1986,1987) normalized (1) and (2) using the representative wave height and period at $x = 0$ denoted by $H$ and $T$, respectively, where the water depth is on the same order as $H$ in shallow water. The normalized equations corresponding to (1) and (2) involve only two dimensionless parameters. One parameter is the normalized slope which is proportional to the surf similarity parameter $\xi$ defined as $\xi = \tan\theta/(H/L_o)^{0.5}$ with $L_o = gT^2/(2\pi)$, which becomes that used by Battjes (1974) for uniform slopes. The other parameter is the normalized bottom friction factor. As a result, the normalized runup depends on these two parameters and the normalized incident wave profile at $x = 0$ (Kobayashi et al. 1987).

Analytical solutions for (1) and (2) with $f_b = 0$ were described by Synolakis (1987). The analytical solutions are limited to nonbreaking waves on uniform slopes without any energy dissipation. To predict the flow characteristics and armor stability in the downrush of regular waves on uniform riprap slopes, Kobayashi and Jacobs (1985) applied the standing wave solution of (1) and (2) with $f_b = 0$ (Carrier and Greenspan 1958; Tuck and Hwang 1972) starting from the time when wave uprush was completed. An empirical formula was used to estimate wave runup. Wave rundown was assumed to be completed when the free surface slope became vertical during wave downrush. In short, analytical solutions are too restrictive and lack versatility for wide applications.

The bottom friction term in (2) is empirical but necessary because the effects of bottom friction on wave runup are not negligible except for steep smooth slopes. The existing knowledge of the bottom friction factor $f_b$ is mostly based on oscillatory boundary layer data well outside the surf zone (e.g., Jonsson 1966, 1980; Grant and Madsen 1986). Very limited laboratory data are available in the surf zone on a beach (Cox et al. 1996) and for coastal structures (Madsen and White 1976; Cornett and Mansard 1994). Measurements of wave boundary layers on natural beaches are rare and very difficult (Trowbridge and Agrawal 1995). The values of $f_b$ have been calibrated for the specific applications of numerical models based on (1) and (2) where these applications are described in the following. The calibrated values of $f_b$ are on the order of 0.01 for smooth slopes and 0.1 for rough slopes.

Finite difference methods to solve the hyperbolic equations (1) and (2) expressed in the conservation-law forms are given in textbooks for computational fluid mechanics (e.g., Richtmyer and Morton 1967; Anderson et al. 1984). Moretti (1987) reviewed numerical methods developed for flows with shocks that are similar to bores or breaking wave fronts. The explicit MacCormack (1969) method, which is a simplified variation of the two-step Lax-Wendroff method (e.g., Anderson et al. 1984), has been applied successfully to compute transient open channel flows with hydraulic jumps as summarized by Chaudhry (1993). Greenspan and Young (1978) used a method of characteristics to compute flow over a dyke. The explicit dissipative Lax-Wendroff method (e.g., Richtmyer and Morton 1967) has been used most widely since Hibberd and Peregrine (1979) and Packwood (1980) used it to compute bores on a beach of uniform slope. The MacCormack and Lax-Wendroff methods based on constant grid spacing are second-order accurate in space and time and allow the formation of bores. Both methods produce high frequency numerical oscillations at wave fronts of sharp gradients and require a procedure to damp the high frequency oscillations without disturbing regions of smooth gradients. A damping term is added in the dissipative Lax-Wendroff method (e.g., Packwood 1980), whereas a smoothing procedure is applied in the MacCormack method (e.g., Chaudhry 1993). These two methods were compared by Johnson et al. (1996) who modified the smoothing procedure slightly to account for the shoreline of zero depth. The computed results were practically identical. The numerical stability criteria for the explicit methods require courant numbers less than one, implying that the waves would not propagate more than one grid spacing in each time step (e.g., Anderson et al. 1984). On the other hand, (1) and (2) with $f_b = 0$ were solved by Watson et al.(1992) using the Weighted Average Flux method to compute a sequence of waves on a beach with an subaerial bar and by Titov and Synolakis (1995) using a variable grid finite-difference method to compute breaking and nonbreaking solitary wave evolution and runup. In short, several finite difference methods have been applied for the computation of breaking and nonbreaking wave runup on slopes with very limited intercomparisons of these methods.

The initial time $t = 0$ for the computation marching forward in time is generally taken to be the time when the specified incident wave train arrives at the seaward boundary of the computation domain. Correspondingly, the initial conditions are given by $\eta = 0$ and $u = 0$ at $t = 0$ in the computation domain where $\eta = 0$ and $u = 0$ satisfy (1) and (2). To compute regular wave runup on a slope, the computation needs to be continued until the computed shoreline oscillation becomes periodic. This transient duration has been found to be several wave periods for steep slopes but increases as the slope becomes gentler (Kobayashi et al. 1989). The cross-shore fluid motion responds much faster than the alongshore fluid motion because of the slow development of longshore current (e.g., Kobayashi and Karjadi 1996). To compute irregular wave runup on a slope, the shoreline oscillations needs to be computed for at least about 200 individual waves to perform the subsequent spectral and statistical analyses of the computed shoreline oscillations as discussed by Kobayashi et al. (1990) who used smaller time steps when numerical difficulties were encountered at the shoreline during their computations of long durations. To avoid the long computation time for irregular waves, Cox et al. (1992) attempted to develop a frequency-domain model based on (1) and (2) but used it to explain the generation of low-frequency waves due to the forcing terms computed using the time-domain model. It appears to be difficult to develop an independent frequency-domain model corresponding to (1) and (2) because these equations do not account for wave breaking explicitly.

The finite difference methods used to solve (1) and (2) do not provide the values of $h$ and $u$ at the seaward and landward boundary nodes. Time-dependent boundary conditions for hyperbolic systems were examined by Thompson (1990). To develop appropriate boundary conditions, (1) and (2) are expressed in the following characteristic form (e.g., Kobayashi et al. 1987).

$$\frac{\partial \alpha}{\partial x} + (u + c)\frac{\partial \alpha}{\partial x} = -g \tan\theta - \frac{f_b}{2h} \mid u \mid u \quad ; \quad \text{along } \frac{dx}{dt} = u + c \tag{3}$$

$$\frac{\partial \beta}{\partial t} + (u - c)\frac{\partial \beta}{\partial x} = g \tan\theta + \frac{f_b}{2h} \mid u \mid u \quad ; \quad \text{along } \frac{dx}{dt} = u - c \tag{4}$$

with

$$c = \sqrt{gh} \quad ; \quad \alpha = 2c + u \quad ; \quad \beta = 2c - u \tag{5}$$

where $c$ is the long wave speed, and $\alpha$ and $\beta$ are the characteristic variables. If $u < c$, the flow is locally subcritical, and $\alpha$ and $\beta$ represents the characteristics advancing landward and seaward, respectively. Using linear long wave theory, $\alpha$ and $\beta$ can be shown to be related to the waves propagating landward and seaward, respectively (Kobayashi et al. 1987; Kobayashi and Wurjanto 1989b). If $u > c$, the flow is locally supercritical, and both characteristics $\alpha$ and $\beta$ advance landward. The supercritical flow may occur in very shallow water. For example, the flow overtopping on the crest of a subaerial structure becomes supercritical during the peak overflow (Kobayashi and Wurjanto 1989a).

The seaward boundary algorithm depends on the wave data available at the seaward boundary $x = 0$. For most practical applications, the free surface elevation of the incident wave train, $\eta_i(t)$ at $x = 0$, is specified using wave theories or wave data in the absence of a structure, whereas the free surface elevation of the reflected wave train, $\eta_r(t)$ at $x = 0$, needs to be predicted to assess the degree of wave reflection in the presence of the structure. Consequently, Kobayashi et al. (1987) expressed the total water depth at the seaward boundary as

$$h(t,x) = d_t + \eta_i(t) + \eta_r(t) \qquad \text{at} \quad x = 0 \tag{6}$$

It is normally possible to choose the location of the seaward boundary such that $u < c$ at $x = 0$ and the value of $\beta = (2c - u)$ at $x = 0$ can be computed using (4) for the computed values of $h$ and $u$ in the computation domain. For the computed value of $\beta$, $\eta_r(t)$ may be estimated using linear long wave theory and $h$ is then given by (6) for the specified $\eta_i(t)$. Finally, $u$ is given by $u = (2\sqrt{gh} - \beta)$. The numerical procedure developed by Kobayashi et al. (1987) has been used widely for its capability of predicting reflected waves approximately. This procedure has been modified by Kobayashi et al. (1989) to account for second-order wave set-down and return current at $x = 0$ which may not be negligible for beaches. This procedure can also be modified to specify the measured free surface elevation, $\eta(t,x) = [\eta_i(t) + \eta_r(t)]$, at $x = 0$ (Cox et al. 1994; Kobayashi and Poff 1994).

If the measured time series of $\eta$ and $u$ at $x = 0$ are available, the seaward boundary algorithm is not necessary because the measured values of $\eta$ and $u$ can be used directly in the finite difference method used to solve (1) and (2) as was done by Raubenheimer and Guza (1996), Raubenheimer et al. (1996), and Elgar et al. (1997). The comparisons made by Kobayashi and Poff (1994) and Raubenheimer and Guza (1996) indicate that the computed results using these different procedures differ within the errors of the numerical model based on (1) and (2) which are typically less than 20% in comparison with free surface measurements.

The landward boundary conditions depend on the crest height of a structure relative to the still water level and wave runup. For a subaerial structure with no wave overtopping, the landward boundary is located at the moving waterline. The numerical algorithms dealing with the moving waterline or shoreline were reviewed in the proceedings edited by Yeh et al. (1996). All the algorithms attempt to satisfy the conservation of water mass but the details differ significantly. The algorithm becomes more complex if the detail of the waterline movement needs to be resolved accurately using small grid spacings and time steps. The predictor-corrector-smoothing procedure described by Hibberd and Peregrine (1979) and

Packwood (1980) was explained in detail by Kobayashi et al. (1987). To overcome numerical difficulties encountered at the moving waterline for the computation of irregular waves of long durations, this procedure was adjusted further as reported by Kobayashi and Poff (1994). In essence, this procedure attempts to satisfy both (1) and (2) at the moving waterline under the constraint of the finite difference method used to solve (1) and (2). After the moving waterline is obtained, the waterline elevation measured by a real or hypothetical wire placed at a specified distance above and parallel to the slope can be found from the elevation of the intersection between the wire and the instantaneous free surface elevation.

If wave overtopping occurs on the crest of a subaerial structure, the landward boundary may need to be taken at the landward edge of the crest because the jet of overtopped water issuing landward may not be described by (1) and (2). Kobayashi and Wurjanto (1989a) used (3)–(5) to develop the landward boundary algorithm for the overtopping flow at the landward edge of the crest located at $x = x_e$. If $u \leq c$ at the grid point next to $x = x_e$, the overtopping flow is subcritical or critical and only (3) for the characteristics $\alpha$ advancing from the computation domain can be used to find $h$ and $u$ at $x = x_e$. For this case, the flow at $x = x_e$ is assumed to be critical, that is, $u = c$ at $x = x_e$. On the other hand, if $u > c$ at the grid point next to $x = x_e$, the overtopping flow is supercritical and both (3) and (4) for the characteristics $\alpha$ and $\beta$ can be used to find $h$ and $u$ at $x = x_e$. After $h$ and $u$ at $x = x_e$ is found, the instantaneous overtopping rate is given by $hu$ at $x = x_e$ where $hu = 0$ at $x = x_e$ when the moving waterline is located seaward of the landward edge of the crest.

For a submerged structure, the landward boundary may be taken at the landward toe of the structure in order to compute the wave transformation over the entire structure. To find $h$ and $u$ at the landward boundary located at $x = x_t$, Kobayashi and Wurjanto (1989b) used (3) for the characteristics $\alpha$ advancing landward under the assumption of $u < c$ at $x = x_t$. For the value of $\alpha$ computed using (3), the free surface elevation of the transmitted wave train, $\eta_t(t)$ at $x = x_t$, may be obtained assuming linear long wave theory. The total water depth $h$ at $x = x_e$ is then the sum of $\eta_t(t)$ and the still water depth at $x = x_e$. Finally, $u = (\alpha - 2\sqrt{gh})$ at $x = x_e$ by use of (5). It should be stated that (1) and (2) with no frequency dispersion do not predict the details of the transmitted waves in the region where the still water depth increases landward. If the incident waves break on the crest of a submerged breakwater or nearshore bar, the transmitted bore-like wave may eventually become undular as it propagates further landward (Kobayashi and Wurjanto 1989b). The Boussinesq equations for dispersive waves are required to predict the development of an undular bore (Peregrine 1966). Even if the incident waves do not break on the crest of a submerged structure or nearshore bar, the number of wave crests will increase due to nonlinear wave interactions on the basis of the field data on a barred beach obtained by Elgar et al. (1997) who showed that the observed energy transfer could be predicted accurately by the Boussinesq equations. In short, the numerical model described here may predict the wave transmission coefficient fairly accurately (Kobayashi and Wurjanto 1989b) but the predicted wave train may not show the increased number of wave crests.

The computed variations of the water depth $h$ and the depth- averaged velocity $u$ with respect to $t$ and $x$ were used by Kobayashi et al. (1986) and Kobayashi and Otta (1987) to predict the hydraulic stability and sliding motion of armor units on a rough impermeable slope. The drag, lift and inertia forces acting on an armor unit were expressed in terms of the computed depth-averaged velocity and acceleration. The numerical stability model predicts the variation of the local stability number along the slope whose minimum value corresponds to the critical stability number for initiation of armor movement. It should be noted that the wave-induced forces acting on an armor unit are not well understood for lack of extensive data. Tørum (1994) measured the fluid velocities and wave-induced forces simultaneously but could not express the lift force in terms of the measured velocities. If armor units are placed on a slope in such a manner that no armor units are exposed to direct wave action, armor units will need to be lifted out of their positions to commence their sliding and rolling motions along the slope (Melby and Kobayashi 1996).

Numerical models similar to that described here were also developed using (1) and (2). These numerical models for rough or smooth impermeable slopes were compared with laboratory data as summarized in the following. The comparisons with regular wave data include: wave reflection and runup on rough slopes (Kobayashi et al. 1986, 1987); zero-damage stability numbers for riprap slopes (Kobayashi et al. 1986; Kobayashi and Otta 1987); free surface and waterline oscillations, dynamic pressure, and stone displacements on a 1:3 rough slope (Kobayashi and Greenwald 1986, 1988); wave reflection and runup on smooth slopes as well as free surface and waterline oscillations and dynamic pressure on a 1:3 smooth slope (Kobayashi and Watson 1987); free surface and velocity oscillations on a 1:2 slope (Allsop et al. 1988); wave reflection, runup and run-down on slopes covered with dolosse (Kobayashi and Wurjanto 1989c); average overtopping rates on smooth structures (Kobayashi and Wurjanto 1989a); wave reflection and transmission coefficients over smooth submerged structures (Kobayashi and Wurjanto 1989b); and wave runup, run-down, velocities and pressure on a 1:3 smooth slope in a large flume (Van der Meer and Breteler 1990). The comparisons with solitary wave data include: free surface elevations and velocities on a smooth submerged breakwater (Losada et al. 1992); and breaking solitary wave runup on a 1:19.85 smooth slope (Kobayashi and Karjadi 1994a). The comparisons with irregular wave data include: reflected wave trains and waterline oscillations on a 1:3 rough slope (Kobayashi et al. 1990); reflected wave trains, free surface oscillations, overtopping flow depth, and average overtopping rates on a 1:2 rough slope fronted by a gentle smooth slope and situated well inside the surf zone (Kobayashi and Raichle 1994); average overtopping rates on dunes (Tega and Kobayashi 1996); and armor stability on leeside slopes of overtopped breakwaters (Kudale and Kobayashi 1996).

The extensive comparisons with the measured free surface elevations and related quantities such as the waterline oscillations, runup and run-down on impermeable slopes indicate that the numerical models based on (1) and (2) can normally predict the free surface elevations within errors of about 20%. The prediction of wave run-down can be less accurate because run-down is sensitive to a thin layer of downrushing water. The more limited comparisons with the measured horizontal velocities indicate that the computed depth-averaged velocities represent the measured velocities within similar errors. The measured overtopping rates generally vary considerably and can normally be predicted only within a factor of about 2. The limited comparisons with the measured dynamic pressure suggest that the hydrostatic pressure approximation employed in (1) and (2) is less accurate on steep slopes. The agreement on gentle slopes is expected to be better because the hydrostatic pressure assumption is routinely applied to obtain the free surface elevation from the measured pressure in very shallow water on beaches (e.g., Raubenheimer et al. 1995, 1996; Raubenheimer and Guza 1996) since Guza and Thornton (1980) made the intercomparisons of local pressure, velocity and free surface elevation on a beach. On the other hand, the hydraulic stability model coupled with the numerical flow model based on (1) and (2) has also been shown to predict the observed zero-damage stability number within errors of about 20% if the drag, lift and inertia coefficients are calibrated. The measured armor displacement can be predicted only qualitatively.

The two-dimensional finite amplitude shallow water equations corresponding to (1) and (2) were recently solved by Liu et al. (1995) to explain the enhanced runup and trapping of tsunamis on the lee side of an island. Liu et al. (1995) solved the two-dimensional equations using a staggered explicit finite difference leap-frog scheme where the nonlinear convective terms were linearized with an upwind scheme of first order accuracy. Good agreement was obtained between the measured and computed free surface displacement and maximum runup around a circular island with a 1:4 smooth side slope for nonbreaking solitary waves. They have noted that no numerical solution has been demonstrated to successfully reproduce wave breaking in a two-dimensional flow. Future studies will be required to develop two-dimensional numerical models for breaking solitary, regular and irregular waves on coastal structures of arbitrary geometry.

### Wave Runup on Permeable Structures

Linear long wave theory combined with the Forchheimer equation expressing the flow resistance inside the porous rubble-mound breakwater was applied successfully to predict the reflection and transmission coefficients of nonbreaking regular waves for the case of no wave overtopping (e.g., Madsen and White 1975). Nonlinear long wave theory is required to predict the flow field and wave runup on a permeable slope more accurately. To predict regular waves on a permeable slope, Thompson (1988) applied the continuity equation (1) and the horizontal momentum equation (2) used by Kobayashi et al. (1986, 1987) to predict regular waves on an impermeable slope. He used a linear seepage flow equation (Darcy's law) to predict the flow inside a porous breakwater. His formulation neglects the mass and momentum fluxes between the flows outside and inside the breakwater.

Kobayashi (1986) extended the finite amplitude shallow water equations to a permeable slope by including the vertical mass flux in (1) and the horizontal momentum flux in (2) to express the mass and momentum fluxes per unit horizontal area into a permeable underlayer. Kobayashi (1986) also presented the vertically-integrated equations of mass and horizontal momentum for the flow in the permeable underlayer where the laminar and turbulent flow resistance was expressed in the form adopted and calibrated by Madsen and White (1975). Kobayashi and Wurjanto (1990) extended the characteristic equations (3) and (4) to the permeable slope and obtained an energy equation including the energy flux into the permeable layer to estimate the rate of energy dissipation due to wave breaking on the permeable slope.

The numerical model developed by Kobayashi and Wurjanto (1990) used the assumption of a thin permeable underlayer to simplify the horizontal momentum equation for the flow in the permeable layer. In the simplified equation, the horizontal hydrostatic pressure gradient drove the flows against the resistance in the thin permeable underlayer. Computation was made for regular waves on permeable slopes. The computed runup, run-down and reflection coefficients were in agreement with available empirical formulas, whereas the computed zero-damage stability number was compared with irregular wave data to show the limitations of the regular wave approximation.

Kobayashi et al. (1990a) applied this numerical model to compute irregular waves on rough permeable slopes. The computed maximum irregular wave runup was in fair agreement with the empirical formula of Ahrens and Heimbaugh (1988). The spectra of the computed waterline oscillations contained noticeable low-frequency components, which increased with the decrease of the surf similarity parameter as expected from runup data on natural beaches (e.g., Holman and Sallenger 1985). The spectra of the computed reflected wave trains indicated the increase of wave reflection with the decrease of frequency as well as the increase of the average reflection coefficient with the increase of the surf similarity parameter. These trends are qualitatively consistent with regular wave data (e.g., Battjes 1974). The computed critical stability number for initiation of armor movement was in fair agreement with the measured stability number for the start of damage (Van der Meer 1988). The comparison of the computed armor stability for the regular and irregular waves suggested that the armor stability would be reduced appreciably and vary less along the slope under the irregular wave action. Furthermore, Kobayashi et al. (1990b) examined the critical incident wave profile associated with the computed minimum stability of armor units to better quantify design wave conditions. In addition, a simplified model was proposed to predict the eroded area using the computed probability of armor movement. This model was in qualitative agreement with the empirical formula for the damage level proposed by Van der Meer (1987, 1988).

Norton and Holmes (1992) developed a numerical model for the reshaping of dynamically stable breakwaters under regular waves. The armor layer was numerically represented by a random assembly of interacting spherical particles. They adopted the numerical flow model of Kobayashi and Wurjanto (1990) which was shown to be in fair agreement with the velocities measured on the slope of a berm breakwater. A force model was used to assess armor stability on the slope and an empirical procedure was adopted to estimate the displacement distance of unstable armor units. The limited comparison of their numerical model with an experiment on the profile evolution of a berm breakwater was promising.

The numerical model of Kobayashi and Wurjanto (1990) based on the assumption of a thin permeable underlayer was found to be inappropriate for the experiment conducted by Kobayashi et al. (1991) for a 1:3 rough permeable slope with a thick permeable underlayer. The computed results did not satisfy the time- averaged equation of mass mainly because the thin layer model did not account for water storage in the region landward of the moving waterline on the permeable slope. Kobayashi and Wurjanto (1992b) and Wurjanto and Kobayashi (1993) extended the earlier model to a permeable underlayer of arbitrary thickness and included this region with the free surface inside the thick permeable underlayer. The inertia terms in the horizontal momentum equation for the flow inside the thick permeable underlayer are not negligible and were included in their numerical model based on the MacCormack (1969) finite difference method. The comparison with the experiment by Kobayashi et al. (1991) showed that this extended numerical model could accurately predict the time series and spectral characteristics of the measured reflected waves and waterline oscillations on the 1:3 permeable slope. The computed results indicated that the wave propagation, attenuation and setup inside the permeable underlayer reduced the intensity of wave breaking and resulting energy dissipation on the permeable slope but increased the energy flux and dissipation inside the thick permeable underlayer. The permeability effects also resulted in the time-averaged landward and seaward mass fluxes above and inside the permeable underlayer, respectively.

A similar numerical model was developed by Van Gent (1994) who used the modified Forchheimer equation for the flow resistance in the permeable layer proposed by Van Gent (1995a) on the basis of the measurements in an oscillating water tunnel. Van Gent (1994) allowed the discontinuity of the free surface elevation outside and inside the permeable slope to account for the formation of a seepage face on the permeable slope but an empirical procedure was required to describe the discontinuity. The formation of the seepage face depends on the degree of permeability (Turner 1993) and is expected to be much less pronounced on permeable breakwaters than on sand beaches. Tørum and Van Gent (1992) showed that the numerical model of Van Gent (1994) was in fair agreement with the velocities measured on a berm breakwater. His model was also in fair agreement with the regular wave profiles, runup and run-down on a berm breakwater measured by Van Gent (1995b), who also proposed a semi-empirical model to predict the profile evolution of a berm breakwater.

In summary, the above comparisons with laboratory data indicate that the numerical models based on the extended finite amplitude shallow water equations including the permeability effects can predict the free surface elevation and velocity on a permeable slope within errors of about 20%. Their capabilities are similar to those of the corresponding numerical models for impermeable structures. The accuracy of the computed flow field inside the permeable underlayer is uncertain for lack of data. The numerical models are presently limited to a single permeable underlayer above an impermeable lower boundary unlike linear wave models (e.g., Madsen and White 1975). The prediction of armor stability and displacement is more empirical and will require the calibrations of the force coefficients and armor displacement procedure for different types of permeable breakwaters. Finally, a horizontally two-dimensional model on a permeable structure of arbitrary geometry will need to be developed to predict the wave motion and armor stability on the head of a breakwater.

### Wave Runup on Beaches

The finite amplitude shallow water equations on a plane slope were solved numerically by Hibberd and Peregrine (1979) to predict the propagation of a bore on a beach of uniform slope. Their numerical solution was capable of describing the behavior of the bore runup and the formation of a landward-facing bore in the downrush. Packwood (1980) included viscous effects and studied periodic and irregular bores on beaches using measured free surface profiles as input at the seaward boundary. Svendsen and Madsen (1984) included the effects of turbulence generated by wave breaking to predict a turbulent bore in the surf zone only.

The computed results by Packwood (1980) appeared realistic and promising but his seaward boundary algorithm did not allow the specification of the incident wave train as input and the prediction or absorption of the reflected wave train. Such a seaward boundary algorithm was developed by Kobayashi et al. (1986, 1987) to predict wave reflection and runup on coastal structures as explained in relation to (6). Kobayashi et al. (1989) modified this seaward boundary algorithm to account for second-order wave set-down and return current at the seaward boundary on a beach of arbitrary profile. The numerical model based on (1) and (2) with the modified seaward boundary algorithm was then applied to predict the wave transformation in the surf and swash zones on gentle slopes as well as the wave reflection and swash oscillation on relatively steep beaches. The slope of a steep beach is generally smaller than the gentle slope of a coastal structure.

Kobayashi et al. (1989) compare the numerical model with the regular spilling wave data on a 1:40 slope by Stive (1980) and Stive and Wind (1982) as well as the undertow measurement for regular spilling waves on a 1:34.25 slope by Hansen and Svendsen (1984). The numerical model was shown to be capable of predicting the development of the wave profile asymmetry about the vertical axis from the symmetric cnoidal wave profile outside the breakpoint to the sawtooth profile in the inner surf zone. The computed shoreline oscillation on the gentle slope showed the dominance of the setup over the swash in accordance with the saturation hypothesis of Huntley et al. (1977). The numerical model was also shown to be incapable of predicting wave shoaling without wave breaking over a long distance, unlike the Boussinesq models for dispersive waves. In addition, comparisons were made with the wave reflection and swash excursion measurements for regular waves plunging and surging on a 1:8.14 slope described by Guza and Bowen (1976) and Guza et al. (1984). The numerical model was also compared with the measured time series of the reflected wave train and shoreline oscillation on a 1:8 slope with and without an idealized bar for incident regular and grouped waves. As a whole, the numerical model predicted both time-varying and time-averaged hydrodynamic quantities in the surf and swash zones on gentle and steep slopes within errors of about 20%. The exception was the time-averaged horizontal velocity (undertow) measured below wave trough level as explained in the following.

The undertow is driven by the vertical imbalance of the vertically-varying momentum flux and the vertically-uniform pressure gradient due to the wave setup in the surf zone on a beach. Because of its importance to the offshore sediment transport, various undertow models have been proposed (e.g., Dally and Dean 1984; Svendsen 1984; Stive and Wind 1986; Svendsen et al. 1987). Cox and Kobayashi (1996) used detailed laboratory measurements and showed the difficulties in estimating the terms involved in the time-averaged cross-shore momentum equation and the eddy viscosity used in existing undertow models. Consequently, Cox and Kobayashi (1997) developed a simple kinematic model based on a logarithmic undertow profile in the bottom boundary layer coupled with a conventional parabolic profile in the interior layer (e.g., Stive and Wind 1986). This model was capable of predicting the measured undertow profiles both inside and outside the surf zone for regular waves provided that the mean bottom shear stress was calibrated and the measured volume flux below wave trough level was used. This model was also shown to be applicable to irregular waves (Cox and Kobayashi 1998). In short, no undertow model is presently accepted widely because each model can be calibrated to yield fair agreement with each data set but the calibrated coefficients vary among existing data sets.

The difficulty in predicting the undertow using the depth-integrated equations (1) and (2) is that these equations predict the depth-averaged velocity $u$ only and do not model the wave breaking processes explicitly. The time-averaged value of $u$ computed by Kobayashi et al. (1989) was negative but its magnitude was underpredicted by a factor of about 2. This underprediction appears to be caused by the effect of a surface roller (Svendsen 1984; Madsen et al. 1994; Cox and Kobayashi 1998). If the magnitude of the time-averaged value of $u$ could be predicted accurately, the undertow profile could be predicted using the kinematic model of Cox and Kobayashi (1997).

For normally-incident irregular waves on beaches, Kobayashi and Wurjanto (1992a) qualitatively compared the numerical model of Kobayashi et al. (1989) with the field data of Holman and Sallenger (1985) on the wave setup and swash statistics on a moderately steep beach with a nearshore bar. The computed setup and swash heights were found to follow the lower bound of scattered data points partly because of the neglect of low-frequency components in the specified incident wave train and longshore variability on the natural beach. A more quantitative comparison was also made with the spectrum of the shoreline oscillation measured on a 1:20 plane beach by Elgar and Guza (1985a, 1985b) who also presented the corresponding wave spectrum in the 1.7 m depth immediately outside the surf zone. The numerical model predicted the dominant low-frequency components of the measured swash spectrum fairly well. On the other hand, Cox et al. (1994) used the measured free surface elevation inside the surf zone as input to the numerical model as explained in relation to (6). Comparisons were made with regular and irregular wave data from the SUPERTANK Project. The numerical model predicted the measured free surface oscillations in the surf and swash zones fairly well, whereas the time-averaged model of Battjes and Stive (1985) underpredicted the wave height and setup in the swash zone considerably.

The numerical model based on (1) and (2) with the different seaward boundary conditions explained in relation to (6) was compared with extensive field data. Raubenheimer et al. (1995) initialized the model with observations from pressure and current sensors collocated about 50 m from the mean shoreline in about 1 m depth on a gently sloping beach. The model was compared with pressure fluctuations measured at five shoreward locations and runup measured with a vertical stack of five wires at elevations of 5, 10, 15, 20 and 25 cm above the beach face. As the wire elevation increased, the measured mean runup location moved seaward, low-frequency energy decreased, and high-frequency sea swell energy increased. The numerical model accurately predicted these trends and the variation of wave spectra and shapes (e.g., wave skewness) across the inner surf zone. Raubenheimer and Guza (1996) presented additional comparisons including data from a steep concave beach. The runup predictions by the numerical model agreed well with observations with the vertically stacked runup wires. The numerical model predictions and observations of sea swell runup excursions and reflection indicated the saturation at sea swell frequencies. The relative amount of reflected energy at sea swell frequencies was observed and predicted by the model to increase both with increasing Irribarren number and decreasing distance from shore. At infragravity frequencies, runup excursions were observed to be unsaturated, and the numerical model accurately predicted the dominance of waves standing in the cross-shore where low-mode edge waves appeared to be negligible in these observations.

In addition, Raubenheimer et al. (1996) compared the numerical model with wave evolutions observed on transects crossing the mid and inner surf zone on three beaches (a steep concave beach, a gently sloping beach, and a beach with an approximately flat terrace adjacent to a steep foreshore). The model was initialized with the time series of sea surface elevation and cross-shore current observed at the most offshore sensors located about 50 to 120 m from the mean shoreline in mean water depths 0.8 to 2.1 m. The model accurately predicted the cross-shore variation of energy at both infragravity and sea swell frequencies. The ratio $\gamma_s$ of the sea swell significant wave height to the local mean water depth $\overline{h}$ was examined because of its importance to time-averaged models. The observed and predicted values of $\gamma_s$ increased with increasing beach slope $\beta$ and decreasing normalized (by a characteristic wavenumber $k$) water depth $k\overline{h}$ and were well correlated with $\beta/k\overline{h}$. Errors in the predicted individual values were typically less than 20%. Numerical simulations were used to examine the effects of infragravity waves on waves in the sea swell band. The computed values of $\gamma_s$ were insensitive to infragravity energy levels.

Finally, Elgar et al. (1997) compared the numerical model with the wave evolution observed across a barred beach. For a low energy (wave height about 0.4 m) bimodal wave field, high- frequency seas dissipated in the surf zone, but lower-frequency swell partially reflected from a steep (slope = 0.1) beach face, resulting in significant cross-shore modulation of swell energy. The numerical model was capable of predicting these combined effects of reflection from the beach face and dissipation across the sand bar and near the shoreline. However, Elgar et al. (1997) also noted the limitations of this shallow water model without frequency dispersion.

In summary, the above comparisons with extensive field and laboratory data indicate that the numerical model based on (1) and (2) for normally incident waves predicts the wave transformation and swash oscillation on beaches of various profiles fairly well (within errors of about 20%) provided that the model is initialized with the known wave conditions in the surf zone or immediately outside the surf zone. In other words, the seaward boundary of the numerical model must be located at a relatively short distance from the shoreline so that frequency dispersion may be neglected in the computation domain. Since the numerical model for normally incident waves can not predict edge waves, the good agreement between the model and field data implied that edge waves were negligible in these data sets. However, it is not clear whether the assumption of normally incident waves was really satisfied in these data sets. A numerical model for obliquely incident waves is needed to assess the range of incident angles for which (1) and (2) may be used to predict the cross-shore wave transformation.

Ryrie (1983) developed a time-dependent numerical model based on the two-dimensional, finite amplitude shallow water equations for predicting longshore fluid motion along a straight shoreline with a plane slope generated by incident periodic bores with a small angle of incidence. The numerical model was not compared with any data. This model was applied by Asano (1996) to predict sediment transport in the swash zone and by Brocchini and Peregrine (1996) to examine the integrated and averaged flow properties of the swash zone. Alternatively, Kobayashi and Karjadi (1994b, 1996) developed a numerical model that could be applied to beaches of arbitrary geometry under obliquely incident regular or irregular waves with small angles of incidence. Alongshore bathymetry variations were assumed not to exceed the alongshore variations associated with the obliquely incident waves.

The numerical model of Kobayashi and Karjadi (1994b, 1996) assumes that $\theta_c^2 \ll 1$ where $\theta_c$ is the characteristic wave angle in radians. Under this assumption, the two-dimensional continuity and cross-shore momentum equations have been shown to be approximated by (1) and (2) for normally incident waves, whereas the two-dimensional alongshore momentum equation has been shown to be simplified as

$$\frac{\partial}{\partial t}(hv) + \frac{\partial}{\partial x}(huv) = -gh\frac{\partial \eta}{\partial y} - \frac{1}{2} f_b \mid u \mid v \tag{7}$$

where $v$ is the depth-averaged alongshore velocity. The alongshore free surface gradient $\partial\eta/\partial y$ in (7) drives the alongshore time-dependent fluid motion. The cross-shore fluid motion for obliquely incident waves with $\theta_c^2 \ll$ is the same as that for normally incident waves, implying that the refraction of nearly normally incident waves is negligible within a relatively short computation distance.

Considering the errors of about 20% associated with the numerical model based on (1) and (2), the assumption of $\theta_c^2 \ll 1$ may be regarded to be acceptable if $\theta_c^2$ is less than about 0.1, that is, $\theta_c$ is less than about 20°. Available data on obliquely-incident wave runup, overtopping, reflection and armor stability on coastal structures (De Waal and Van der Meer 1992; Juhl and Sloth 1994; Isaacson et al. 1996; Galland 1994) suggest that the effects of incident wave angles may be neglected in view of the scatter of data points if $\theta_c$ is less than about 20°. No data is available to estimate the upper limit of $\theta_c$ for beaches. It should be noted that the numerical model for $\theta_c^2 \ll 1$ can not predict edge waves, shear waves and rip currents on beaches. For lack of data, it is not certain whether these phenomena occur on coastal structures as well.

For the cross-shore wave motions computed using (1) and (2) along three cross-shore lines, Kobayashi and Karjadi (1994b, 1996) solved (7) using the MacCormack (1969) method along the middle cross-shoreline where obliquely incident regular or irregular waves were specified at the seaward boundary located immediately outside the surf zone. First, the numerical model was compared with the regular wave data by Visser (1991). The wave height, setup and runup were predicted well but the longshore current profile was predicted poorly. The computed alongshore velocity $v$ indicated the dominance of the mean (longshore current) over the oscillatory component about the mean in the surf zone. The development of the longshore current from the initial condition of $v = 0$ to the steady current was computed to be very slow unlike the quick response of the cross-shore flow field. Second, the numerical model was compared with the field data by Thornton and Guza (1986). The measured cross-shore variations of the root-mean-square wave height and longshore current in the surf zone were predicted fairly well. The causes of large cross-shore and alongshore velocities near the shoreline predicted by the model were examined but could not be ascertained for lack of velocity data near the shoreline.

Empirical lateral mixing has been introduced in time-averaged models to predict the realistic longshore current profile induced by regular breaking waves (e.g., Longuet-Higgins 1970). Svendsen and Putrevu (1994) showed that the lateral mixing could be caused by the nonlinear interaction of vertically-varying cross-shore and longshore currents. They used linear wave theory with depth- limited breaker height to describe the regular wave motion. On the other hand, the time-averaged model of Thornton and Guza (1986) did not require lateral mixing to predict the longshore current profile induced by irregular breaking waves because their random wave model accounted for the breaking of individual waves over a breaker zone. This may explain why the numerical model of Kobayashi and Karjadi (1994b, 1996) without any empirical lateral mixing could not predict the realistic longshore current profile induced by regular breaking waves. However, the lateral mixing mechanism for regular and irregular waves needs to be elucidated in a unified manner.

The depth-integrated momentum equations (2) and (7) neglect the dispersion due to vertical nonuniformities of the instantaneous horizontal velocities where dispersion is the terminology used in hydraulic engineering (Rodi 1980). Kobayashi et al. (1997) included dispersion terms in (2) and (7) to account for additional cross-shore and alongshore momentum fluxes due to the vertical variations of the instantaneous horizontal velocities. To predict these unknown momentum flux corrections, two new equations were derived from the corresponding three-dimensional shallow-water momentum equations using a method of moments. The three equations for the cross-shore continuity, momentum, and momentum flux correction were solved numerically to predict the water depth and the cross-shore depth-averaged and near-bottom velocities as explained by Johnson et al. (1996). The two equations for the alongshore momentum and momentum flux correction were solved numerically to predict the alongshore depth-averaged and near-bottom velocities. Kobayashi et al. (1997) compared the developed model with the same data sets as Kobayashi and Karjadi (1994b, 1996) to examine the dispersion effects due to the momentum flux corrections. The dispersion effects on the wave height and setup were shown to be minor. The dispersion effects significantly improved the agreement with the longshore current profile induced by regular breaking waves but were secondary for the longshore current profile induced by irregular breaking waves. Consequently, this time-dependent model with the momentum flux corrections clarifies the lateral mixing mechanism for regular and irregular waves on planar beaches in a unified manner.

Karjadi and Kobayashi (1996) compared this numerical model with the field data for a barred beach by Smith et al. (1993). The model under the assumption of alongshore uniformity could not predict the broad peak of the longshore current in the bar trough region. The small alongshore variation of wave setup induced by a small alongshore variation of obliquely incident irregular waves was shown to significantly modify the driving force and longshore current profile in the bar trough region. On the other hand, the longshore current profile on planar beaches was shown to be insensitive to the small alongshore variation of obliquely incident waves. This may explain why existing longshore current models based on the assumption of alongshore uniformity were regarded to be adequate before their comparisons with the barred beach data.

In summary, the numerical models for obliquely incident waves have mainly been used to examine the alongshore fluid motion because of the assumption of small incident angles. Under this assumption, the cross-shore fluid motion and resulting runup depend only on the incident wave conditions and the beach profile along each cross-shore line. This may partially justify the comparisons made by Raubenheimer et al. (1995, 1996) between the numerical model for normally incident waves and their field data for which the incident irregular waves were directional but may have satisfied the assumption of the incident wave angle less than about 20°. However, a two-dimensional model for arbitrary incident wave angles will be required to predict the alongshore variation of wave runup on beaches of arbitrary three-dimensional profile.

The permeability effects of sand beaches on wave runup have been studied little probably because their effects appear to be small unlike rubble mound structures. Packwood (1983) developed a numerical model based on (1) and (2) with vertical seepage into an initially dry beach to calculate the influence of a permeable bed on the runup of a single bore on a gently sloping sand beach. Fine-medium grade sands were shown to have very little effect on the phase of wave uprush. Some differences between impermeable and permeable bed solutions were found in the phase of wave downrush. The numerical model was not compared with any data and the computed differences might be within the errors of the model.

On the other hand, wave setup and runup are important for the groundwater flow inside a sand beach. The mean onshore pressure gradient due to wave setup drives a circulation of water within a porous beach (Longuet-Higgins 1983b). Numerical models based on Darcy's law for vertically two-dimensional groundwater have been developed and applied to examine the groundwater response to artificial drainage proposed for beach stabilization (e.g., Li et al. 1996). These models generally account for tidal fluctuations but neglect wave setup and runup. The infiltration of water into sand beaches due to wave runup was studied using laboratory and field data in order to include the wave effects on the groundwater flow (Kang et al. 1994; Kang and Nielsen 1996). A numerical model for wave runup and infiltration into partially saturated sand will be required to predict the infiltration rate onto the water table inside a sand beach.

## CONCLUSIONS

The quantitative understanding of irregular wave runup and overtopping on inclined coastal structures and irregular wave runup on beaches have improved considerably for the last decade owing to the improved laboratory and field experimental capabilities followed by the development of time-dependent numerical models. However, these improvements have essentially been limited to normally incident waves on coastal structures and beaches of alongshore uniformity. Furthermore, laboratory experiments were typically conducted in relatively deep water to separate the incident and reflected waves in front of the structures using linear wave theory as well as to separate the problem of wave transformation in front of the structures from the problem of wave runup on the structures. Relatedly, field experiments on wave runup on beaches were conducted along straight shorelines without any structure to avoid complications caused by the interactions of incident waves with structures.

Most coastal structures are located in relatively shallow water and exposed to breaking or broken waves during design storms. Laboratory experiments under actual design conditions are usually conducted for site-specific problems and may not be summarized in generalized manners. As a result, systematic laboratory experiments on coastal structures located inside the surf zone will be necessary to investigate the wave transformation and breaking on beaches in front of structures and the subsequent runup and overtopping of the transformed and broken waves on the structures.

Field experiments on wave runup and overtopping on coastal structures during storms are desirable but it will be difficult to measure all the relevant quantities required to quantify the wave runup and overtopping processes. Relatedly, it is desirable to measure wave runup on beaches in the vicinity of coastal structures but extensive arrays of instrument will be required to measure the detailed spatial and temporal variations of the nearshore wave motions and runup on such beaches. Laboratory experiments in directional wave basins with extensive instrumentation are feasible but very time-consuming.

Numerical time-dependent models will need to be extended for directional random waves on coastal structures and beaches of arbitrary three-dimensional geometry, although such models will require considerable computation time. Furthermore, the time- dependent boundary conditions of the horizontally two- dimensional, finite amplitude shallow water equations are not well established.

In short, the progress in the experimental and numerical studies on irregular wave runup and overtopping may have been impressive but the remaining tasks appear to be more difficult and challenging. A hybrid approach based on laboratory and field experiments coupled with numerical models appears to be the most practical and promising approach to investigate the wave runup processes on inclined structures and beaches of arbitrary geometry due to spatially-varying directionally random waves.

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