Performance of Coastal Structures under Combined Storm Surge and Breaking Waves for Sequences of Hurricanes

by

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ABSTRACT

A coastal structure is normally designed to withstand wind waves during the peak of a design storm. This conventional approach may be appropriate for a preliminary design involving a large number of design variables. A detailed design will require the prediction of the performance and life cycle cost of the structure during its entire service life. Since future storms cannot be predicted deterministically, synthetic sequences of storm waves and water levels at a project site need to be generated numerically using long-term wave and water level data.

A synthesis of existing models and formulas is made here to compute the virtual performance of rubble mound structures in shallow water under combined storm surge and breaking waves for sequences of hurricanes. A Monte Carlo simulation method is used to generate sequences of hurricanes and tropical storms on the basis of a Poisson distribution and meteorological data. To reduce computation time, simple models are applied to deterministically predict the time series of wind, storm tide (sum of storm surge and astronomical tide), and significant waves for each storm. The irregular wave transformation including wave breaking and wave setup is predicted using the cross-shore, time-averaged equations for wave energy and momentum. This Monte Carlo simulation yields the time series of the mean water level and significant waves at the toe of the structure during the entire duration of each of a large number of storms during a specified number of years. As an example, ten 500-yr simulations are performed using available data at Panama City, Florida. The simulated results are analyzed to show the importance of the combined effect of storm surge, astronomical tide and wave setup in determining the mean water

depth and depth-limited wave height at the toe of a coastal structure in shallow water. The computed time series of the mean water depth and significant waves for each of the generated storms are used to predict the corresponding time series of the irregular wave overtopping rate and cumulative damage to the armor layer of a rubble-mound structure. The crest height and armor weight of the structure are first designed against the peak of a 100-yr storm. The structure designed conventionally is then exposed to approximately 350 storms for each 500-yr simulation. The computed wave overtopping rate and volume during the entire duration of each storm are analyzed to assess the severity of flooding hazards. The computed progression of damage to the armor layer is caused episodically by several major storms but slows down as the structure ages. The computed results are also used to quantify the equivalent duration of the peak of a storm which yields the same overtopping water volume. The equivalent peak duration is found to be approximately 3 hr. The damage increment is shown to correspond to the damage caused by approximately 1,000 waves during the peak of a hurricane storm.

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Chapter 1

INTRODUCTION

A significant portion of the world's population lives near the sea. The increasing development of the coastal zones due to growing population and recreational activities increases damage caused by storms with high winds that generate storm surge and large waves. The gradual rise of the mean sea level along many shorelines will threaten the coastal development. Consequently, to reduce the vulnerability of coastal communities to coastal hazards such as flooding and beach erosion, the improved prediction of storm damage is necessary. The purpose of this study is to advance our knowledge in the prediction of storm damage by analyzing the performance of coastal structures during their entire service lives.

Conventional rubble-mound structures have been designed deterministically for no or little damage to the armor layer within an acceptable wave overtopping rate during the peak of a design storm [e.g., Hudson (1959); Shore Protection Manual (1984); Van der Meer (1988a)]. This conventional approach has been adopted widely for its simplicity but will not reveal what will really happen to the structure during its entire service life. Designing the structure against the peak of the design storm may be appropriate for the preliminary design involving a large number of design variables. However, the detailed design will require the prediction of the performance of the structure including the cumulative damage to the armor layer under sequences of storms (Melby and Kobayashi 1998a) and the estimation of life cycle costs including construction costs, maintenance costs and economic losses.

To predict the performance of a structure designed using the conventional approach, synthetic sequences of future storm waves and water levels at the toe of the structure need to be generated numerically using long-term wave and water level data. For the probabilistic design of composite breakwaters in relatively deep water, Hanzawa *et al.* (1996) used the yearly maximum waves of a short duration (1000 waves), where storm surge was neglected. The use of the yearly maximum waves was out of necessity for lack of long-term wave data suited for their Monte Carlo simulation.

For rubble mound structures in relatively shallow water, storm surge and wave setup are important in determining the mean water depth and depth-limited irregular breaking waves at the toe of the structure. The depth-limited irregular breaking waves will cause irregular wave overtopping and damage to the structures in shallow water (Kobayashi and Raichle 1994; Melby and Kobayashi 1998a). Consequently, it is necessary to predict the simultaneous time series of the water level and wave characteristics for each synthetic storm. Advanced wind, surge and wave models are available [e.g., Cardone *et al.* (1992); Scheffner *et al.* (1996); Booij *et al.* (1999)], but storm surge and wind waves are generally predicted separately. Relatedly, coastal engineers involved in the design of coastal structures estimate the exceedance probabilities of the peak storm surge and significant wave height separately where the storm duration and the time series of the storm surge and significant wave height are generally neglected for the design of coastal structures.

As a first attempt to examine the combined effects of storm surge and wind waves on the performance of rubble mound structures in shallow water, use is made of the Monte Carlo simulation method combined with simple hurricane wind, surge and wave models which was adopted by Kriebel and Dean (1984) to predict the frequency distribution of dune erosion by a single hurricane. The number of hurricanes and tropical storms per year is assumed here to be given by a Poisson distribution [e.g., Scheffner *et al.* (1996)] where extratropical storms are not considered because of the difficulty in parameterizing such storms.

The cross-shore irregular breaking wave transformation on the beach in front of the structure is predicted using the nonlinear time-averaged wave model CSHORE (Johnson and Kobayashi 1998, 2000) which has been shown to be capable of predicting the wave setup and root-mean-square wave height at the toe of the structure situated inside the surf zone (Kearney and Kobayashi 2000).

The predicted time series of the mean water depth and the representative wave height and period for each storm are used to predict the wave overtopping rate and volume during the entire storm as well as the increment of the damage to the armor layer during the storm. Since the damage progression depends on sequences of storms and may slow down as the structure ages (Melby and Kobayashi 1998a), the structure is exposed to approximately 350 storms generated numerically for the duration of 500 years. The 500-yr Monte Carlo simulation is repeated ten times to estimate the variability of the long-term damage progression. The computation time for one 500-yr simulation was of the order of minutes due to the use of the very simple wind, surge and wave models.

This study is organized as follows. First, the Monte Carlo simulation model CYCLONE for hurricane wind, storm surge and offshore significant waves is explained concisely in Chapter 2 using the computed results at Panama City, Florida as an example. Second, the cross-shore transformation of irregular breaking waves is computed using CSHORE to predict the wave setup and wave heights at the toe of a structure located in the water depths of 4, 2 and 0 m below the mean sea level (MSL). The beach slope in front of the structure is simply assumed to be 1/800 and 1/40 without regard to beach profile evolution. This numerical model is described in Chapter 3. Third, wave overtopping of the structure with a seaward slope of 1/2 is computed for the entire duration of each storm using the empirical formula of Van der Meer and Janssen (1995). The numerical model OVERTOP used for this computation is described in Chapter 4. Fourth, the progression of damage to the armor layer due to approximately 350 storms is predicted using the damage progression formula by Melby and Kobayashi (2000). The numerical model DAMAGE used for this computation is described in Chapter 5. Lastly, the findings of this research are summarized and concluded in Chapter 6. The numerical models CYCLONE, CSHORE2, OVERTOP and DAMAGE used for the completion of this study are listed in Appendix A, B, C and D, respectively. It is noted that the concise results of this study will be published by Pozueta, Kobayashi and Melby (2002) and by Kobayashi, Pozueta and Melby (2002).

Chapter 2

NUMERICAL MODEL CYCLONE: STORM SURGE AND OFFSHORE WAVES

The numerical model CYCLONE listed in Appendix A consists of the deterministic models for hurricane wind, storm surge and significant waves coupled with the Monte Carlo simulation of the parameters used to characterized the frequency, track, size and strength of hurricanes as well as astronomical tide at a specific site. Since the Monte Carlo simulation deals with a large number of hurricanes, use is made of the simple deterministic models described in Shore Protection Manual (1977) and modified slightly by Kriebel (1982), where the references related to these models were quoted by Kriebel (1982) and are not repeated here. In the following, each of the four major modules of CYCLONE are explained concisely. The simple models in the modules could be replaced in the future by an advanced hurricane wind model (e.g., Cardone *et al.* 1992), a more accurate storm surge model (e.g. Westerink *et al.* 1992), and a third-generation wave model (Booij *et al.* 1999; Ris *et al.* 1999).

2.1 Hurricane Wind Model

A hurricane is a large, migratory cyclone of tropical origin. An average hurricane is a nearly circular low-pressure system. The center or eye of the hurricane, is a region of high temperatures, low pressures, and calm winds. Since surface winds are greatest in regions of the greatest change in pressure, surface winds increase gradually from the edge of the storm, reaching a maximum in this region of the largest pressure gradients. Wind speed then drops rapidly to a relative calm in the eye of the hurricane.

The following parameters are normally used to describe the intensity, size, speed, and direction of motion of the storm:

- Central pressure, P_o, relative to peripheral pressure, P_n
- Radius to maximum wind, R_{max}
- Forward speed of hurricane translation, V_f
- Direction of hurricane translation

The pressure drop from the *peripheral pressure* to the *central pressure*, $P_n - P_o$, is a universally accepted parameter of the hurricane intensity. The peripheral pressure is usually defined as the sea level pressure of the last closed isobar around a low pressure on a synoptic chart. For the numerical simulation, the peripheral pressure is approximated by standard atmospheric pressure, 29.92 in. Hg (inches of Mercury).

The *radius of maximum wind* is defined as the radial distance from the storm center to the point of maximum wind speed. This parameter is basically a measure of the hurricane size.

The *forward speed of hurricane translation* is defined as the velocity of the entire storm system. For numerical simulation, an average value of the storm speed, measured over the continental shelf or at the time of the storm landfall, is assumed to be representative of the most critical effects of the hurricane.

The *direction of hurricane translation* is defined as the direction of translation of the storm system relative to a given shoreline location. Like the forward speed, the direction of the storm may vary considerably over the duration of the storm;

however, the direction of motion is assumed constant as the storm approaches the shoreline.

The pressure field of a hurricane may simply be assumed to be given by [e.g., Shore Protection Manual (1977)]

$$\frac{P(r) - P_o}{P_n - P_o} = exp\left(-\frac{R}{r}\right) \tag{2.1}$$

where $P(r)$ = atmospheric pressure at a radial distance r from the storm center; P_o= hurricane central pressure; P_n= peripheral pressure; and R = radius of maximum wind.

An approximate equation for gradient wind flow (the pressure gradient force balanced by the centrifugal and Coriolis forces) in a moving hurricane with uniform forward speed may be expressed as [e.g., Kriebel (1982)]

$$\rho_a \frac{U_{gr}^2}{r} + \rho_a \left(f + \frac{V_f \sin(\alpha)}{r}\right) U_{gr} = \frac{\partial P}{\partial r} \tag{2.2}$$

where ρ_a = air density; U_{gr} = gradient wind speed; f = Coriolis parameter given by $f = 2\omega \sin(\phi)$ with ω = angular frequency of the earth ($\omega = 2\pi/24$ radians per hour) and ϕ = latitude in degrees (positive in the Northern Hemisphere); V_f = hurricane forward speed; α = angle from the forward velocity vector counterclockwise to the point of interest as shown in Fig. 2.1; and $\partial P/\partial r$ = pressure gradient force. If $V_f = 0$, equation 2.2 reduces to the equation given in Shore Protection Manual (1977), which suggested a different correction term for V_f.

For the numerical solution of the gradient wind at any point in a moving hurricane, the storm center is traced in the fixed horizontal coordinate system and the radius, r, and the angle, α, of any point are determined relative to the moving storm center. For the given input parameters, P_o, P_n, R, ρ_a, V_f, and f, the gradient wind speed can be obtained from equation 2.2 with equation 2.1 as a function of r and α

$$U_{gr} = U_c\left[\sqrt{\gamma^2+1}-\gamma\right] \tag{2.3}$$

with

$$\begin{aligned}
\gamma &= \frac{1}{2}\left(\frac{U_c}{U_g}+\frac{U_f}{U_c}\right)\\
U_f &= V_f \sin(\alpha)\\
U_c &= \sqrt{\frac{P-P_o}{\rho_a}\frac{R}{r}}\\
U_g &= \frac{U_c^2}{rf}\\
P &= P_o+(P_n-P_o)exp\left(-\frac{R}{r}\right)
\end{aligned}$$

where U_c = cyclostropic wind based on equation 2.2 with $f = 0$ and $V_f = 0$; U_g = geostrophic wind based on equation 2.2 with $V_f = 0$ and no centrifugal force; and U_f = forward speed component parallel to the gradient wind direction in Fig. 2.1. The gradient wind speed U_{gr} is the maximum approximately at $r = R$ and $\alpha = -90°$ for which $U_f = -V_f$.

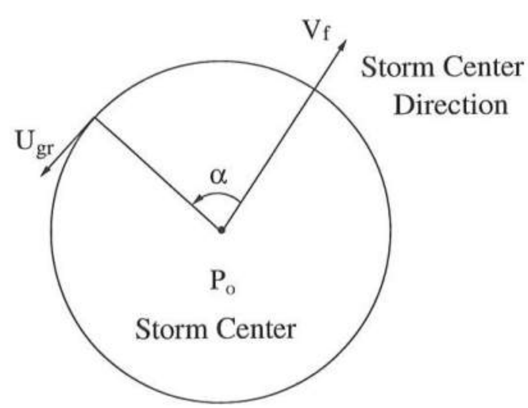

Figure 2.1: Definition Sketch of Angle for Idealized Hurricane Wind Field.

2.2 Storm Surge Model

Computer simulation of storm surge involves numerical integration of the hydrodynamic equations that govern long wave motion. The forcing terms associated with the governing equations, i.e., atmospheric pressure and wind stress, are estimated using equations 2.1 and 2.3 for a set of the hurricane parameters generated numerically using historical probability distributions. With the geometry of the coastline and bathymetry prescribed, the time series of storm surge is calculated by incrementally moving the storm along its track and computing the atmospheric pressure, wind stresses, water velocities and water level on the continental shelf.

Water level response on the order of one hour to one day along the open coast are normally classified into four major categories:

- Direct effects of the hurricane wind and pressure
- Secondary effects such as rainfall and river runoff
- Astronomical tide
- Wave set-down outside the surf zone and wave setup inside the surf zone

Direct effects of the hurricane system are typically the major components of any large storm surge. The rapid rise in water level that accompanies an intense hurricane is due mainly to the effects of strong winds and low pressure as the storm passes a given point in shallow water. In very deep water, wind setup has little effect on the mean water level, and the drop in barometric pressure is responsible for the water level rise. An approximate hydrostatic equilibrium exists under the hurricane pressure field, in which water level increases from the storm periphery to a maximum at the low-pressure storm center. As the hurricane moves toward shore over the continental shelf, winds are generally responsible for the greatest portion of the storm surge. The rise in water level of up to 1 m was observed to occur well before the hurricane winds or pressure reached the shoreline. This increase in water level called the initial rise or forerunner was estimated from tide gage records well before the rapid rise due to direct wind effects (Shore Protection Manual 1977).

Secondary effects such as rainfall, river runoff and tidal inlet flows are neglected in view of the approximations adopted here for the simple storm surge model.

Astronomical tide is generally important in storm tide (sum of storm surge and tide) prediction. Storm surge and astronomical tide are independently generated and then linearly superimposed to predict the storm tide level in the subsequent Monte Carlo simulation.

The increase of the mean water level due to irregular breaking waves is important in the surf zone. This wave setup is included in the irregular wave transformation model CSHORE2 in Section 3.1.

The two-dimensional, depth-integrated momentum and continuity equations with no tide are expressed as [e.g., Westerink *et al.* (1992)]

$$\frac{\partial U}{\partial t} + \frac{\partial}{\partial x}\left(\frac{U^2}{D}\right) + \frac{\partial}{\partial y}\left(\frac{UV}{D}\right) = fV - gD\frac{\partial \eta}{\partial x} - \frac{D}{\rho}\frac{\partial P}{\partial x} + \frac{1}{\rho}(\tau_{wx} - \tau_{bx}) \quad (2.4)$$

$$\frac{\partial V}{\partial t} + \frac{\partial}{\partial x}\left(\frac{UV}{D}\right) + \frac{\partial}{\partial y}\left(\frac{V^2}{D}\right) = -fU - gD\frac{\partial \eta}{\partial y} - \frac{D}{\rho}\frac{\partial P}{\partial y} + \frac{1}{\rho}(\tau_{wy} - \tau_{by}) \quad (2.5)$$

$$\frac{\partial \eta}{\partial t} + \frac{\partial U}{\partial x} + \frac{\partial V}{\partial y} = 0 \quad (2.6)$$

where t = time; x = cross-shore coordinate, positive onshore; y = alongshore coordinate; U = cross-shore volume flux per unit width; V = alongshore volume flux per unit width; η = free surface elevation above the mean sea level; D = total water depth above seabed; f = Coriolis parameter which is the same as f in equation 2.2; g = gravitational acceleration; P = atmospheric pressure at the free surface given by equation 2.1; ρ = water density; τ_{wx} and τ_{wy} = surface wind stress in the x and y directions, respectively, and τ_{bx} and τ_{by} = bottom shear stress in the x and y directions, respectively. The depth-averaged cross-shore velocity, U/D, is taken to be positive onshore. The depth-averaged alongshore velocity, V/D, is positive in the positive y-direction.

To simplify equations 2.4 to 2.6, use is made of the Bathystrophic Storm Tide Theory based on the following approximations (Shore Protection Manual 1977):

- Depth contours are straight and parallel to the shoreline and $U = 0$ is assumed. This approximation and the quasi-steady flow approximation in equation 2.6 yield that V is approximately independent of y.
- The alongshore variations of P and η are assumed to be small and neglected in equation 2.5.

Based on these approximations, equations 2.4 and 2.5 are simplified as

$$\frac{\partial \eta}{\partial x} = \frac{fV}{gD} - \frac{1}{\rho g}\frac{\partial P}{\partial x} + \frac{\tau_{wx}}{\rho g D} \quad (2.7)$$

$$\frac{\partial V}{\partial t} = \frac{\tau_{wy} - \tau_{by}}{\rho} \quad (2.8)$$

which are solved numerically to obtain the cross-shore and temporal variations of η and V. The numerical method adopted here is essentially the same as that used in Shore Protection Manual (1977).

The surface wind stresses are expressed as (Shore Protection Manual 1977)

$$\tau_{wx} = \rho k |W| W_x \quad (2.9)$$

$$\tau_{wy} = \rho k |W| W_y$$

where W = wind speed at elevation of 10 m above the water surface; W_x and W_y = 10-m wind velocity components in the x and y directions; and k = empirical wind stress coefficient which depends on W below the critical wind speed taken as 14 knots.

The values of W_x, W_y and $W = (W_x^2 + W_y^2)^{1/2}$ are obtained from the gradient wind speed U_{gr} given by equation 2.3 and its direction together with the atmospheric boundary layer adjustments on the water surface and near the land (Kriebel 1982).

The bottom stress is expressed as

$$\tau_{by} = \frac{\rho K |V| V}{D^2} \tag{2.10}$$

where K = bottom friction coefficient of the order of 0.001.

2.3 Hurricane Wave Model

The empirical method presented in the Shore Protection Manual (1977) is employed to determine the maximum deep-water significant wave height at the point of maximum wind in the hurricane. The deep water significant wave height for any point in the wind field is then approximated by a simple scaling procedure in which the local wave height is assumed to be proportional to the local wind speed. Changes in wave height due to shoaling and bottom friction losses are neglected in the offshore region. The local wind direction relative to the shore normal is considered to account for the effect of wave refraction crudely. The cross-shore irregular wave transformation in the nearshore region is computed using CSHORE2 as described in Section 3.1.

The maximum significant wave height in deep water at the point of maximum wind is empirically estimated as:

$$(H_o)_{max} = 16.5\, exp\left(\frac{R\Delta P}{100}\right)\left[1.0 + \frac{0.208 V_f}{\sqrt{U_R}}\right] \qquad \text{in feet} \tag{2.11}$$

where R = radius of maximum wind in nautical miles; $\Delta P = (P_n - P_o)$ in inches of mercury with P_n and P_o = peripheral and central pressures of the hurricane, respectively; U_R = maximum wind speed in knots based on equation 2.3 and reduced by the atmospheric boundary and land effects; and V_f = forward speed of the hurricane in knots.

The local significant wave height, which is smaller than $(H_o)_{max}$ given by equation 2.11, might be obtained by considering the local wind velocity and direction at the point of interest on the shoreline (Kriebel 1982). The ratio of the local wind speed, U_r, to the maximum wind speed, U_R, is used to reduce the maximum deep water wave height to account for the primary effect of local wind wave generation. The resulting local significant deep-water wave height is multiplied by the cosine of the angle between the local wind vector and the shore normal to account for the effect of wave refraction.

The local significant wave height under the local wind speed U_r blowing in the direction θ_o counterclockwise from the shore normal coordinate taken to be positive offshore, is crudely estimated as

$$H = (H_o)_{max} \frac{U_r}{U_R} \left|\cos \theta_o\right|^{1/2} \qquad 90^\circ < \theta_o < 270^\circ \tag{2.12}$$

where the range $90^\circ < \theta_o < 270^\circ$ corresponds to the local wind with a positive onshore component. For offshore wind with $-90^\circ < \theta_o < 90^\circ$, $H = 3$ feet is assumed as a typical height for offshore wind. The significant wave period T corresponding to equation 2.12 is assumed to be given by (Shore Protection Manual 1977)

$$T = 2.13\sqrt{H} \qquad \text{in seconds} \tag{2.13}$$

where H = significant wave height in feet.

2.4 Monte Carlo Simulation

Meteorological data of historical probability distributions for the hurricane parameters discussed in Section 2.1 are available at some sites. The occurrence of each of these parameters is in most cases independent of the occurrence of other parameters. Therefore, a hypothetical hurricane may be simulated numerically by a randomly generated combination of the parameters, where each parameter is based on its historical probability distribution. If enough random combinations of the parameters are evaluated, the entire sample group of hypothetical hurricanes should be statistically representative of those hurricanes occurring in nature over a long period of time in the absence of long-term climate changes. A Monte Carlo simulation is adopted here to generate random combinations of the hurricane parameters.

Unlike conventional approaches based on joint probability techniques, which analyze a finite number of parameter combinations, the Monte Carlo technique assumes that an infinite number of hypothetical hurricanes exist. In each hurricane simulation, different random numbers are used to select each storm parameter from the continuous probability distributions except for the number of hurricanes per year, as described first in the following.

2.4.1 Poisson Distribution

It is assumed that the number n of storms arriving a specific coastline in one year follows a Poisson distribution [Benjamin and Cornell (1970); Scheffner *et al.* (1996)]

$$P_n = \frac{\nu^n exp(-\nu)}{n!} \qquad \text{for } n = 0, 1, 2, \ldots, \infty \tag{2.14}$$

where P_n = probability of having n storms per year; and ν = average number of storms per year.

Example simulations will be made subsequently for Panama City, Florida, for which $\nu = 0.7$. The values of P_n calculated using equation 2.14 with $\nu = 0.7$ are listed in Table 2.1. The cumulative probability for given n is the sum of the probabilities P_m with $m \leq n$. The probability P_n is practically zero for $n \geq 6$. The value of $500P_n$ listed in this table is the expected number of years having n storms with $n = 0, 1, 2, 3, 4,$ and 5.

Table 2.1: Poisson Distribution for Average Storm Number $\nu = 0.7$.

n	P_n	Cumulative Probability	$500P_n$
0	0.4966	0.4966	248.3
1	0.3476	0.8442	173.8
2	0.1217	0.9659	60.9
3	0.0284	0.9943	14.2
4	0.0050	0.9993	2.5
5	0.0007	1.0	0.3

For each simulation year, a random number between 0.0 and 1.0 is generated from the uniform probability distribution. This number is compared with the cumulative probability which is also in the range of 0.0 to 1.0. Using Table 2.1 as an example, if the random number generated is less than 0.4966, $n = 0$. If the random number lies between 0.4966 and 0.8442, $n = 1$. This procedure is followed up to the range between 0.9993 and 1.0 for which $n = 5$.

Assuming that each year is independent statistically, the number of storms for each year is generated using a random number generator for the duration of the numerical simulation which is taken to be 500 years so as to estimate the storm surge and significant wave height for the 100-yr recurrence interval.

Ten 500-yr simulations are performed here in order to determine the consistency and variability of the simulated frequency distributions. Table 2.2 shows the number of storms for each of the ten 500-yr simulations. The average number of storms is 354 in comparison to the expected number, $500\nu = 350$. The degree of variability is apparent in Table 2.2 and the ten 500-yr simulations may be necessary for reliable estimates of extreme events. Table 2.2 also lists the number of time levels used for each of the 500-yr simulations as will be explained in Section 2.5.7.

Table 2.2: Number of Storms for Each of Ten 500-yr Simulations for $\nu = 0.7$.

Simulation N500	No. of storms	No. of time levels
1	354	6370
2	380	7406
3	323	6389
4	352	7059
5	377	6990
6	350	7119
7	367	7238
8	321	6234
9	368	8248
10	347	7111
Average	354	7016

2.4.2 Numerical Generation of Hurricane Parameters

Each of the parameters characterizing a hurricane is generated numerically using a random number generator from the uniform probability distribution between 0.0 and 1.0. These parameters vary continuously unlike the discrete number of storms. A hurricane parameter is denoted by h_p, and the corresponding cumulative probability distribution is $F(h_p)$, which is assumed to increase monotonically with the increase of h_p. A random number is denoted by r and its cumulative probability distribution is given by $F(r) = r$ for $0 \leq r < 1$. To generate the unique value of h_p for the specific random number r, h_p is assumed to increase monotonically with the increase of r. Then, the cumulative probabilities of the two statistical variables must be the same [e.g., Benjamin and Cornell (1970)]

$$F(h_p) = F(r) = r \tag{2.15}$$

which implies that the value of the generated random number equals the cumulative probability of h_p. The corresponding value of h_p can hence be found from the known function $F(h_p)$ estimated from available historical data. The examples of $F(h_p)$ based on linear interpolations will be presented in Section 2.5. The advantage of the Monte Carlo simulation is that $F(h_p)$ can be any function as long as $F(h_p)$ increases monotonically with the increase of h_p.

The following hurricane parameters, denoted h_p in equation 2.15, are generated in this manner:

- Central pressure, P_o
- Radius to maximum wind, R_{max}
- Forward speed, V_f
- Direction of hurricane translation, which classifies a storm either as landfalling or alongshore
- Landfall location relative to the shoreline point of interest or offshore distance of the intersection point between the alongshore storm track and the cross-shore line from the shoreline point of interest.

It is noted that two parameters are required to specify the storm track which is assumed to be a straight line. The values of P_o, R_{max}, and V_f are assumed to be constant along the storm track.

The astronomical tide during a storm is assumed to vary sinusoidally with time and is simply added to the storm surge computed using equations 2.7 and 2.8. The amplitude and phase of the astronomical tide are treated as statistical parameters, whereas the tidal period is assumed to be 12.4 hr. The tidal amplitude is generated numerically from the specified cumulative probability distribution in the same way as the hurricane parameter h_p. The tidal phase is assumed to be random and distributed uniformly between 0.0 and 2π. The phase is hence obtained by multiplying a random number between 0.0 and 1.0 by 2π.

The initial rise of a hurricane is not well understood but is assumed to be constant and determined uniquely by the direction of the hurricane translation (Kriebel 1982). The constant initial rise is reduced to zero as the hurricane center travels on land. The assumed initial rise is added to the sum of storm surge and astronomical tide. The total free surface elevation above the mean sea level is called storm tide here. The storm tide includes all the effects related to a hurricane except for the effects of wind waves which cause wave set-down outside the surf zone and wave setup in the surf zone.

2.4.3 Exceedance Probability

The time series of the storm tide, significant wave height and period at the most landward node used for the computation are stored for each storm. A conventional approach utilizes only the maximum values of the storm tide and significant wave height during each storm. The computed maximum storm tide and maximum significant wave height are ranked in descending order from the highest to the lowest. The exceedance probability P_E for the value of rank M is estimated as

$$P_E = \frac{M}{\nu N} \tag{2.16}$$

where ν = average number of storms per year; N = number of years for the simulation; and νN = average number of storms during the N-year simulation. For the example shown in Tables 2.1 and 2.2, $\nu = 0.7$, $N = 500$, and $\nu N = 350$. The exceedance probability P_E given by equation 2.16 is based on the maximum value for each storm and would reduce to the standard form with $\nu = 1$ if the annual maximum value were used to estimate the exceedance probability.

The recurrence interval or return period for the value of rank M is given by

$$T_r = \frac{N}{M} = \frac{r}{P_E} \tag{2.17}$$

where T_r = recurrence interval in years for the value of rank M based on the maximum value for each storm; and r = average interval between storms given by $r = \nu^{-1}$. The recurrence interval T_r given by equation 2.17 is consistent with the equation given by Sarpkaya and Isaccson (1981) and would reduce to the standard form with $r = 1$ if the annual maximum value were used to estimate the recurrence interval.

2.5 Input Data to CYCLONE

Reasonable estimates of the cumulative probability distributions for the hurricane parameters are necessary for the present Monte Carlo simulation. Based largely on the earlier studies using data from 1871 to 1972, Kriebel (1982) estimated these cumulative probability distributions at Panama City, Florida. His estimated distributions are adopted here as an example of the application of the Monte Carlo simulation method developed in this study.

2.5.1 Storm Translation Direction and Landfall

The direction of storm translation, defined at the point of landfall for landfalling storms or at the crossing of the cross-shore line for alongshore storms, was estimated using the polar diagram of the directional distribution of hurricanes and tropical storms. The cumulative probability distribution for the storm translation direction, $zeta$, used here is shown in Fig. 2.2. The angle $zeta$ is defined as the counterclockwise angle of the storm direction from the cross-shore coordinate x, taken to be positive offshore as shown in Figs. 2.3 and 2.4.

In Fig. 2.2 and subsequent figures, estimated discrete values indicated by open circles are connected by straight solid lines. The storm is assumed to be landfalling if $110° \leq zeta \leq 250°$ and alongshore otherwise.

To identify the storm track relative to the shoreline point of interest located at $x = 0$ and $y = 0$ where y is the alongshore coordinate, it is necessary to define the landfall location, Y_c, along the y axis or the crossing point, X_c, along the x axis. The assumed cumulative probability distributions for X_c and Y_c are shown in Figs. 2.3 and 2.4, respectively, where the width of the continental shelf is 150 nautical miles (1 nautical mile = 1.85 km).

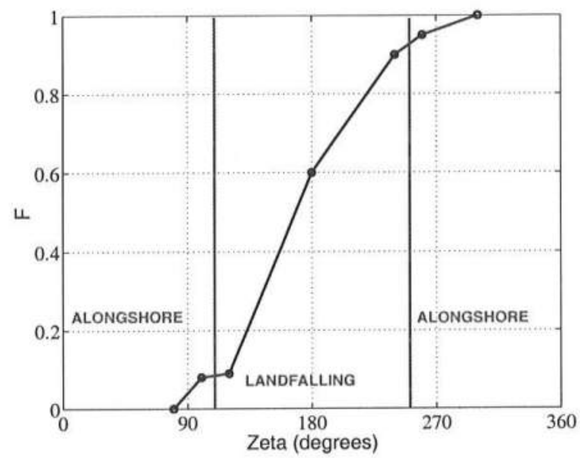

Figure 2.2: Cumulative Probability, F, for Storm Translation Direction, *zeta*.

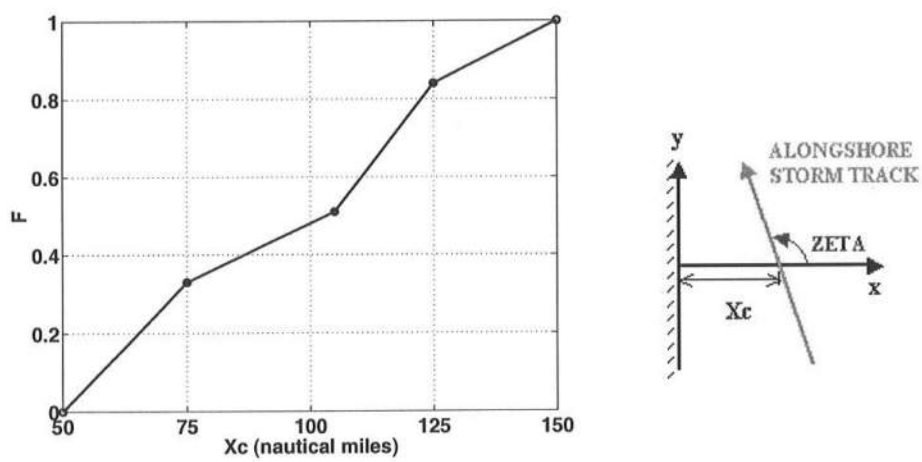

Figure 2.3: Definition Sketch of Offshore Distance X_c and Cumulative Probability, F, of X_c for Alongshore Storms.

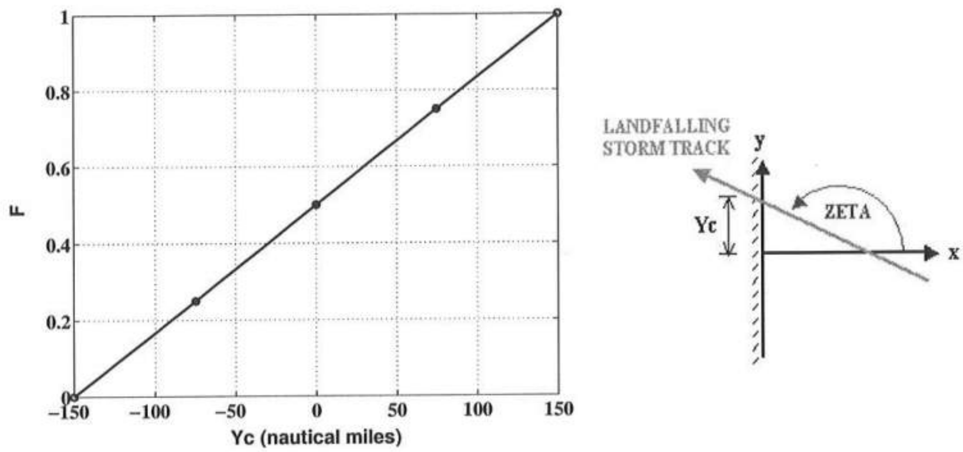

Figure 2.4: Definition Sketch of Alongshore Location Y_c and Cumulative Probability, F, of Y_c for Landfalling Storms.

2.5.2 Storm Radius

The cumulative probability distribution for the radius, R_{max}, to maximum wind is limited to hurricane data only, since a tropical storm rarely exhibits a well defined radius to maximum wind. It is however assumed that hurricanes and tropical storms have the same cumulative probability distribution shown in Fig. 2.5.

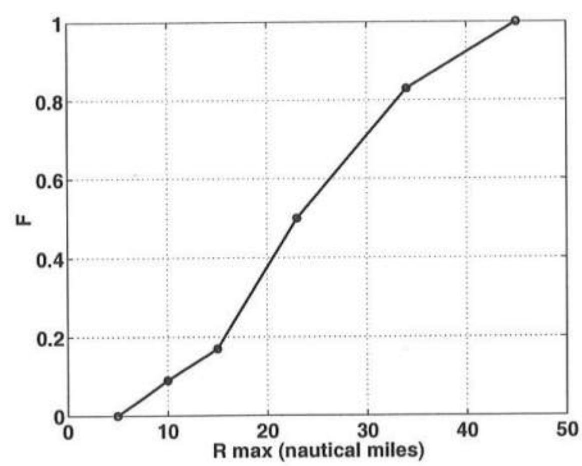

Figure 2.5: Cumulative Probability, F, for Radius, R_{max}, to Maximum Wind.

2.5.3 Storm Central Pressure

The central pressure is the most important storm parameter. The cumulative probability distribution for central pressure at Panama City is shown in Fig. 2.6, where 1 in. Hg = 3,386 N/m^2.

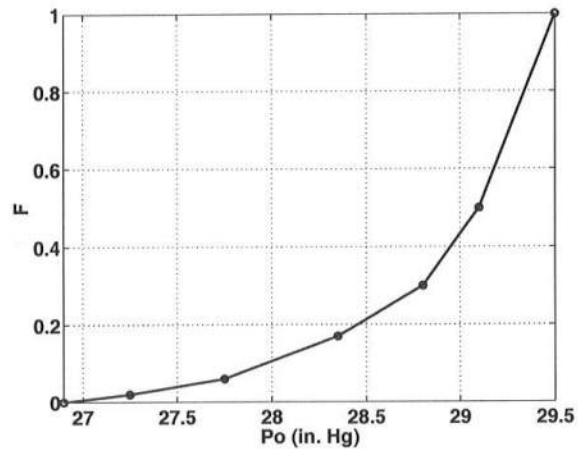

Figure 2.6: Cumulative Probability, F, for Central Pressure, P_o in Inches of Mercury.

2.5.4 Storm Forward Speed

Unlike other parameters, forward speed is often different for the two classes of storms. Alongshore storms in the Gulf of Mexico are usually slower than landfalling storms due to the storm interaction with land. On the East Coast, they are typically faster than landfalling storms due to the effects of increasing latitude. The cumulative distributions for landfalling and alongshore storms are assumed to be the same for simplicity as shown in Fig. 2.7 where 1 knot = 0.515 m/s.

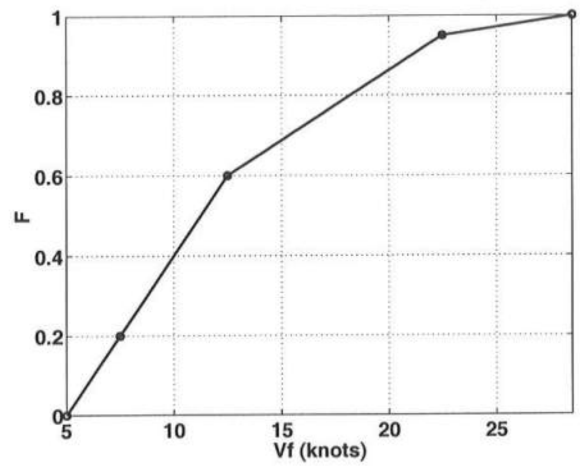

Figure 2.7: Cumulative Probability, F, for Forward Velocity, V_f in Knots.

2.5.5 Astronomical Tide

The astronomical tide is linearly superimposed on storm surge as explained in Section 2.4.2. Daily maximum tidal amplitudes for August, September and October, are assumed representative of the tidal amplitudes that would occur during the 500-yr simulation period. The cumulative probability for the tidal amplitude at Panama City is shown in Fig. 2.8 where 1 ft = 0.305 m.

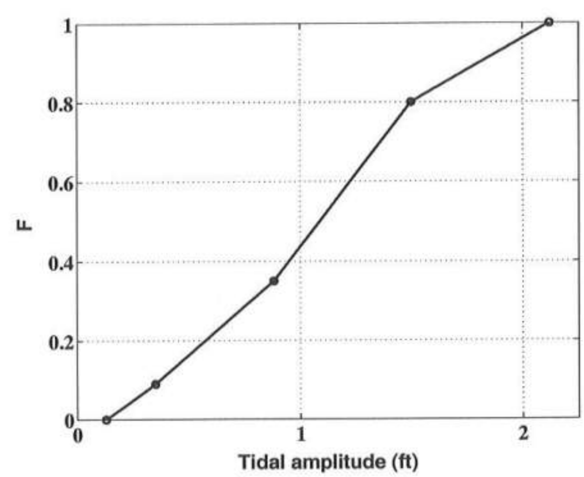

Figure 2.8: Cumulative Probability, F, of Astronomical Tidal Amplitude in Feet.

2.5.6 Initial Water Level Rise

The initial rise is an important component of the total surge elevation. Based on earlier studies for the Florida Panhandle, if a storm strikes the coast normally or within 30 degrees of normal, then the initial rise is assumed to be the maximum and equal to 2 ft. If the angle at which the hurricane strikes the coast is in the range of 30 to 60 degrees to either side of the shore normal, then initial rise of 1 ft is assigned. Otherwise the initial rise is set equal to zero. Storm tide is defined here as the sum of storm surge, astronomical tide and initial rise.

2.5.7 Bathymetry of Continental Shelf

The Florida Panhandle is used here as an example because of the availability of the historical hurricane data for the Pensacola-Panama City area.

Fig. 2.9 shows the depth contours on the continental shelf with the cross-shore line used for the computation of storm surge.

Fig. 2.10 shows the bottom profile used as input to the numerical model CYCLONE which uses feet for the vertical lengths and nautical miles for the horizontal distances. The bottom slope decreases landward and is approximately 1:800 in shallow water.

The finite difference method explained in Shore Protection Manual (1977) is used to solve equations 2.7 and 2.8 and predict the storm surge, η, along the profile shown in Fig. 2.10. Use is made of the constant time step $\Delta t=$ 0.5 hr and the constant cross-shore grid spacing $\Delta x=$ 5 nautical miles. The number of cross-shore nodes is 31. The most landward node is located in the water depth of 16.4 ft (5 m) below the mean sea level. The most seaward node is located at a distance of 150 nautical miles in the depth of 3,281 ft (1,000 m) where the components of storm

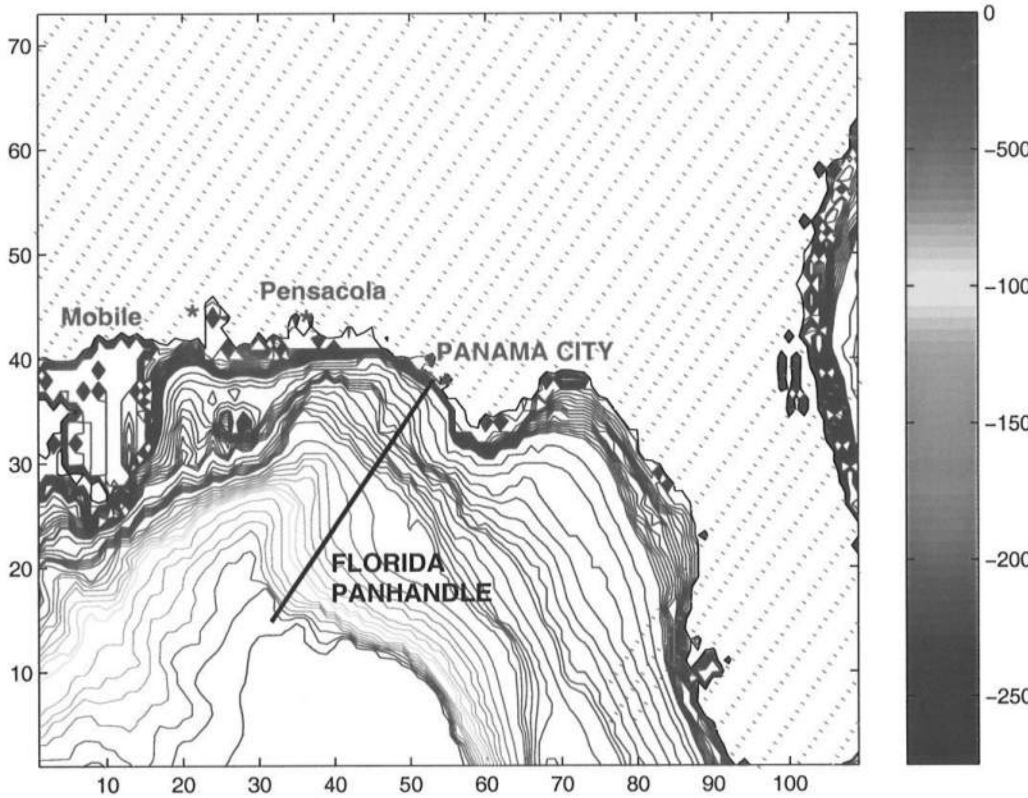

Figure 2.9: Continental Shelf and Bathymetry Offshore Panama City.

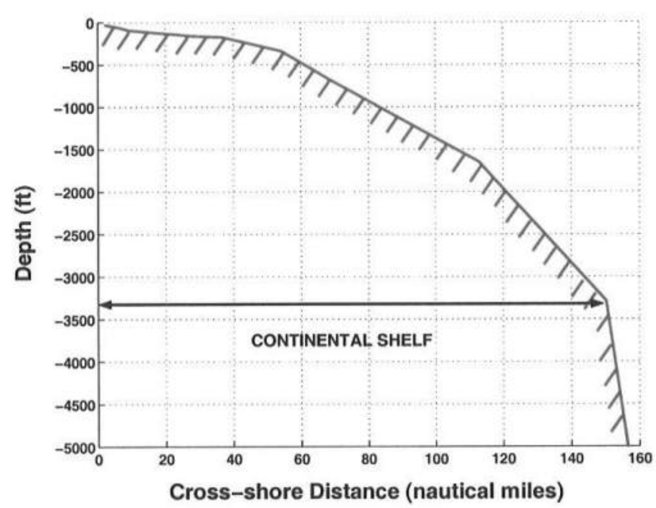

Figure 2.10: Bottom Profile Along Cross-Shore Line in Fig. 2.9.

surge due to the cross-shore wind stress and the Coriolis effect in equation 2.7 are assumed to be negligible.

The initial location of a hurricane at time $t = 0$ is selected at a distance of more than 150 nautical miles from the landfall point at $x = 0$ and $y = Y_c$ in Fig. 2.4 or the crossing point at $x = X_c$ and $y = 0$ in Fig. 2.3. The alongshore volume flux V in equation 2.8 may hence be assumed to be zero at $t = 0$. The center of the hurricane is tracked along its straight path at each time step to estimate the atmospheric pressure using equation 2.1 and the wind speed and direction using equation 2.3 with Fig. 2.1 along the cross-shore line at $y = 0$. The computation is continued until the hurricane center moves out of the domain of influence which is taken to be 150 nautical miles alongshore from the cross-shore line, 150 nautical miles offshore and 50 miles over the land. The computation is also terminated when the computed storm tide (the sum of storm surge, astronomical tide and initial rise) becomes less than -3 ft.

The significant wave height and period at the most landward node are estimated using equations 2.12 and 2.13 during the computation of the storm tide. The computed significant waves do not account for depth-limited wave breaking and are regarded as the significant waves outside the surf zone.

The time series of the storm tide and the significant wave height and period at the most landward node computed at an interval of 0.5 hr for each storm during 500 years are stored and used as input to the numerical model CSHORE2 as explained in Section 3.2. The number of values stored for each of the storm tide, wave height and wave period has been listed in Table 2.2 for each of the ten 500-yr simulation. On the average, 7016 values are stored for 354 storms. The average storm duration is hence approximately 10 hours.

2.6 Computed Results from CYCLONE

The output from CYCLONE for the ten 500-yr simulations in Table 2.2 is discussed in the following where the input used for the simulations has been discussed in Section 2.5. These computed results are intended as examples for the Monte Carlo simulation method developed here. This simulation method can be applied to other sites.

First, the maximum values of the significant wave height and storm tide during each storm are obtained to perform a conventional statistical analysis based on the peak value for each storm. The computed maximum storm tide and significant wave height for each storm at the most landward node used for this computation are analyzed to obtain the recurrence interval (return period) T_r given by equation 2.17 for each of the ten 500-yr simulations identified by the integer $N500 = 1 - 10$.

Fig. 2.11 shows the computed distributions of the maximum storm tide and significant wave height as a function of the recurrence interval in years.

The statistical variability is large in the low probability range of the ten curves. In the higher probability range, where there are many storms of moderate intensity, the simulated results are much more consistent. In this range of the recurrence interval of 5 to 25 years, the ten curves show good agreement and the magnitude of specific events can be determined with more certainty. Below the 5-yr level, more frequent minor storms dominate and the computed results may not be accurate because the hurricane parameters in Section 2.5 were estimated mostly for hurricanes.

The ten 500-year simulations are also averaged to reduce the statistical variability in each individual curve. The averaged curves shown in Fig. 2.12 indicate a 500-yr storm tide of slightly more than 3.5 m and a 500-yr wave height of nearly 13 m.

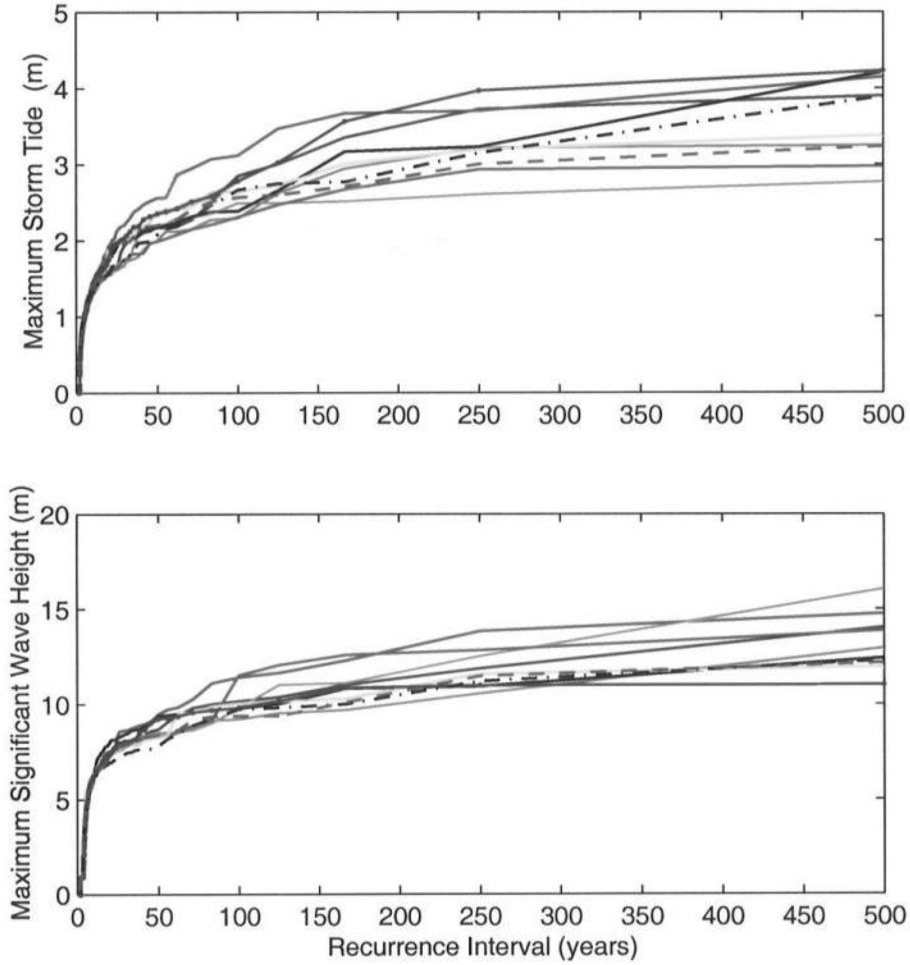

Figure 2.11: Computed Maximum Storm Tide and Significant Wave Height for Ten 500-yr Simulations: (Blue —) N500=1; (Red —) N500=2; (Green —) N500=3; (Cyan —) N500=4; (Magenta —) N500=5; (Black —) N500=6; (Yellow —) N500=7; (Blue -•-) N500=8; (Black -.-) N500=9; (Red - -) N500=10.

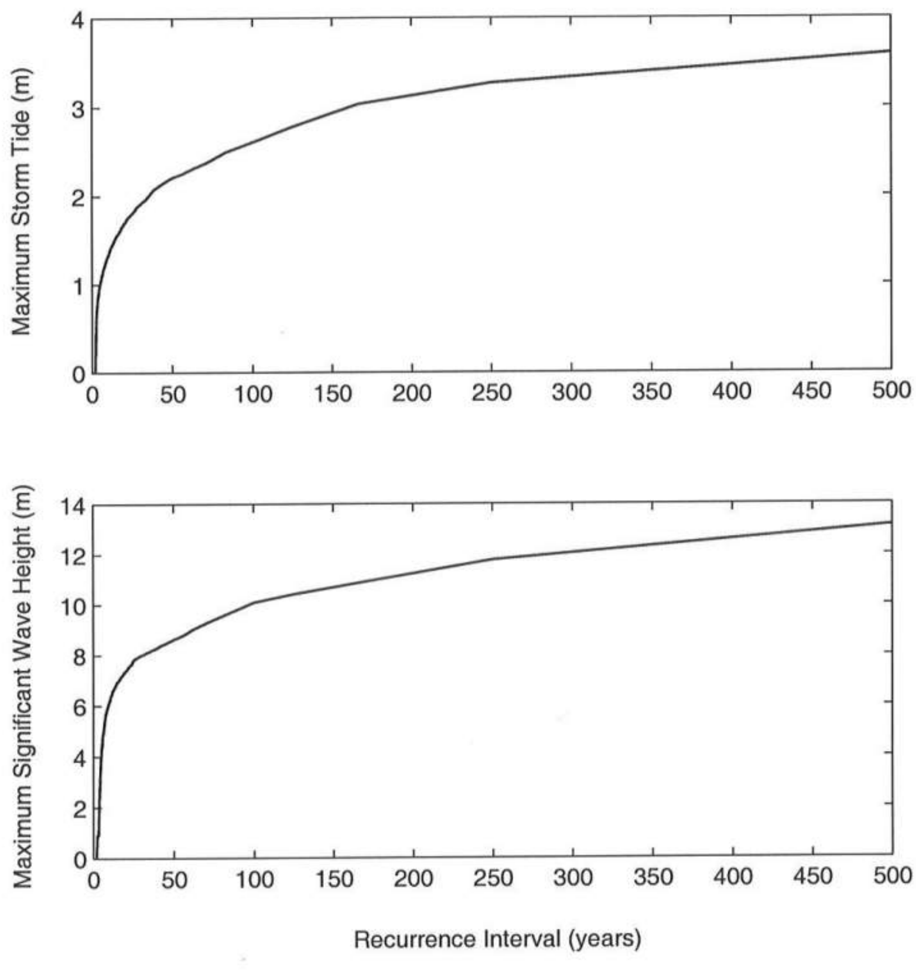

Figure 2.12: Maximum Storm Tide and Significant Wave Height Averaged for Ten 500-yr Simulations.

Second, the correlation of the storm tide and significant wave height at the most landward node at the same time level for each storm during the first 500-yr simulation ($N500 = 1$) is shown in Fig. 2.13, which shows 6370 points in light of Table 2.2. The correlation coefficient is 0.36 and low.

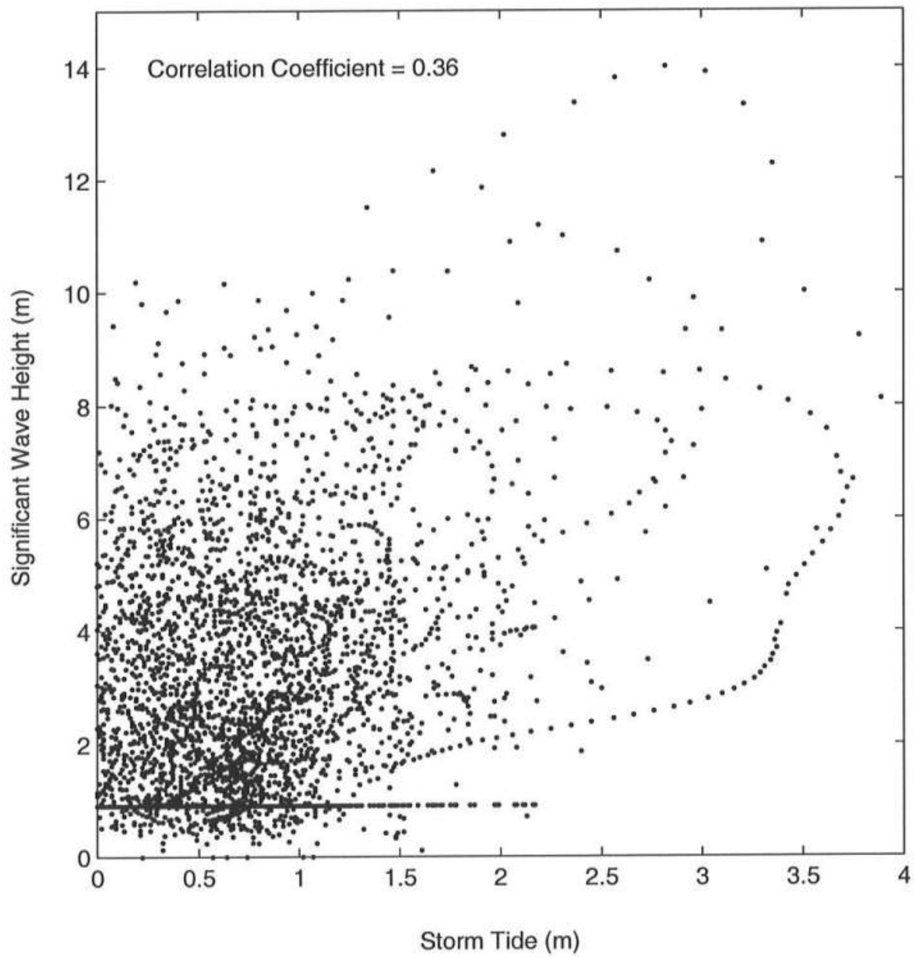

Figure 2.13: Storm Tide and Significant Wave Height at Same Time Level for the First 500-yr Simulation.

Fig. 2.14 shows 70,164 points of the nearshore storm tide and offshore significant wave height for each of the 3,539 storms from the ten 500-yr simulations. The correlation coefficient is 0.37 and low although the quasi-steady methods are used to estimate the waves and storm surge under the same wind field. The low correlation indicates the difficulty in specifying the design storm tide and significant wave height outside the surf zone because the design storm tide and the design significant wave height may not occur simultaneously.

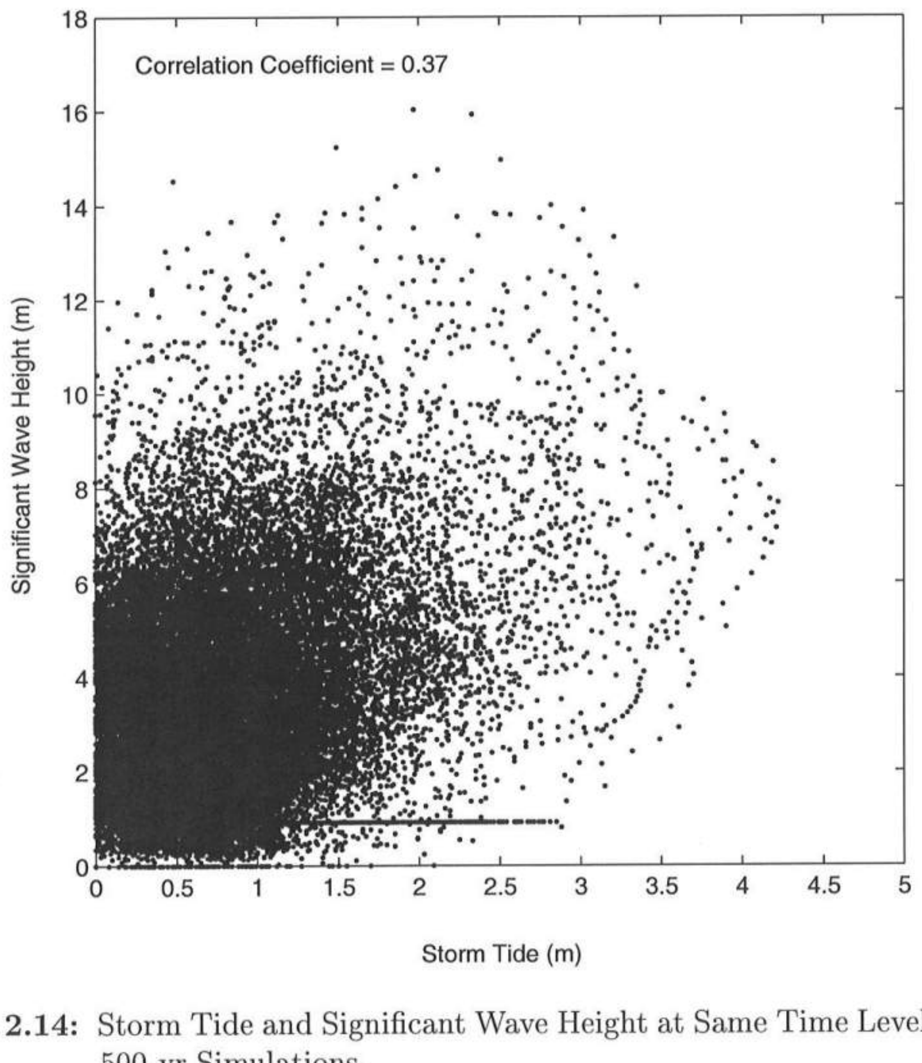

Figure 2.14: Storm Tide and Significant Wave Height at Same Time Level for Ten 500-yr Simulations.

Chapter 3

NUMERICAL MODEL CSHORE: WAVE SETUP AND NEARSHORE WAVES

The storm surge and deep-water wave models in Sections 2.2 and 2.3 have been developed for the entire continental shelf with a large grid spacing. The deep-water wave model cannot predict the irregular breaking wave transformation and wave setup in the water depth of the order of 10 m or less. The cross-shore irregular wave transformation on the beach in front of the breakwater is predicted using the time-averaged irregular wave model CSHORE (Kobayashi and Johnson 1998; Johnson and Kobayashi 2000), which has been shown to be capable of predicting the wave setup and height at the toe of the structure situated inside the surf zone (Kearney and Kobayashi 2000). Since the original computer program computes the cross-shore variations for only one set of the offshore wave height and storm tide, it is modified to allow sequences of storm waves and water levels. The modified computer program is called CSHORE2 and attached in Appendix B.

The storm tide, the offshore significant wave height H_s and the significant wave period T_s in the offshore computation in Section 2.5, stored at an interval of 0.5 hr during each storm, are used as input to CSHORE2 under the assumption of stationarity at each time level. The seaward boundary location for the shallow water computation is taken somewhat arbitrarily to be at the location of the water depth $d = 10$ m below MSL.

The storm tide predicted at the most landward node in water depth of 5 m below the mean sea level is assumed to represent the constant storm tide in the shallow water zone in water depth less than 10 m below the mean sea level since the distance to the shoreline is relatively short and the spatial variation of the storm tide may be neglected.

Since the toe depth at the structure is site-specific, the effect of the toe depth on the wave setup and wave height is examined by predicting the significant wave height and wave setup at the toe depth of 0, 2 and 4 m below the mean sea level.

3.1 Governing Equations

The numerical model CSHORE based on the assumptions of alongshore uniformity and normal incident waves is a nonlinear time-averaged model developed to predict the cross-shore variations of the wave setup, $\bar{\eta}$, and root-mean-squared wave height, H_{rms}, from outside the surf zone to the swash zone where H_{rms} is defined as $H_{rms} = \sqrt{8}\sigma$, with σ = standard deviation of the free surface elevation.The notations used in Chapter 3 are based on those used in the papers related to CSHORE and independent of those used in Chapter 2. The significant wave height H_{mo} is defined by $H_{mo} = \sqrt{2}H_{rms} = 4\sigma$. This model is based on the time-averaged continuity, momentum and energy equations derived by time-averaging the nonlinear equations used in the time-dependent model of Kobayashi and Wurjanto (1992). The time-averaged equations can be solved numerically with much less computation time but require empirical relationships to close the problem. The time-averaged rate of energy dissipation due to random wave breaking is estimated by modifying the empirical formula of Battjes and Stive (1985) to account for the landward increase of $H_{rms}/\bar{h}$ near the shoreline where $\bar{h}$ = mean water depth. The skewness, s, and kurtosis, K, of the free surface elevation included in the time-averaged momentum and energy equations are expressed empirically as a function of $H_{rms}/\bar{h}$.

The derivation of the governing equations including the effect of bottom friction was given by Kobayashi and Johnson (1998). The governing equations without the effect of bottom friction presented by Johnson and Kobayashi (1998) are summarized in the following because the effect of bottom friction is generally negligible except in the swash zone.

The time-averaged cross-shore momentum equation with the overbar denoting time-averaging is written as

$$\frac{dS_{xx}}{dx} = -\rho g\bar{h}\frac{d\bar{\eta}}{dx} \tag{3.1}$$

$$S_{xx} = \frac{1}{8}\rho g H_{rms}^2\left[\left(2n-\frac{1}{2}\right)+C_s\right] \quad ; \quad H_{rms}=\sqrt{8}\sigma \tag{3.2}$$

$$C_s = \sigma_* s - \sigma_*^2 \quad ; \quad \sigma_* = \frac{\sigma}{\bar{h}} \tag{3.3}$$

in which x = cross-shore coordinate taken to be positive landward; S_{xx} = cross-shore radiation stress; ρ = fluid density; g = gravitational acceleration; η = instantaneous free surface elevation above the still water level (SWL) in the absence of waves; $\bar{h}$ = mean water depth including wave setup $\bar{\eta}$; σ = standard deviation of η; s = skewness of η; n = finite-depth adjustments parameter with $n = 1$ in shallow water; and C_s = nonlinear correction term for S_{xx}. For linear progressive waves in finite depth, n is normally expressed as [e.g., Battjes and Stive (1985)]

$$n = \frac{1}{2}\left[1+\frac{2k_p\bar{h}}{\sinh\left(2k_p\bar{h}\right)}\right] \tag{3.4}$$

where k_p = linear wave number corresponding to the spectral peak period T_p outside the surf zone. The cross-shore variation of T_p may be neglected in equation 3.4 because $n = 1$ in shallow water for any reasonable representative wave period used to calculate k_p. The cross-shore radiation stress S_{xx} based on linear wave theory is given by equation 3.2 with $C_s = 0$. C_s is on the order of unity near the still water shoreline and can not be neglected in the swash zone (Kobayashi and Johnson 1998).

The time-averaged wave energy equation is expressed as

$$\frac{d}{dx}(\bar{E_F}) = -\bar{D_B} \tag{3.5}$$

$$\bar{E_F} = \frac{1}{8}\rho g H_{rms}^2 n C_p (1 + C_F) \tag{3.6}$$

$$C_F = \frac{3}{2}s\sigma_* \left(1 - \sigma_*^2\right) + \frac{1}{2}\sigma_*^2 (K - 5) + \sigma_*^4 \tag{3.7}$$

where $\bar{E_F}$ = energy flux per unit width; $\bar{D_B}$ = rate of wave energy dissipation due to wave breaking; C_p = phase velocity based on T_p with $C_p = \sqrt{g\bar{h}}$ in shallow water; C_F = nonlinear correction term for $\bar{E_F}$; and K = kurtosis of η. The finite-depth adjustment is included in equation 3.6 where nC_p in equation 3.6 is the group velocity based on T_p. The cross-shore energy flux $\bar{E_F}$ based on linear wave theory is given by equation 3.6 with $C_F = 0$ where C_F is on the order of unity near the still water shoreline (Kobayashi and Johnson 1998).

The momentum equation 3.1 with 3.2 and the energy equation 3.5 with 3.6 need to be solved numerically to predict the cross-shore variations of the wave setup $\bar{\eta} = (\bar{h} + z_b)$ with z_b = bottom elevation, positive above SWL, and the root-mean-square wave height $H_{rms} = \sqrt{8}\sigma$. These equations reduce to those used in the existing time-averaged models [e.g., Battjes and Stive (1985)] if $C_s = 0$ and $C_F = 0$. To estimate the nonlinear correction terms C_s and C_F using equations 3.3 and 3.7 with $\sigma_* = \sigma/\bar{h}$, the skewness s and the kurtosis K are assumed to be expressed as a function of $H_* = H_{rms}/\bar{h}$

$$s = \begin{cases} 2H_* & \text{for } H_* \le 0.5 \\ 1.5 - H_* & \text{for } 0.5 \le H_* \le 1.0 \\ 0.7H_* - 0.2 & \text{for } 1.0 \le H_* \end{cases} \tag{3.8}$$

$$K = 3 + s^{2.2} \tag{3.9}$$

Finally, the energy dissipation rate $\bar{D_B}$ due to wave breaking in the energy equation 3.5 needs to be estimated. The empirical formula proposed by Battjes and Stive (1985) is adopted here for its simplicity.

$$\bar{D_B} = \frac{\alpha}{4}\rho g f_p Q H_m^2 \tag{3.10}$$

with

$$\frac{Q-1}{\ln Q} = \left(\frac{H_{rms}}{H_m}\right)^2 \tag{3.11}$$

$$H_m = \frac{0.88}{k_p}\tanh\left(\frac{\gamma k_p \bar{h}}{0.88}\right) \tag{3.12}$$

$$\gamma = 0.5 + 0.4\tanh\left(33\frac{H_{rmso}}{L_o}\right) \quad ; \quad L_o = \frac{gT_p^2}{2\pi} \tag{3.13}$$

where α = empirical coefficient recommended as $\alpha = 1$; f_p = spectral peak frequency given by $f_p = T_p^{-1}$; Q = local fraction of breaking waves in the range $0 \leq Q \leq 1$; H_m = local depth-limited wave height; k_p = linear wave number calculated using f_p and $\bar{h}$; γ = empirical parameter determining $H_m = \gamma\bar{h}$ in shallow water; L_o = deep-water wavelength based on T_p; and H_{rmso} = deep-water value of H_{rms} calculated using linear wave shoaling theory with T_p, $\bar{h}$ and H_{rms} specified at the seaward boundary of the numerical model.

The empirical parameter γ is uncertain in light of the field data by Raubenheimer *et al.* (1996) but is estimated using equation 3.12 without any additional calibration. Relatedly, Battjes and Janssen (1978) indicated that $\bar{D_B}$ given by equation 3.10 would underestimate the actual energy dissipation rate and produce $H_{rms} > H_m$ near the shoreline, although equation 3.11 with $Q \leq 1$ requires $H_{rms} \leq H_m$. They recommended use of a cutoff of $H_{rms} = H_m$ when $H_{rms} \geq H_m$. This adjustment leads to $H_{rms} = \gamma\bar{h}$ near the shoreline. However, $H_{rms}/\bar{h}$ is not a constant and increases landward. As a result, equation 3.10 with equations 3.11

to 3.13 is assumed to be valid only in the outer zone $x < x_i$ with x_i = cross-shore location where Q computed by equation 3.11 becomes unity and the still water depth decreases landward in the region $x > x_i$. The latter condition is required for a barred beach to allow $Q < 1$ landward of the bar crest where $Q = 1$ may occur. For the inner zone $x > x_i$, the ratio $H_* = H_{rms}/\bar{h}$ is assumed to be expressed as

$$H_* = \gamma + (\gamma_s - \gamma)x_*^{\beta} \qquad ; \qquad x_* = \frac{x - x_i}{x_s - x_i} > 0 \tag{3.14}$$

where γ_s = value of H_* on the order of two at the still water shoreline located at $x = x_s$; and β = empirical parameter. Kobayashi and Johnson (1998) calibrated $\gamma_s = 2$ and $\beta = 2.2$ using laboratory beach experiments. Equation 3.14 describes the landward increase of H_* from $H_* = \gamma$ at $x = x_i$ to $H_* = \gamma_s$ at $x = x_s > x_i$. For the inner zone $x > x_i$, the momentum equation 3.1 and 3.14 are used to predict the cross-shore variations of $\bar{h}$ and H_{rms}, whereas the energy equation 3.5 is used to estimate $\bar{D_B}$ which must be positive or zero.

3.2 Input to CSHORE2

As mentioned above, the output at the most landward node of the bottom profile shown in Fig. 2.10 for the computation using CYCLONE constitutes the input for the computation using CSHORE2 at each time level (0.5 hr time step) of the CYCLONE computation. The beach slope at Panama City is gentle and assumed to be constant and 1/800. To examine the effect of the beach slope on the wave transformation, computation is also made for a steep slope of 1/40. The assumption of stationary wave conditions may be appropriate for 0.5 hours.

The output from CYCLONE includes H_s = offshore significant wave height, which is assumed to be the same as the significant wave height, $H_{mo} = 4\sigma$; T_s = significant wave period; and S = storm tide which is the sum of storm surge, astronomical tide and initial rise.

The significant waves are assumed to be in the water depth of 10 m below the mean sea level. The storm tide is assumed to be constant in the depth less than 10 m. The wave height H_{mo} is assumed to be less than or equal to $(10 + S)$ m, to account for wave breaking in the still water depth of $(10 + S)$ m.

The input to CSHORE2 at its most seaward node includes H_{rms} = root-mean-square wave height given by $H_{rms} = H_{mo}/\sqrt{2}$; T_p = spectral peak period, which is assumed to be given by $T_p = 1.05T_s$ (Shore Protection Manual 1984). The wave setup at the most seaward node is assumed to be zero where wave set-down is normally small.

The computation based on CSHORE2 is made in the region where the water depth below the mean sea level is in the range of 0 - 10 m using the grid spacing Δx = 10 m over the cross-shore distance of 8,000 m and 400 m for the bottom slopes of 1/800 and 1/40, respectively.

CSHORE2 computes the cross-shore variations of the wave setup $\bar{\eta}$ and the root-mean-square wave height H_{rms}. The mean water depth $\bar{h}$ is the sum of the wave setup $\bar{\eta}$, storm tide S, and water depth below the mean sea level. The wave periods T_s and T_p are assumed to remain the same in the surf zone because no time-averaged model can predict the variations of wave periods.

The computed values of $\bar{\eta}$, H_{rms} and $\bar{h}$ at the water depth 0, 2 and 4 m below the mean sea level are stored at each time level of the CYCLONE computation summarized in Table 2.2. The average number of time levels for the ten 500-yr simulations is 7,016. The computation time for one 500-yr simulation was of the order of minutes due to the use of the computationally-efficient models CYCLONE and CSHORE2.

3.3 Computed Results from CSHORE2

The computed cross-shore variations of $\bar{h}$ and H_{rms} at each time level during the storm are used to find and store the values of wave setup $(\bar{h} - d_s)$, $\bar{h}$ and H_{rms} at the locations of $d = 4$, 2 and 0 m where the toe of a hypothetical structure will be situated. The still water depth d_s in the absence of waves equals the sum of the storm tide specified as input and the local depth d below MSL. The mean water depth $\bar{h}$ is the sum of the still water depth d_s and the wave setup computed by CSHORE.

First, examples of the cross-shore and temporal variations computed by CSHORE2 are presented. Second, the stored time series of $(\bar{h} - d_s)$, $\bar{h}$ and H_{rms} at $d = 4$, 2 and 0 m during each storm for the ten 500-yr simulations are used to obtain the storm setup $(\bar{h} - d)$, defined here as the sum of the storm tide $(d_s - d)$ and the wave setup $(\bar{h} - d_s)$, and the spectral significant wave height H_{mo}, given by $H_{mo} = \sqrt{2}\, H_{rms}$. The maximum values of storm setup, $(\bar{h} - d)$, and significant wave height, H_{mo}, during each storm are ranked and plotted as a function of the recurrence interval for each 500-yr simulation in the same way as in Fig. 2.11.

As mentioned before, two different bottom slopes have been considered in this study. Results for the slopes of 1/800 and 1/40, hereafter referred to as gentle and steep slopes, respectively, are presented in the following separate sections.

3.3.1 Results for Bottom Slope of 1/800

Fig. 3.1 shows the cross-shore variations of the root-mean-square wave height, H_{rms}, and wave setup with the landward distance from the most seaward node located in the water depth 10 m below the mean sea level where the storm tide is 0.61 m and the root-mean-square wave height and spectral period are 4.68 m and 10.4 s, respectively. The wave height decreases landward rapidly and then linearly, partly because the input wave height from the CYCLONE computation does not include depth-limited wave breaking and the wave height predicted by CYCLONE may be too large at this depth. The wave setup increases landward rapidly and then linearly as well.

The water depths of 4, 2 and 0 m below the mean sea level on the bottom slope of 1/800 are located at the landward distances of 4,800, 6,400 and 8,000 m, respectively, from the most seaward node.

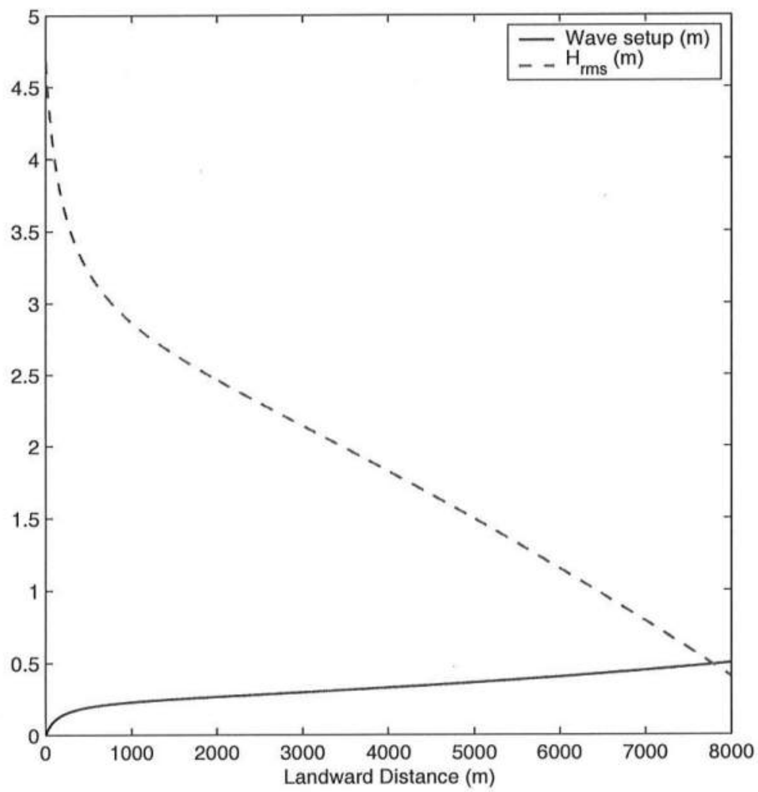

Figure 3.1: Cross-shore Variations of H_{rms} and Wave Setup for the Bottom Slope of 1/800.

Fig. 3.2 shows the computed temporal variations during a storm, with the time step of 0.5 hr, of the significant wave height, H_{mo}, and the mean water depth, $\bar{h}$, including the storm tide and wave setup at the water depths of 4, 2 and 0 m below the mean sea level. This particular storm has caused the maximum values of H_{mo} and $\bar{h}$ at each of the three locations for the first 500-yr simulation (N500 = 1). The significant wave height is almost the same initially at the locations of 2 and 4-m depth before irregular wave breaking occurs at these locations. When the significant wave becomes large, the wave height is essentially limited by the mean depth at these three locations and the temporal variations of H_{mo} and $\bar{h}$ are similar.

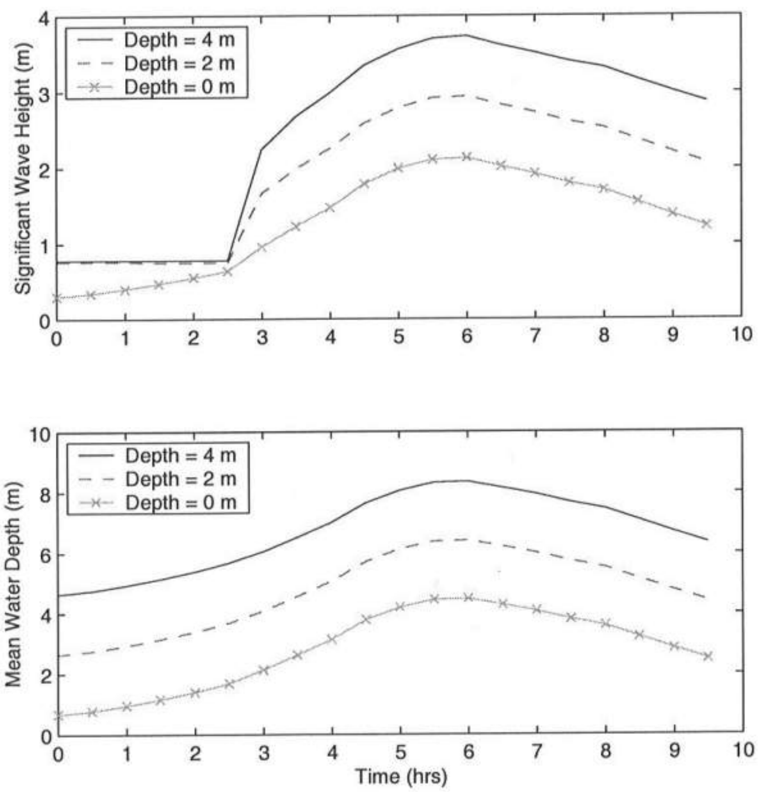

Figure 3.2: Temporal Variations of Significant Wave Height and Mean Water Depth for a Specific Storm at Water Depths of 4, 2 and 0 m Below the Mean Sea Level for the Bottom Slope of 1/800.

Fig. 3.3 shows the maximum storm setup and significant wave height as a function of the recurrence interval for the first 500-yr simulation. The maximum storm setup increases with the decrease of the depth because of the landward increase of wave setup.

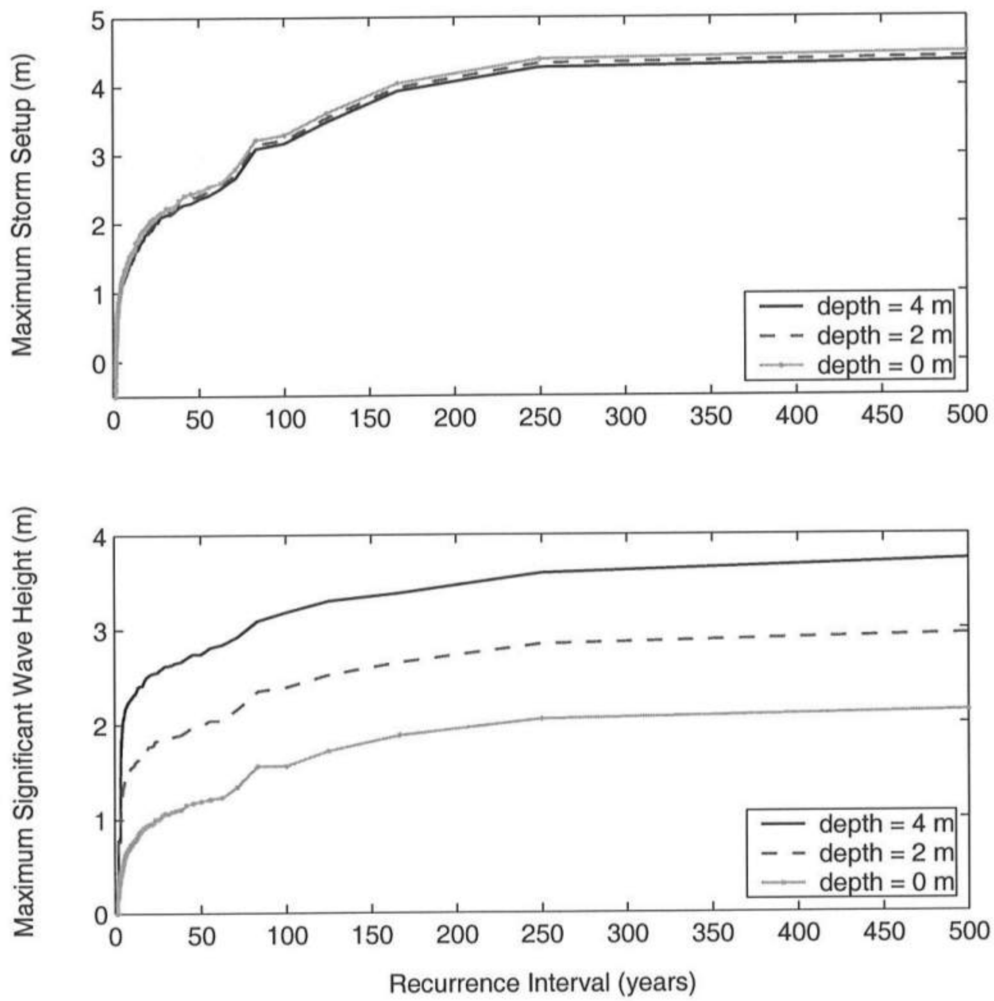

Figure 3.3: Computed Maximum Storm Setup and Significant Wave Height at 0, 2 and 4 m Depths for One 500-yr Simulation for the Bottom Slope of 1/800.

Fig. 3.4 shows the averaged frequency distributions for the ten 500-yr simulations. Fig. 3.5 shows the comparisons of Figs. 2.12 and 3.4, which indicate the effect of wave setup on the landward increase of the mean water level in the presence of waves and the major reduction of the significant wave height due to wave breaking in the surf zone.

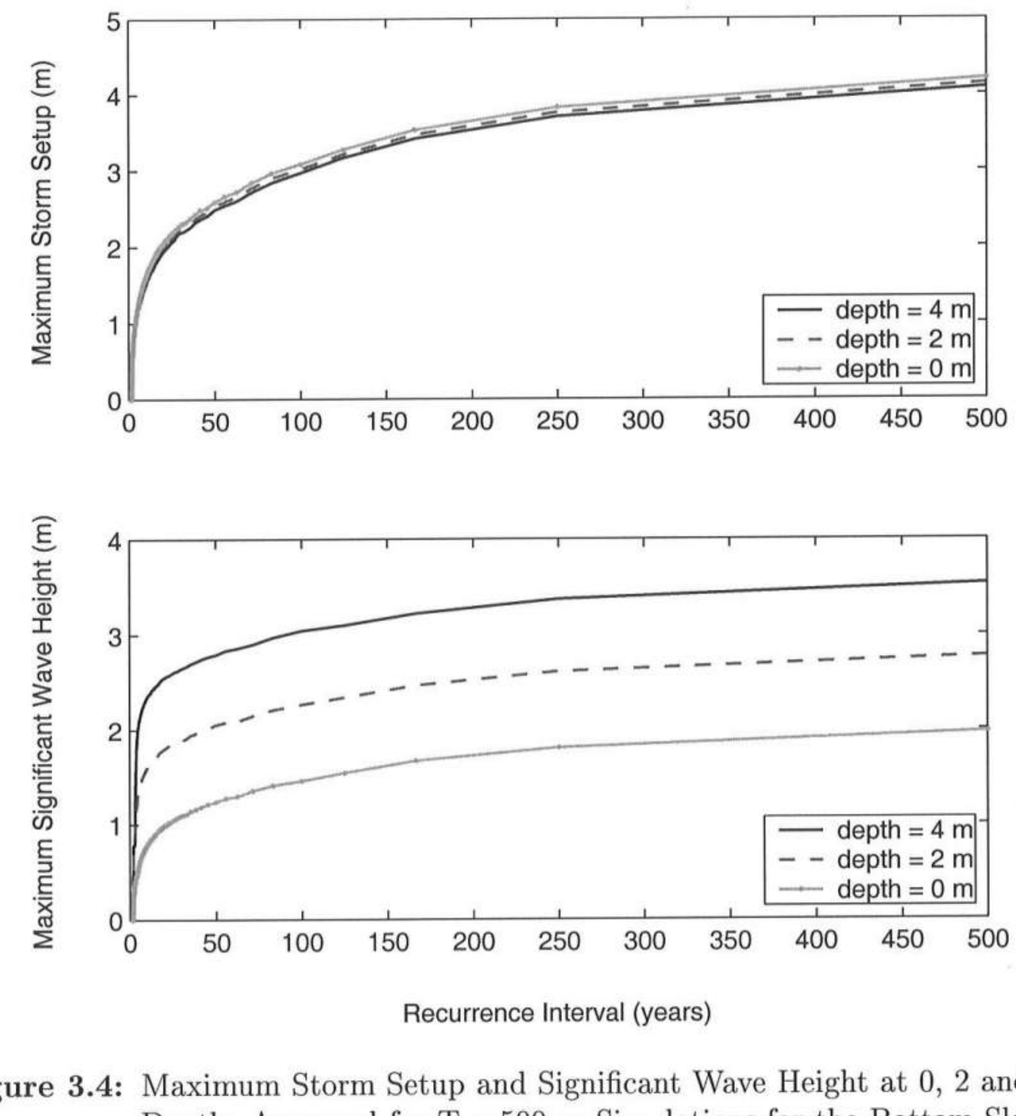

Figure 3.4: Maximum Storm Setup and Significant Wave Height at 0, 2 and 4-m Depths Averaged for Ten 500-yr Simulations for the Bottom Slope of 1/800.

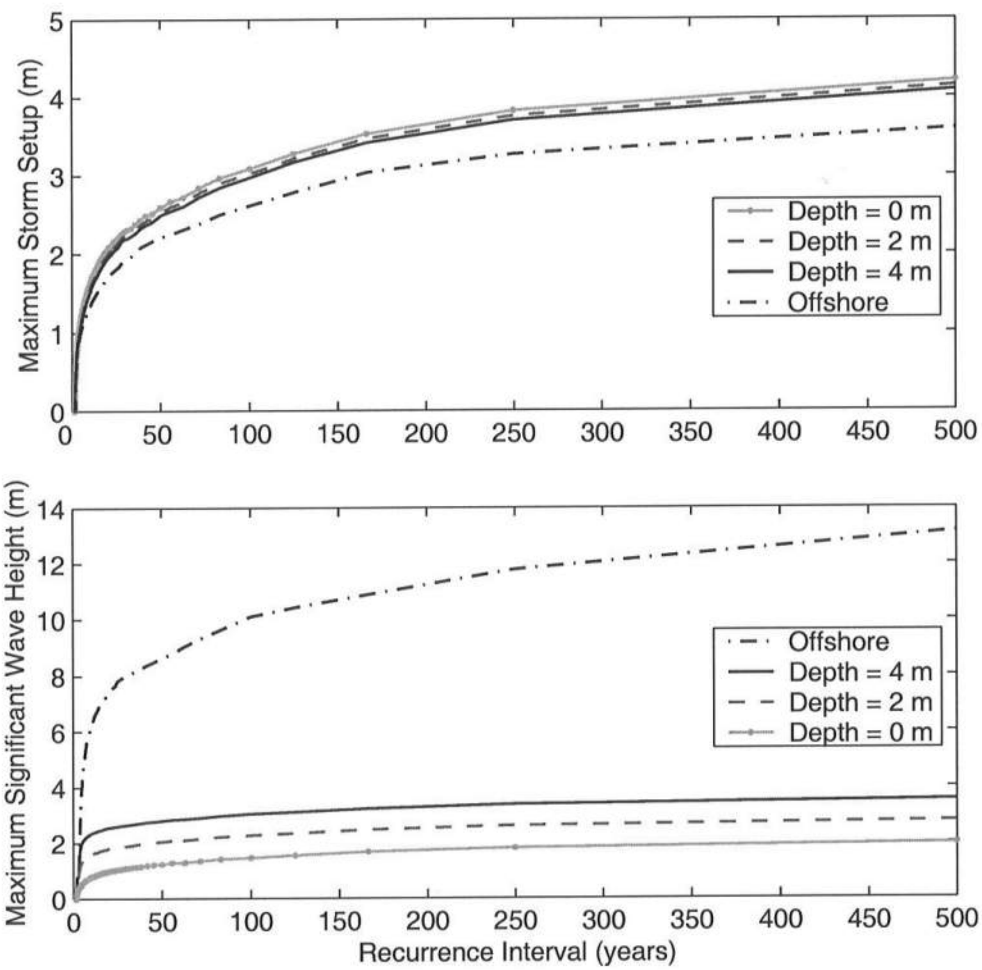

Figure 3.5: Comparisons of Frequency Distributions Shown in Fig. 2.12 (offshore) and Fig. 3.4 (0, 2 and 4-m depths) for the Bottom Slope of 1/800.

3.3.2 Results for Bottom Slope of 1/40

The effects of the beach slope on the wave transformation are examined using the bottom slope of 1/40 instead of the bottom slope of 1/800 which is representative at Panama City, Florida.

The cross-shore variation of the root-mean-square wave height, H_{rms}, and wave setup, $(\bar{h} - d_s)$, and the temporal variations of the significant wave height, H_{mo}, and the mean water depth, $\bar{h}$, are shown in Figs. 3.6 and 3.7, respectively, in comparison to those shown for the gentle slope of 1/800 in Figs. 3.1 and 3.2. Fig. 3.6 shows the essentially linear variations of H_{rms} and wave setup. The water depths of 4, 2 and 0 m below the mean sea level in Fig. 3.7 are located at the landward distances of 240, 320 and 400 m, respectively, in Fig. 3.6.

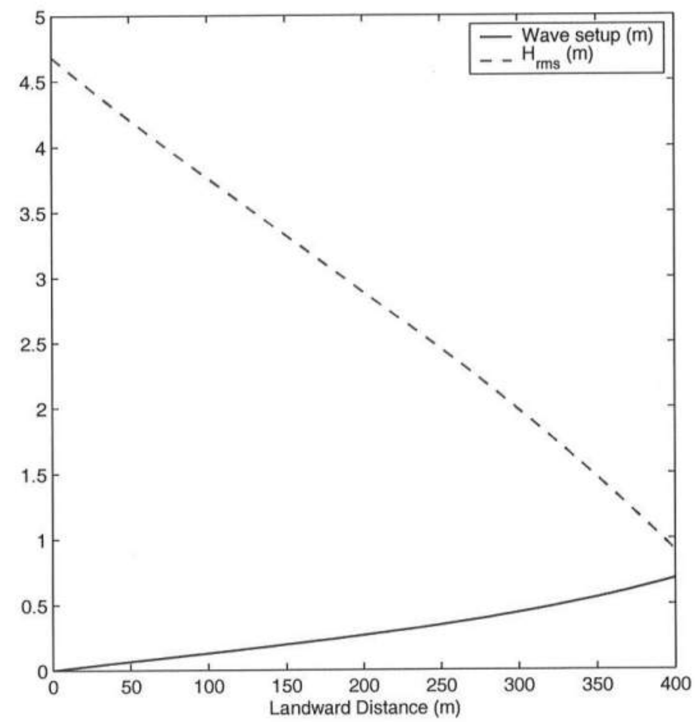

Figure 3.6: Cross-shore Variations of H_{rms} and Wave Setup for the Bottom Slope of 1/40.

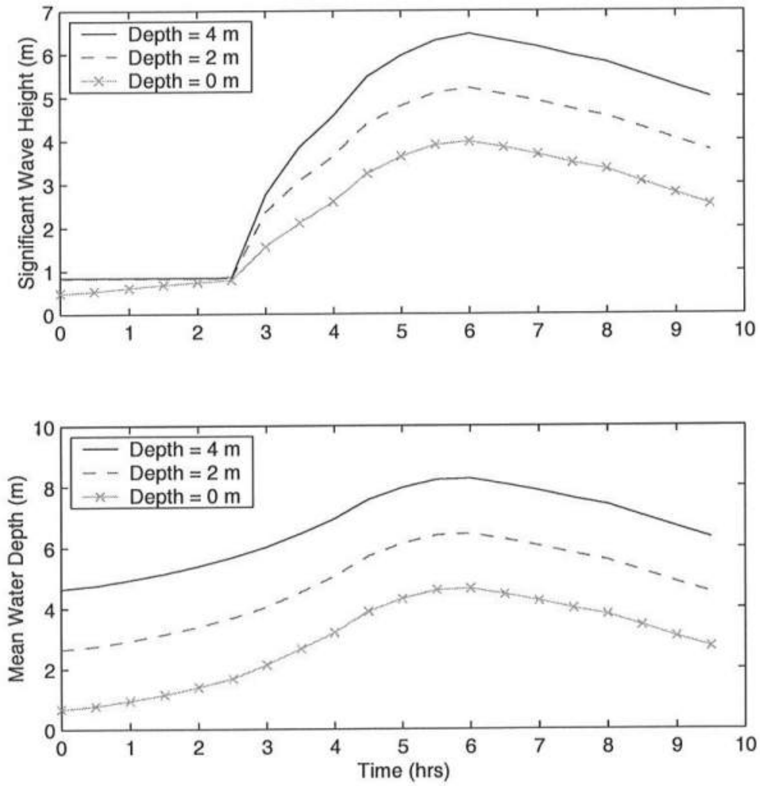

Figure 3.7: Temporal Variations of Significant Wave Height and Mean Water Depth for a Specific Storm at Water Depths of 4, 2 and 0 m Below the Mean Sea Level for the Bottom Slope of 1/40.

The computed results for the steep slope of 1/40 with a much narrower surf zone indicate slightly larger differences of wave setup at d = 4, 2 and 0 m and larger significant wave heights at d = 4, 2 and 0 m. These results are illustrated in Figs. 3.8, 3.9 and 3.10 in the same manner as in Figs. 3.3, 3.4 and 3.5 for the gentle slope of 1/800.

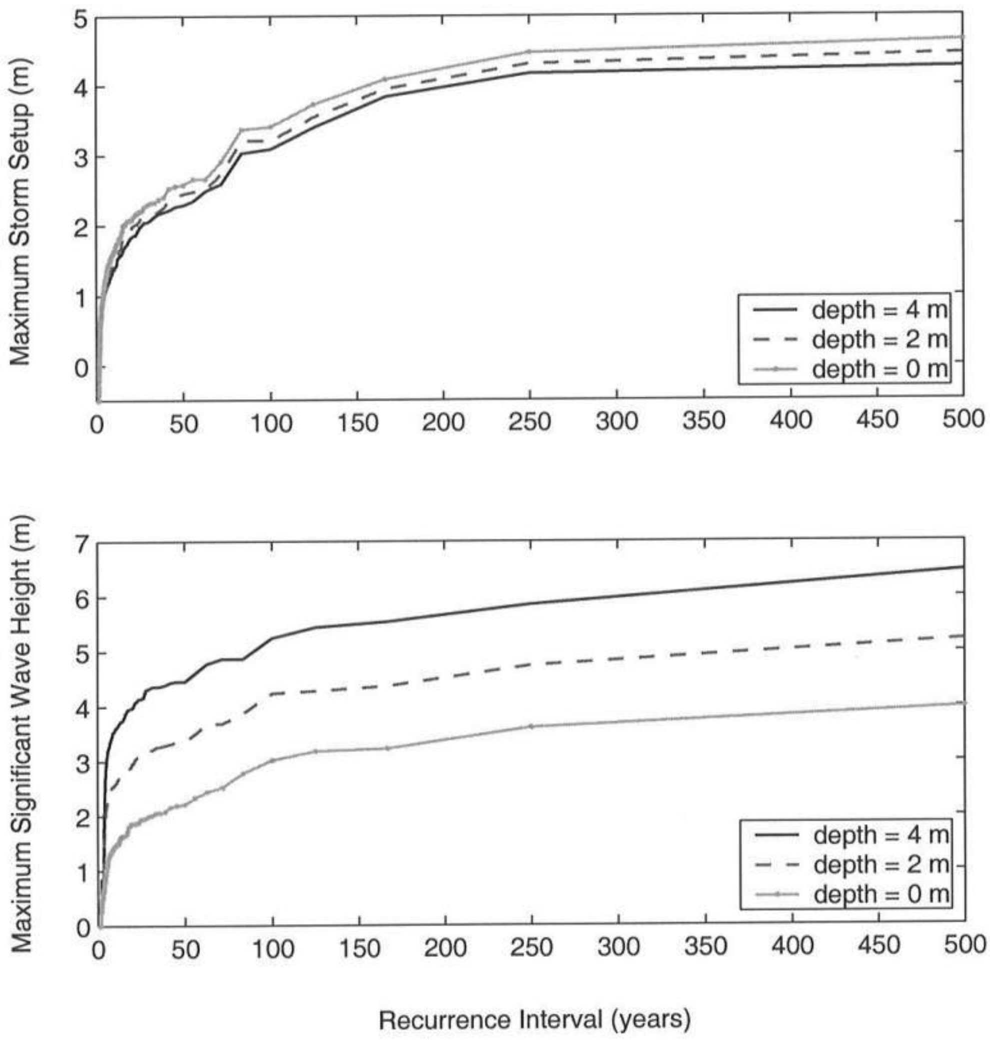

Figure 3.8: Computed Maximum Storm Setup and Significant Wave Height at 0, 2 and 4 m Depths for One 500-yr Simulation for the Bottom Slope of 1/40.

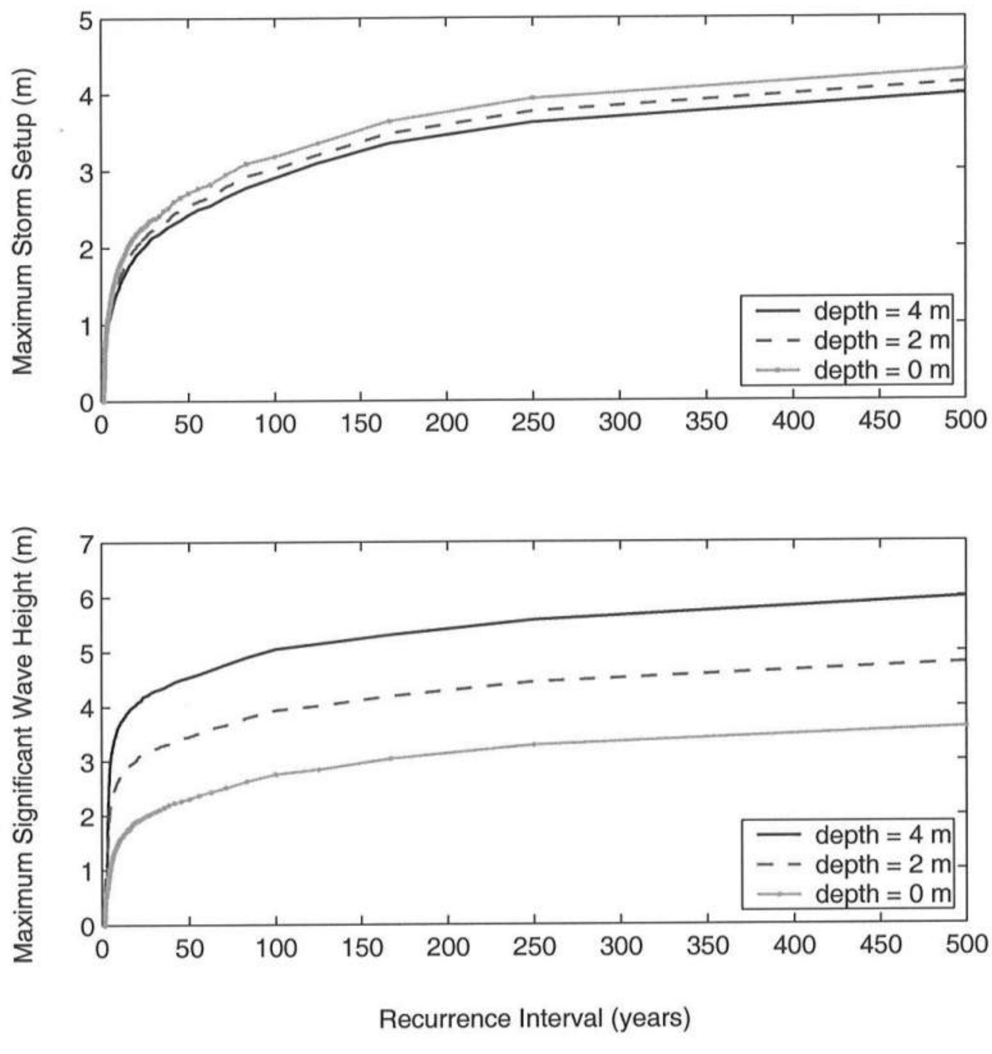

Figure 3.9: Maximum Storm Setup and Significant Wave Height at 0, 2 and 4-m Depths Averaged for Ten 500-yr Simulations for the Bottom Slope of 1/40.

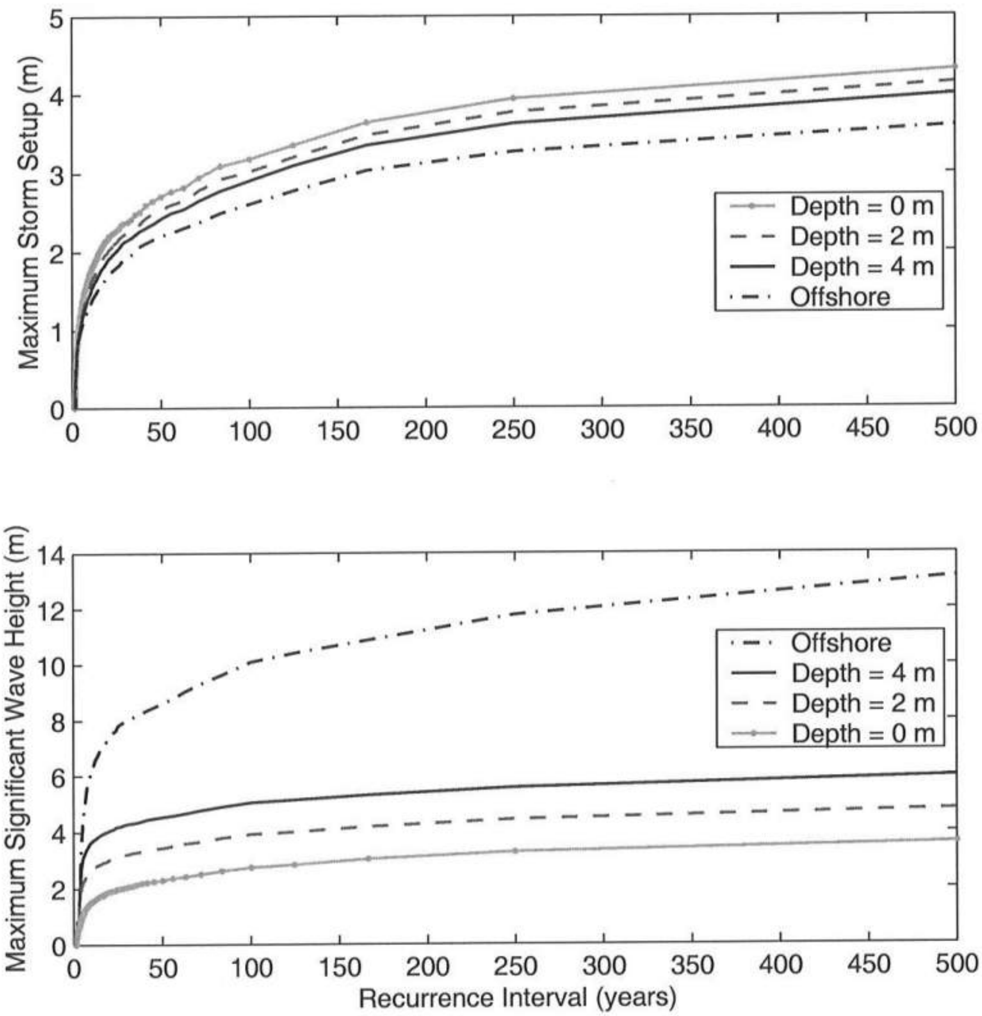

Figure 3.10: Comparisons of Frequency Distributions Shown in Fig. 2.12 (offshore) and Fig. 3.9 (0, 2 and 4-m depths) for the Bottom Slope of 1/40.

3.4 Depth-Limited Breaking Wave Heights

Figs. 3.11 and 3.12 show all the points of the computed significant wave height, H_{mo}, and corresponding mean water depth, $\bar{h}$, at the same time level for the ten 500-yr simulations for the bottom slopes of 1/800 and 1/40, respectively.

The solid straight line in Figs. 3.11 and 3.12 corresponds to

$$H_{mo} = \gamma_b \bar{h} \tag{3.15}$$

which may be regarded as the depth-limited wave height in the inner surf zone. The values of γ_b for the gentle and steep slopes are approximately 0.5 and 0.8 in Figs. 3.11 and 3.12, respectively.

The computed range of γ_b and the increase of γ_b with the increase of the slope are consistent with available field data in inner surf zones (Raubenheimer *et al.* 1996). Fig. 3.11 indicates that equation 3.15 with $\gamma_b = 0.5$ slightly overestimates the significant wave height in the outer surf zone. On the contrary, Fig. 3.12 indicates that equation 3.15 with $\gamma_b = 0.8$ slightly underestimates the significant wave height in the inner surf zone.

To predict the significant wave height H_{mo} in the inner surf zone using equation 3.15 with an appropriate value of γ_b, it is necessary to predict the mean water depth $\bar{h}$ which is the sum of storm tide, wave setup and water depth below the mean sea level. Storm tide is normally predicted by a storm surge model or estimated from available storm surge and astronomical tide data. Wave setup needs to be predicted for estimated offshore wave heights.

These computed results are not verified but indicate the importance of an accurate prediction of irregular breaking waves on an actual beach profile in front of a structure in shallow water. It will also be necessary to predict the beach profile evolution including bar formation during a storm.

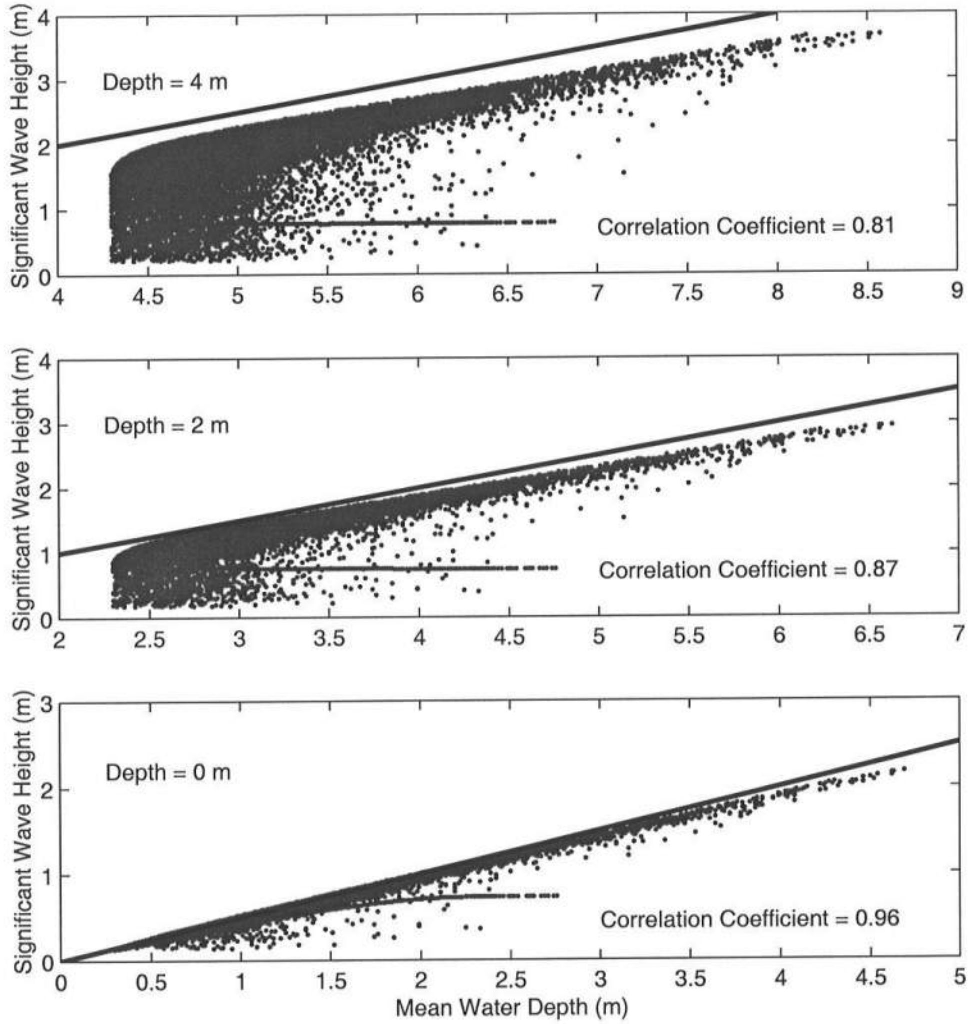

Figure 3.11: Computed Significant Wave Height H_{mo} as a Function of Corresponding Mean Water Depth $\bar{h}$ for All the Time Levels of Ten 500-yr Simulations for the Bottom Slope of 1/800.

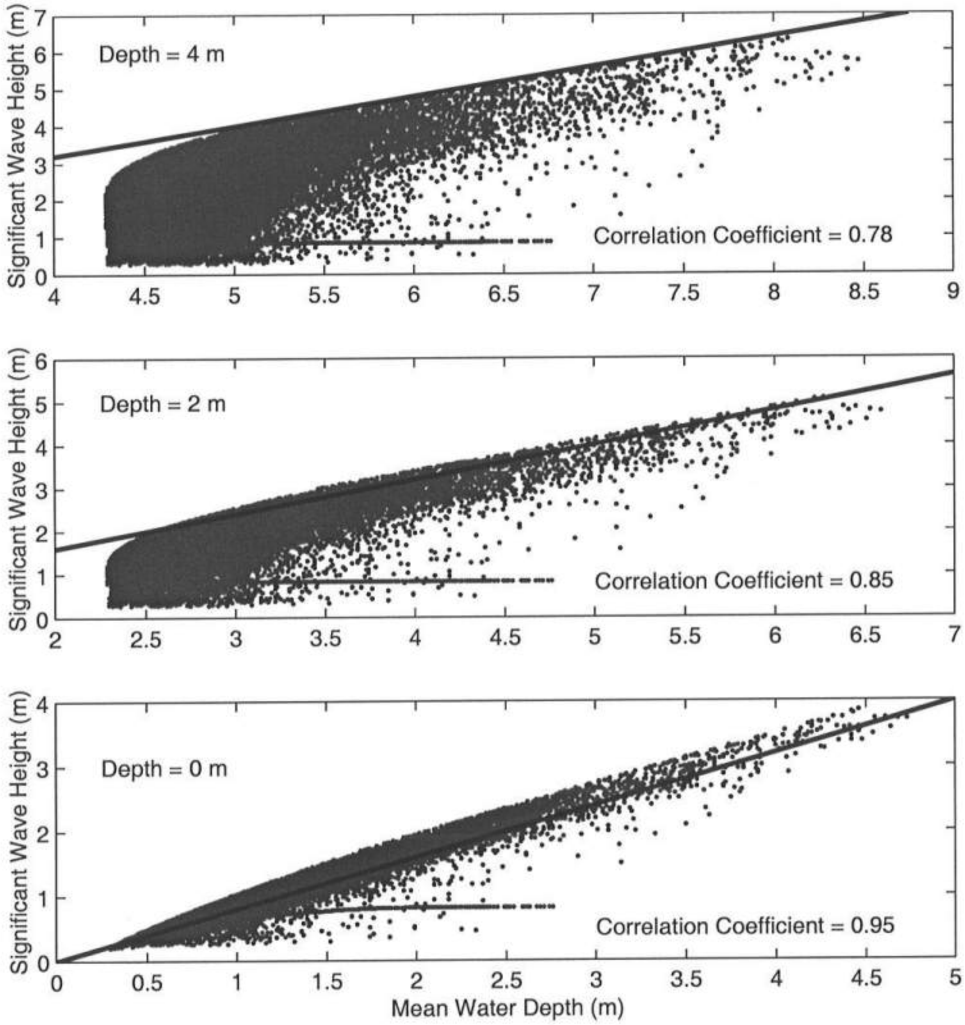

Figure 3.12: Computed Significant Wave Height H_{mo} as a Function of Corresponding Mean Water Depth $\bar{h}$ for All the Time Levels of Ten 500-yr Simulations for the Bottom Slope of 1/40.

The contribution of wave setup to the storm setup $(\bar{h} - d)$ and the mean water depth $\bar{h}$ is not negligible in the water depth $d = 0$–4 m below MSL as shown in Figs. 3.5 and 3.10. Wave setup is normally related to the corresponding offshore significant wave height.

Figs. 3.13 and 3.14 show the computed wave setup ($\bar{h} - d_s$) as a function of the offshore significant wave height H_s for all the time levels of the ten 500-yr simulations for the bottom slopes of 1/800 and 1/40, respectively. The computed wave setup for the offshore significant wave height larger than approximately 10 m may not be reliable because the offshore significant wave height specified as input to CSHORE2 has been limited by $(10 + S)$ m as explained in Section 3.2.

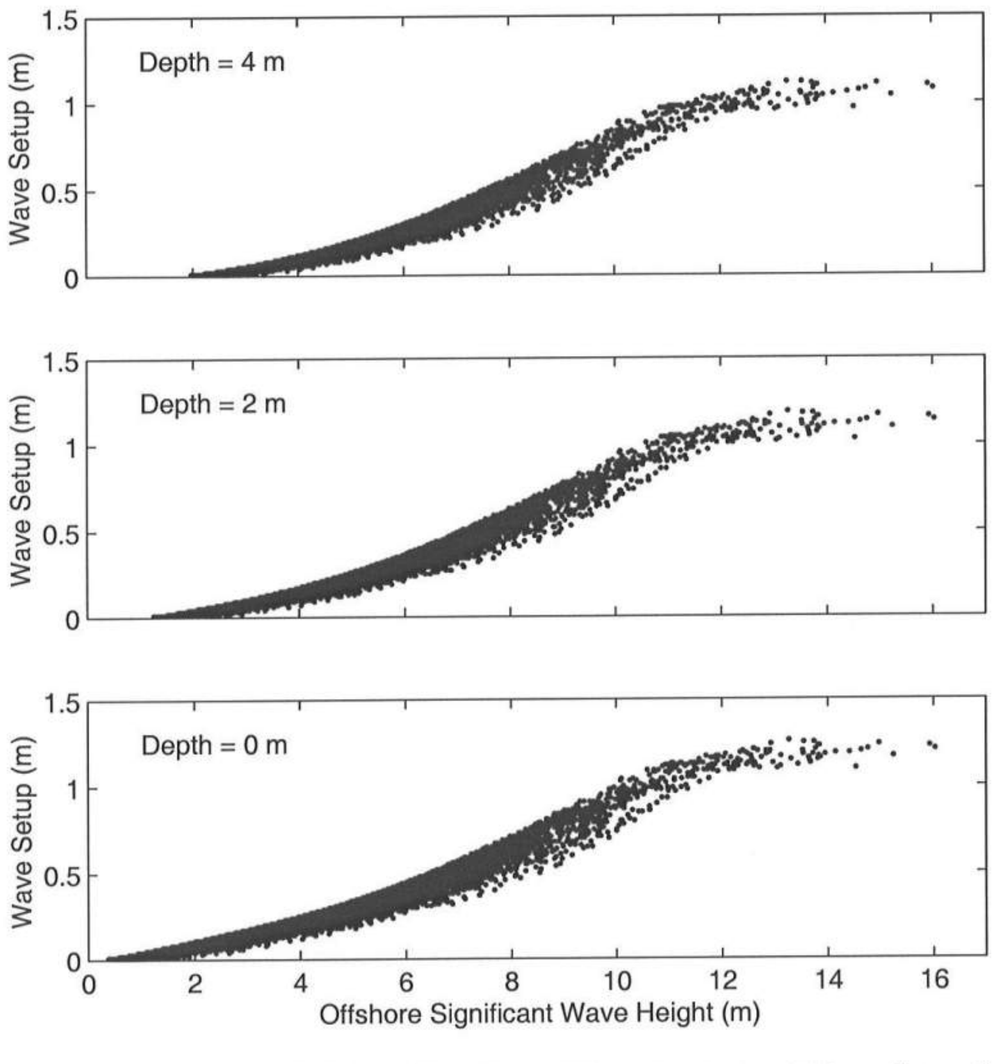

Figure 3.13: Computed Offshore Significant Wave Height and Wave Setup for Ten 500-yr Simulations for the Bottom Slope of 1/800.

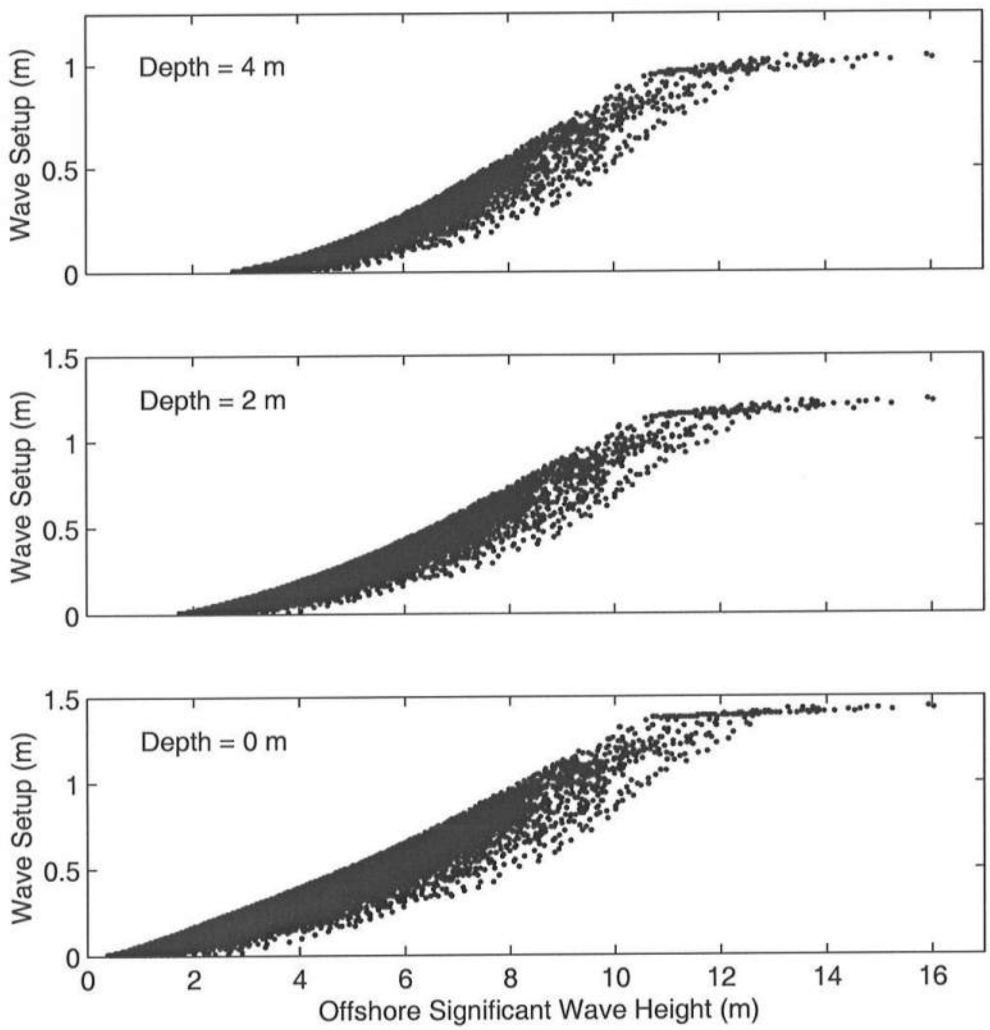

Figure 3.14: Computed Offshore Significant Wave Height and Wave Setup for Ten 500-yr Simulations for the Bottom Slope of 1/40.

Figs. 3.13 and 3.14 indicate that the computed wave setup at d = 0–4 m is of the order of 10 % of the offshore significant wave height in the range of the wave height less than 10 m. The wave setup near the shoreline measured on natural beaches by Raubenheimer *et al.* (2001) indicated that the wave setup was of the order of 20 % of the offshore significant wave height for beach slopes in the range of 1/20 - 1/150 where wave setup increases landward. No field data is available for the very gentle slope 1/800 and very large significant wave heights.

3.5 100-Year Design Waves

The computed results for the ten 500-yr simulations are used to estimate the 100-yr design water depths and significant waves in the following.

Table 3.1 lists the maximum storm tide and the significant wave height and period corresponding to the recurrence interval of 100 years at the most landward node used for the CYCLONE computation. The integer $N500 = 1 - 10$ is used to identify each 500-yr simulation.

Table 3.1: Maximum Offshore Storm Tide, Significant Wave Height and Period for Each of Ten 500-yr Simulations for a Recurrence Interval of 100 Years.

N500	Storm Tide (m)	H_s (m)	T_s (sec)
1	2.85	10.16	12.30
2	3.12	11.42	13.04
3	2.29	9.21	11.71
4	2.49	9.82	12.09
5	2.31	11.53	13.10
6	2.39	9.79	12.07
7	2.64	10.05	12.23
8	2.78	9.82	12.09
9	2.67	9.75	12.04
10	2.57	9.40	11.83
Average	2.61	10.09	12.25

The statistical variabilities of the computed storm tide and the significant wave height H_s have been discussed in relation to Fig. 2.11. The significant wave period T_s has been assumed to be given by equation 2.13 as a function of H_s. Table 3.1 shows that the statistical variability is larger for the storm tide than for the significant wave height and period. The average values in Table 3.1 may be regarded as the 100-yr storm tide near the shoreline of the mean sea level and the 100-yr offshore significant wave height and period.

Tables 3.2 and 3.3 list the maximum storm setup $(\bar{h} - d)$ (sum of storm tide and wave setup) and significant wave height H_{mo} corresponding to the recurrence interval of 100 years computed by CSHORE2 for the bottom slopes of 1/800 and 1/40, respectively. The statistical variabilities of the storm setup and H_{mo} for the ten 500-year simulations indicated by the integer $N500 = 1-10$ are reduced in comparison to those shown in Table 3.1. The storm setup increases with the decrease of the water depth d below the mean sea level because of the landward increase of wave setup. The significant wave heights H_{mo} at $d =$ 4, 2 and 0 m for the bottom slope of 1/40 in Table 3.3 are larger than those for the bottom slope of 1/800 in Table 3.2.

Table 3.2: Maximum Storm Setup and Maximum Significant Wave Height for Each of Ten 500-yr Simulations for a Recurrence Interval of 100 Years on 1/800 Bottom Slope.

	$d = 4$ m		$d = 2$ m		$d = 0$ m	
N500	Storm Setup (m)	H_{mo} (m)	Storm Setup (m)	H_{mo} (m)	Storm Setup (m)	H_{mo} (m)
1	3.17	3.18	3.22	2.39	3.29	1.56
2	3.74	3.24	3.78	2.49	3.83	1.69
3	2.96	3.03	3.02	2.28	3.09	1.48
4	2.66	2.87	2.70	2.08	2.76	1.28
5	2.86	3.07	2.92	2.29	3.00	1.46
6	2.87	3.05	2.93	2.27	3.00	1.45
7	2.83	2.97	2.88	2.19	2.95	1.39
8	2.96	2.99	3.01	2.22	3.07	1.44
9	2.72	3.01	2.75	2.21	2.81	1.38
10	2.94	3.02	3.00	2.26	3.07	1.45
Average	2.97	3.04	3.02	2.27	3.09	1.46

Table 3.3: Maximum Storm Setup and Maximum Significant Wave Height for Each of Ten 500-yr Simulations for a Recurrence Interval of 100 Years on 1/40 Bottom Slope.

	$d = 4$ m		$d = 2$ m		$d = 0$ m	
N500	Storm Setup (m)	H_{mo} (m)	Storm Setup (m)	H_{mo} (m)	Storm Setup (m)	H_{mo} (m)
1	3.09	5.25	3.21	4.24	3.41	3.02
2	3.69	5.52	3.73	4.32	3.81	3.12
3	2.89	4.84	3.02	3.82	3.19	2.69
4	2.61	4.98	2.66	3.76	2.78	2.52
5	2.82	5.33	3.01	4.10	3.22	2.86
6	2.81	5.11	2.97	3.95	3.16	2.76
7	2.76	4.88	2.86	3.71	3.00	2.57
8	2.89	5.02	2.98	3.87	3.12	2.67
9	2.67	4.71	2.82	3.73	3.02	2.64
10	2.87	4.87	3.00	3.82	3.14	2.67
Average	2.91	5.05	3.03	3.93	3.18	2.75

The comparison of the storm setup in Tables 3.2 and 3.3 and the storm tide in Table 3.1 indicates that the contribution of wave setup to the storm setup is approximately 0.5 m which is less than the wave setup of approximately 1 m corresponding to the offshore significant wave height of approximately 10 m in Table 3.1. Consequently, the use of the 100-yr storm tide and the 100-yr significant waves estimated separately will result in the overestimation of wave setup. The reason of this overestimation arises from the fact that the storm tide and offshore significant waves generated by the same hurricane are not well correlated as shown in Fig. 2.14. The average values in Tables 3.2 and 3.3 may be regarded as the 100-yr storm setup and significant wave height at the cross-shore locations of d = 4, 2 and 0 m for the bottom slopes of 1/800 and 1/40, respectively.

Tables 3.4 and 3.5 summarize the 100-yr water depths and significant waves at the locations of $d = 4$, 2 and 0 m for the bottom slopes of 1/800 and 1/40, respectively. The still water depth d_s is the sum of the water depth d below the mean sea level and the 100-yr storm tide. The 100-yr mean water depth $\bar{h}$ is the sum of the depth d and the 100-yr storm setup. The difference between $\bar{h}$ and d_s is caused by wave setup. The design of coastal structures located in very shallow water should use the mean water depth $\bar{h}$ but existing formulas for wave run-up, overtopping and armor stability use the still water depth d_s. These formulas developed using laboratory experiments include wave setup implicitly but wave setup depends on the beach profile in the surf zone (e.g., Raubenheimer *et al.* 2001) which may not be reproduced in typical laboratory experiments.

Table 3.4: Summary of 100-yr Water Depths and Wave Heights on 1/800 Bottom Slope.

Water depth below MSL, $d(m)$	4.0	2.0	0.0
Design water depth, $d_s(m)$	6.61	4.61	2.61
Mean water depth, $\bar{h}(m)$	6.97	5.02	3.09
Significant wave height, $H_{mo}(m)$	3.04	2.27	1.46
Design wave height, $0.78d_s(m)$	5.16	3.60	2.04
Significant wave period, T_s	12.25	12.25	12.25

Table 3.5: Summary of 100-yr Water Depths and Wave Heights on 1/40 Bottom Slope.

Water depth below MSL, $d(m)$	4.0	2.0	0.0
Still water depth, $d_s(m)$	6.61	4.61	2.61
Mean water depth, $\bar{h}(m)$	6.91	5.03	3.18
Significant wave height, $H_{mo}(m)$	5.05	3.93	2.75
Breaking wave height, $0.78d_s(m)$	5.16	3.60	2.04
Significant wave period, T_s	12.25	12.25	12.25

Tables 3.4 and 3.5 also list the 100-yr significant wave height H_{mo} and the corresponding breaking wave height $0.78d_s$ [e.g., Shore Protection Manual (1984)] which is sometimes used as a crude design wave height in the absence of wave data. For the beach with the very gentle slope of 1/800, the breaker height of $0.78d_s$ is much larger than the corresponding significant wave height H_{mo}. The exceedance probability of this breaker height for given H_{mo} could be estimated using the empirical wave height distribution in the surf zone proposed by Battjes and Groenendijk (2000) but this distribution does not account for wave setup. Finally, the 100-yr significant wave period T_s listed in Tables 3.4 and 3.5 correspond to the averaged value of T_s listed in Table 3.1. This does not account for the cross-shore variation of T_s but existing time-averaged models can not predict the cross-shore variation of T_s.

The 100-yr still water depth and significant waves listed in Tables 3.4 and 3.5 are used for a preliminary design of the crest height of a rubble-mound structure located in the water depths of 4, 2 and 0 m below the mean sea level in Chapter 4. The crest height is designed for acceptable overtopping rates during the peak of the 100-yr storm for both gentle and steep slopes using the empirical formula by Van der Meer and Janssen (1995). The structure designed conventionally is then exposed to all the storms in the ten 500-yr simulations in order to predict the maximum overtopping rate, overtopping volume and overtopping duration during each storm. The computed results are analyzed statistically to asses the degree of flooding due to wave overtopping. In Chapter 5, the size of armor stone of a rubble-mound structure is designed against the 100-yr storms listed in Tables 3.4 and 3.5 for the bottom slopes of 1/800 and 1/40, respectively, using the formulas in Shore Protection Manual (1984) and by Van der Meer (1988a, b). The progression of damage to this armor layer due to the sequences of the storms from the ten 500-yr simulations is predicted using the formulas for damage progression and variability proposed by Melby and Kobayashi (1998a, 1998b, 1999, 2000) together with the critical stability number for no damage proposed by Smith *et al.* (1992). This Monte Carlo simulation of the damage progression and variability allows one to estimate the frequency of repair required for the armor layer designed conventionally.

Chapter 4

NUMERICAL MODEL OVERTOP: WAVE OVERTOPPING OF STRUCTURES

Coastal structures are build along shorelines to protect the land area from storm tide and high waves during a severe storm. A major factor in the design of coastal structures is the prevention of intrusion of sea water onto the land by excessive overtopping. The degree of wave overtopping is measured by the amount of overtopped water onto the land area and determines the severity of flooding landward the structure protecting the shoreline. In this chapter, the crest height of a typical rubble mound breakwater is designed for minor overtopping because only minor overtopping is usually allowed unless protective measures are provided against excessive overtopping.

The criteria to determine the amount of overtopping allowed at a specific location need to be set by consideration of not only technical aspects such as the integrity and function of the structure but also many other factors such as the utilization of the land behind the structure. According to Dutch guidelines, for conventional rubble mound structures or Dutch dykes with landward sides covered with clayey soil and relatively good grass, the allowable average overtopping rate is $Q = 0.001\ m^3/s$ per m (Van der Meer and Janssen 1995). On the other hand, for the protection of a relatively densely populated coastal area, the allowable overtopping rate of $Q = 0.01\ m^3/s$ per m has been adopted as a guideline in Japan (Goda 1985).

Consequently, one order of magnitude variability of tolerable overtopping rates is considered in this study and the crest height H_c of the structure above the local bottom is designed for $Q = 0.001$ and 0.01 m^3/s per m.

The toe depth of a hypothetical structure is input to the numerical model OVERTOP and specified to be at the water depths $d = 4$, 2 and 0 m below MSL. The crest height H_c of this hypothetical structure is first designed against the peak of the 100-yr storm listed in Tables 3.4 and 3.5 for the beach slopes of 1/800 and 1/40 in front of the structure, respectively. The crest height designed for the 100-yr storm is then exposed to the sequences of storms generated during the 500-yr simulation. The temporal variations of the water depth and waves during each storm are also accounted for to estimate the volume of water overtopping during the entire storm.

4.1 Computational Procedure

Several empirical formulas are available for the calculation of irregular wave overtopping rates. The numerical model OVERTOP uses the empirical formulas by Van der Meer and Janssen (1995) where two different expressions are developed for breaking and non-breaking waves, respectively.

4.1.1 Computation of Structure Crest Height for Allowable Overtopping Rates

The empirical equations used to calculate the crest height H_c of the structure based on the allowable overtopping rates of $Q = 0.001$ and 0.01 m^3/s per m for the 100-yr storm are summarized in the following.

The dimensionless time-averaged overtopping rate Q_b for breaking waves ($\xi_{op} < 2$) is given by

$$Q_b = \frac{Q}{\sqrt{gH_s^3}}\sqrt{\frac{s_{op}}{\tan\alpha}} \qquad \text{for } \xi_{op} < 2 \tag{4.1}$$

where Q_b = dimensionless overtopping rate for breaking waves and ξ_{op} = surf similarity parameter given by $\xi_{op} = \tan\alpha/\sqrt{s_{op}}$; Q = allowable time-averaged overtopping rate (in m^3/s per m); g = acceleration due to gravity; H_s = significant wave height for the 100-yr storm; s_{op} = wave steepness given by $s_{op} = 2\pi H_s/(gT_p^2)$; T_p = spectral peak period for the 100-yr storm; and α = specified seaward slope of the structure. In the following computation, $\tan\alpha = 0.5$ is assumed.

The dimensionless crest height for breaking waves is expressed as a function of the dimensionless overtopping rate Q_b

$$R_b = -\frac{1}{5.2} ln\left(\frac{Q_b}{0.06}\right) \qquad \text{for } \xi_{op} < 2 \tag{4.2}$$

where R_b = dimensionless crest height for breaking waves and equation 4.2 was developed using data in the range of $0.3 < R_b < 2$.

Finally, the corresponding crest freeboard R_c above the still water level is given by

$$R_c = \frac{R_b H_s (\tan\alpha)\gamma}{\sqrt{s_{op}}} \qquad \text{for } \xi_{op} < 2 \tag{4.3}$$

where γ = combined reduction factor given by $\gamma = \gamma_b\gamma_h\gamma_f\gamma_\beta$; and $\gamma_b, \gamma_h, \gamma_f, \gamma_\beta$ = reduction factors for influence of a berm, shallow foreshore, roughness, and angle of wave attack, respectively. The minimum value of γ using this combination of factors was recommended to be 0.5. These reduction factors are explained in Section 4.1.2.

The expressions for the dimensionless overtopping rate Q_n, the dimensionless crest height R_n, and the crest freeboard R_c above the still water level for non-breaking waves ($\xi_{op} > 2$) are given in the following:

$$Q_n = \frac{Q}{\sqrt{gH_s^3}} \qquad \text{for } \xi_{op} > 2 \tag{4.4}$$

$$R_n = -\frac{1}{2.6} ln \left(\frac{Q_n}{0.2}\right) \qquad \text{for } \xi_{op} > 2 \tag{4.5}$$

$$R_c = R_n H_s \gamma \qquad \text{for } \xi_{op} > 2 \tag{4.6}$$

where equation 4.5 was developed using data in the range of $0.5 < R_n < 4$.

The structure crest height H_c above the local bottom is determined by

$$H_c = R_c + d_s \tag{4.7}$$

where R_c = crest freeboard above the still water level obtained using equations 4.3 and 4.6 for the 100-yr storm; and d_s = still water depth (sum of storm tide and local depth below MSL) for the 100-yr storm.

4.1.2 Influence of Shallow Foreshore and Roughness

Typical rubble mound structures with no berm are considered in this study. Waves are assumed to be normally incident to the structure. Consequently, the reduction factors due to a berm and angle of wave attack are taken as $\gamma_b = 1$ and $\gamma_\beta = 1$.

The influence of a shallow foreshore is accounted for with the following empirical reduction factor γ_h which was developed using data for a foreshore slope of 1/100 and $d_s/H_s \geq 1$ (Van der Meer and Janssen 1995).

$$\gamma_h = 1 \qquad \text{for } d_s/H_s \geq 4 \tag{4.8}$$

$$\gamma_h = 1 - 0.03\left(4 - \frac{d_s}{H_s}\right)^2 \qquad \text{for } d_s/H_s < 4$$

where d_s = still water depth (sum of the storm tide and the local depth below MSL).

Finally, the influence of slope roughness is given by the reduction factor γ_f. For a rubble mound slope consisting of stones placed on a traditional two layer thickness ($H_s/D = 1.5 - 6$ with D = stone diameter), the reduction factor γ_f was recommended as

$$\gamma_f = 0.50 - 0.55 \qquad \text{for } \xi_{op} < 3\text{–}4 \tag{4.9}$$

where ξ_{op} = surf similarity parameter given by $\xi_{op} = \tan\alpha/\sqrt{s_{op}}$. For larger values of the surf similarity parameter ξ_{op}, the reduction factor γ_f was suggested to approach unity. A general expression to estimate the reduction factor γ_f for any value of the surf similarity parameter ξ_{op} is needed. The following relation is tentatively assumed to represent the reduction factor for a rough slope for any value of the surf similarity parameter.

$$\begin{aligned} \gamma_f &= 0.55 && \text{for } \xi_{op} \leq 3.5 \\ \gamma_f &= \frac{\xi_{op}}{\xi_{op} + 2.9} && \text{for } \xi_{op} > 3.5 \end{aligned} \tag{4.10}$$

which is plotted in Fig. 4.1.

4.1.3 Exposure of Structure Crest Height to Sequences of Storms

The structure crest height H_c computed using equations 4.1 to 4.10 for the 100-yr storm is now exposed to the temporal variations of the water depth and significant waves during each storm to estimate the overtopping rate and volume during each of the storms in the 500-yr simulations. The temporal variation of the overtopping rate is computed using the time step of 0.5 hr.

The overtopping rate Q for breaking waves ($\xi_{op} < 2$) is given by the empirical formula of Van der Meer and Janssen (1995) in equation 4.1 which is rewritten as

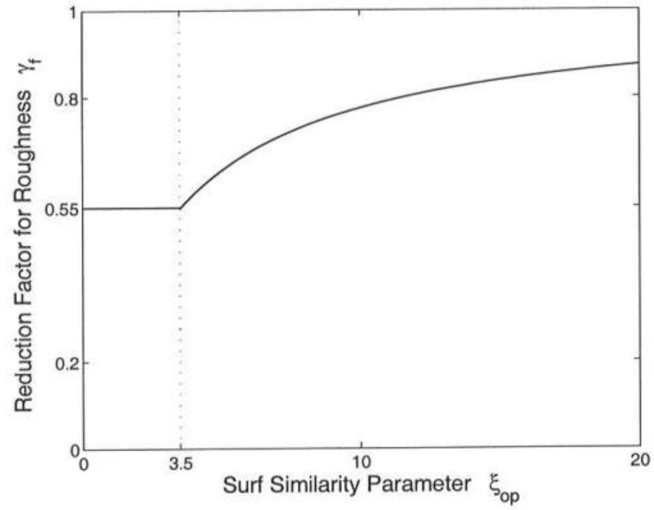

Figure 4.1: Reduction Factor γ_f for the Influence of Roughness as a Function of the Surf Similarity Parameter ξ_{op}.

$$Q = Q_b \sqrt{g H_s^3} \sqrt{\tan \alpha / s_{op}} \qquad \text{for } \xi_{op} < 2 \tag{4.11}$$

where Q_b = dimensionless overtopping rate; g = acceleration due to gravity; H_s = significant wave height; s_{op} = wave steepness given by $s_{op} = 2\pi H_s/(gT_p^2)$; T_p = spectral peak period; and α = seaward slope of the structure taken as $\tan\alpha = 0.5$. The values of H_s and T_p are those at the toe of the structure at a specific time during a storm.

The dimensionless overtopping discharge Q_b in equation 4.11 is estimated using equation 4.2 which is rearranged as

$$Q_b = 0.06\, exp(-5.2 R_b) \qquad \text{for } \xi_{op} < 2 \tag{4.12}$$

where the dimensionless crest height R_b is estimated using equation 4.3 for the known crest height R_c above the still water level at the specific time during a storm.

$$R_b = \frac{R_c\sqrt{s_{op}}}{H_s(\tan\alpha)\gamma} \qquad \text{for } \xi_{op} < 2 \tag{4.13}$$

where γ = combined reduction factor given by $\gamma = \gamma_b\gamma_h\gamma_f\gamma_\beta$.

Rearranging equations 4.4 to 4.6, the overtopping rate Q for the case of non-breaking waves ($\xi_{op} > 2$) is computed as

$$Q = Q_n\sqrt{gH_s^3} \qquad \text{for } \xi_{op} > 2 \tag{4.14}$$

with

$$Q_n = 0.2\,exp(-2.6R_n) \qquad \text{for } \xi_{op} > 2 \tag{4.15}$$

$$R_n = \frac{R_c}{H_s\gamma} \qquad \text{for } \xi_{op} > 2 \tag{4.16}$$

The empirical formula for Q by Van der Meer and Janssen (1995) does not account for wave setup explicitly and it is appropriate only when the mean water depth $\bar{h}$ varying with time during a storm is less than the crest height H_c and the structure crest is not submerged. During the crest submergence ($\bar{h} > H_c$), the overtopping rate Q is estimated as

$$Q = \sqrt{g}(\bar{h} - H_c)^{1.5} \qquad \text{for } \bar{h} > H_c \tag{4.17}$$

where g = gravitational acceleration and the velocity of overflow may be approximated by $[g(\bar{h} - H_c)]^{0.5}$.

The overtopping rate does not reveal the amount of water overtopping the structure crest during a storm. Therefore, the overtopping volume during each storm in the 500-yr simulation is calculated by integrating the computed overtopping rate Q with respect to time where the time step is 0.5 hr.

4.2 Input to OVERTOP

The crest height H_c of the structure with the seaward slope $\tan\alpha = 0.5$ is designed against the 100-yr storm whose conditions have been listed in Tables 3.4 and 3.5 and summarized again in the Table 4.1. It should be noted that the 100-yr values of d_s and H_{mo} are estimated separately and may not occur simultaneously. The value of $\tan\alpha$ is actually specified as input to OVERTOP and can be changed easily.

Table 4.1: Water Depths and Wave Heights for 100-yr Recurrence Interval where Offshore $H_s = 10.1$ m and $T_s = 12.3$ s.

Water depth below MSL, $d(m)$	4.0	2.0	0.0
Still water depth, $d_s(m)$	6.61	4.61	2.61
Mean depth $\bar{h}(m)$ for 1/800 slope	6.97	5.02	3.09
Mean depth $\bar{h}(m)$ for 1/40 slope	6.91	5.03	3.18
Breaker height, $0.78d_s(m)$	5.16	3.60	2.04
Wave height $H_{mo}(m)$ for 1/800 slope	3.04	2.27	1.46
Wave height $H_{mo}(m)$ for 1/40 slope	5.05	3.93	2.75

The required crest height against the 100-yr storm conditions is obtained for the allowable overtopping rate $Q = 0.001$ and 0.01 m^3/s per meter which may be regarded as a typical range for minor overtopping [e.g., Goda (1985); Van der Meer and Janssen (1995)] as mentioned at the beginning of this chapter. The significant wave height H_s is approximated as $H_s \simeq H_{mo}$. The spectral peak period T_p is approximately given by $T_p \simeq 1.05T_s$ with $T_s =$ significant wave period. The still water depth d_s at the toe of the structure is the sum of the water depth below the mean sea level and storm tide where storm tide varies during a storm.

The seaward slope of the structure is taken as $\tan\alpha = 1/2$ as stated earlier for clarity. Incident waves are assumed to be normal to the structure. The armor layer consists of stone placed in a traditional two-layer thickness. Fig. 4.2 shows the typical structure considered in this study. Damage to the armor layer examined in the next chapter may increase wave overtopping but is not considered here.

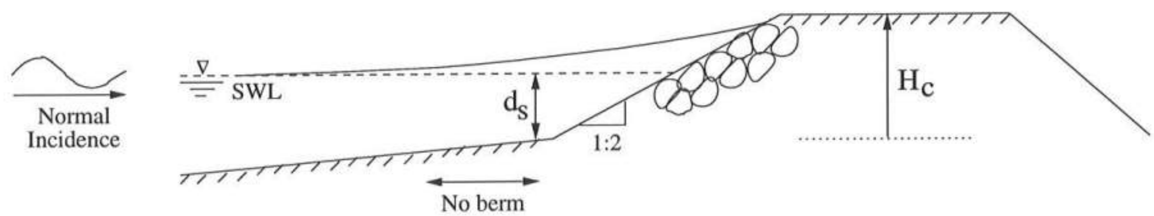

Figure 4.2: Cross-section of a Typical Rubble Mound Structure.

4.3 Computed Results for Bottom Slope of 1/800

The output from OVERTOP for the ten 500-yr simulations is discussed for the bottom slope of 1/800 in the following.

First, the high and low structure crest heights H_c above the local bottom required against the 100-yr storm are computed for the allowable overtopping rates of $Q = 0.001$ and 0.01 m^3/s per meter at the locations of depth $d = 4$, 2 and 0 m on the bottom slope of 1/800 . The corresponding crest heights are listed in Table 4.2 where the computed high and low crest heights for the bottom slope of 1/40 are also listed for comparison. Table 4.2 indicates that the increase of H_c by about one meter reduces Q by one order of magnitude. The required crest height on the steep slope is larger than that on the gentle slope because of the larger significant wave height at the toe of the structure as listed in Table 4.1.

Table 4.2: High and Low Structure Crest Heights H_c above Local Bottom Designed for Wave Overtopping Rates $Q = 0.001$ and $0.01 (m^3/s/m)$, Respectively, for 100-yr Storm.

Bottom Slope	Crest Height	$d = 4m$	$d = 2m$	$d = 0m$
$\frac{1}{800}$	High $H_c(m)$	11.8	8.4	4.9
	Low $H_c(m)$	10.3	7.3	4.2
$\frac{1}{40}$	High $H_c(m)$	15.2	11.0	6.8
	Low $H_c(m)$	13.0	9.3	5.6

Second, the temporal variation of Q during each of the 3,539 storms in the ten 500-yr simulations is computed for each of the high and low crest heights listed in Table 4.2. The maximum value of Q for each storm is also found and stored. The overtopping water volume during each storm is calculated by integrating Q with respect to time where the time step is 0.5 hr. The maximum overtopping rate and the overtopping water volume are ranked separately for the storms in each 500-yr simulation and expressed as a function of the recurrence interval. The corresponding computed results are presented in the following.

4.3.1 Maximum Overtopping Rate

The maximum overtopping rate Q for each storm in each 500-yr simulation is presented as a function of the recurrence interval for the beach slope of 1/800. Figs. 4.3 to 4.5 show the maximum overtopping rate for the high structure crest at the locations of depth d = 4, 2 and 0 m, respectively. Figs. 4.6 to 4.8 show the maximum overtopping rate for the low structure crest at the locations of depth d = 4, 2 and 0 m, respectively.

The high and low crest heights have been designed for the overtopping rate of 10^{-3} and 10^{-2} m^3/s per meter, respectively, for the 100-yr storm. It can be observed in these figures that the variability of the maximum overtopping rate among the ten 500-yr simulations exceeds one order of magnitude for botrh high and low crest heights for the recurrence interval larger than about 50 years. This implies that the allowable overtopping rate for the design of the crest height needs to account for the statistical variability of one order of magnitude. This uncertainty explains why the existing guideline for the allowable overtopping rate is crude [e.g., Goda (1985); Van der Meer and Janssen (1995)].

The maximum overtopping rate averaged for the ten 500-yr simulations for the high and low crest heights at the locations of d = 4, 2 and 0 m on the 1/800 slope is shown in Fig. 4.9. The difference of one order of magnitude between the high and low crest heights remains approximately the same for the recurrence interval exceeding about 50 years. It should be noted that for the low crest height at d = 0 m, the structure crest is submerged for about 2 hr during two storms. The maximum overtopping rate for the recurrence interval larger than 200 years may not be accurate in this case due to the use of the simple formula for the overflow rate expressed in equation 4.17.

Comparison of the lines for d = 4, 2 and 0 m in Fig. 4.10 for the high and low crest heights indicates that the maximum overtopping rate increases more rapidly with the increase of the recurrence interval as the water depth d is decreased. On the other hand, the difference between the designed high and low crest heights in Table 4.2 decreases with the decrease of d. Consequently, the use of a slightly higher crest height at d = 0 m is effective in reducing the wave overtopping rate during severe storms.

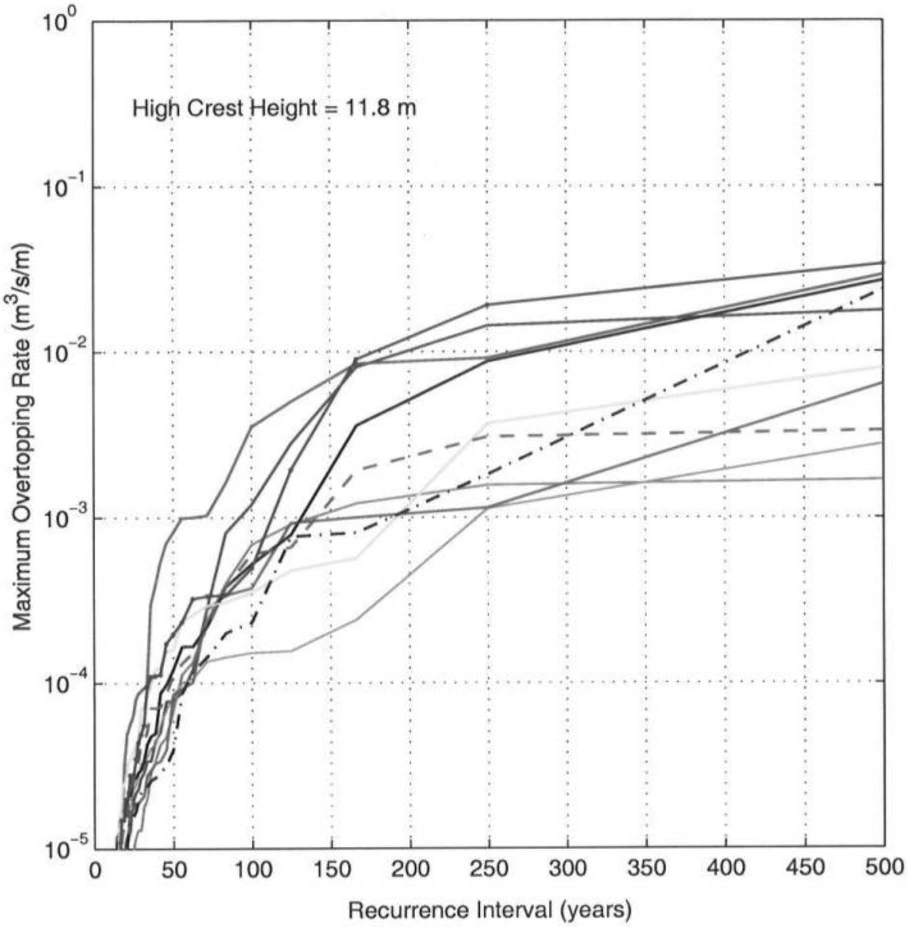

Figure 4.3: Maximum Overtopping Rate for High Structure Crest on 1/800 Slope at Location of $d = 4$ m as a Function of the Recurrence Interval for Ten 500-yr Simulations: (Blue —) N500=1; (Red —) N500=2; (Green —) N500=3; (Cyan —) N500=4; (Magenta —) N500=5; (Black —) N500=6; (Yellow —) N500=7; (Blue -•-) N500=8; (Black -.-) N500=9; (Red - -) N500=10.

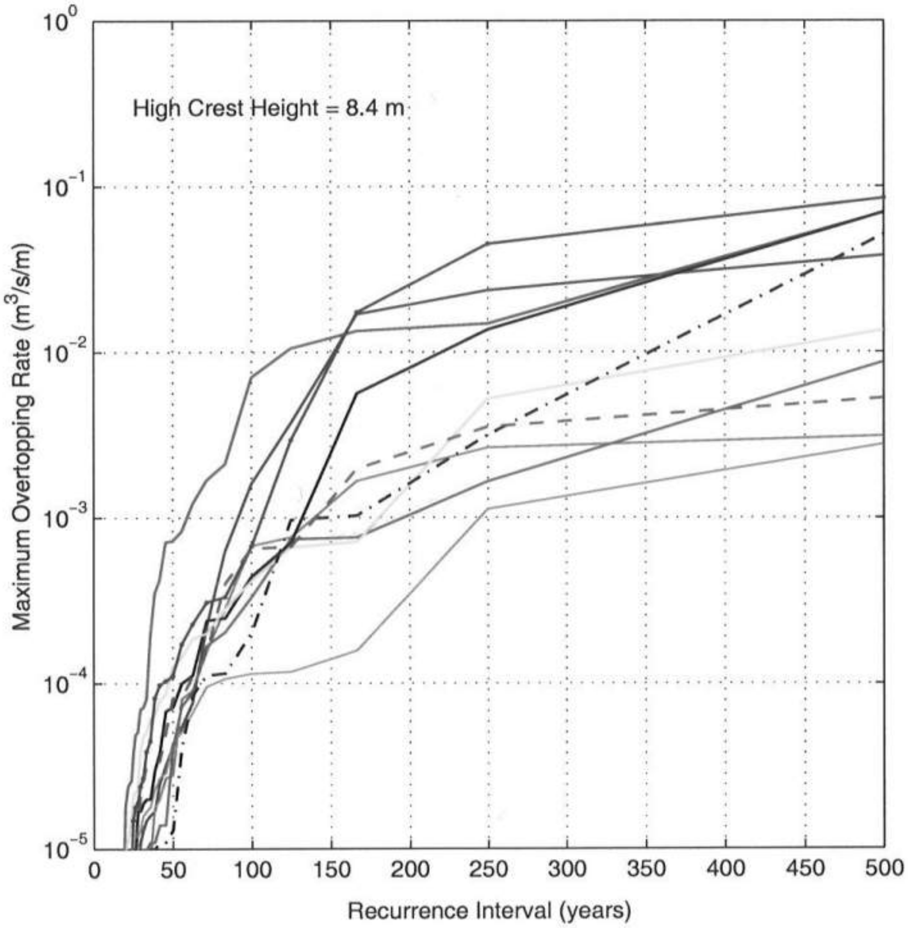

Figure 4.4: Maximum Overtopping Rate for High Structure Crest on 1/800 Slope at Location of $d = 2$ m as a Function of the Recurrence Interval for Ten 500-yr Simulations: (Blue —) N500=1; (Red —) N500=2; (Green —) N500=3; (Cyan —) N500=4; (Magenta —) N500=5; (Black —) N500=6; (Yellow —) N500=7; (Blue -•-) N500=8; (Black -.-) N500=9; (Red - -) N500=10.

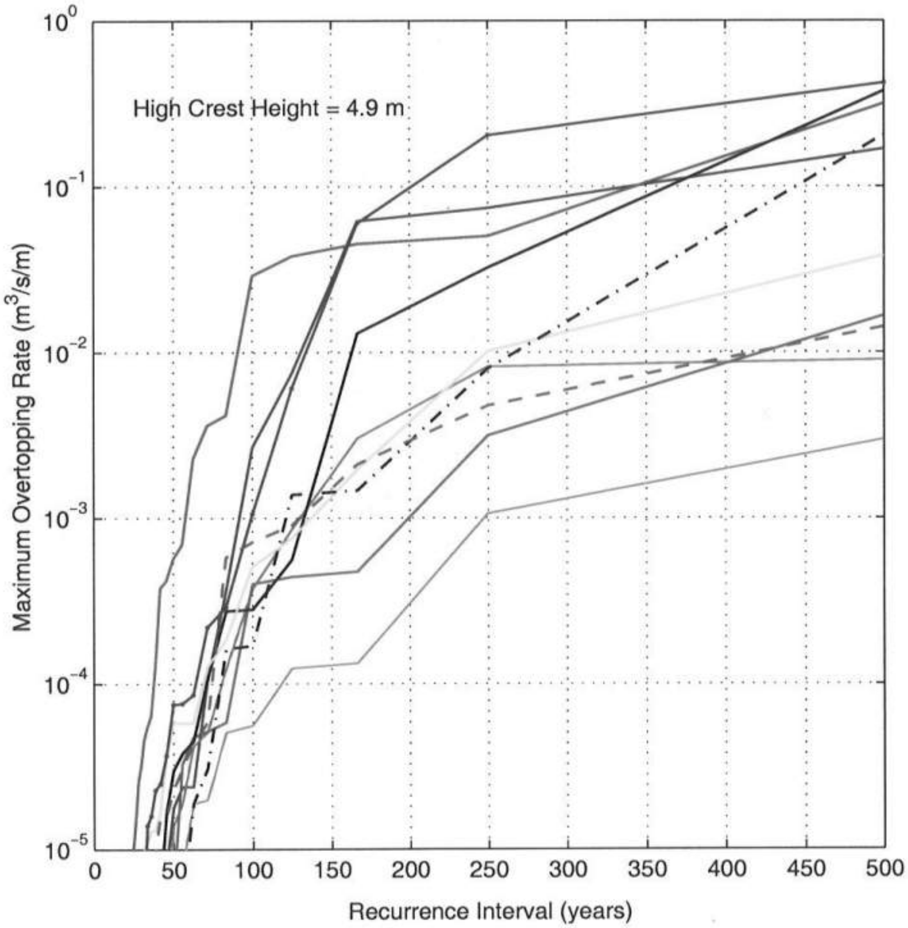

Figure 4.5: Maximum Overtopping Rate for High Structure Crest on 1/800 Slope at Location of $d = 0$ m as a Function of the Recurrence Interval for Ten 500-yr Simulations: (Blue —) N500=1; (Red —) N500=2; (Green —) N500=3; (Cyan —) N500=4; (Magenta —) N500=5; (Black —) N500=6; (Yellow —) N500=7; (Blue -•-) N500=8; (Black -.-) N500=9; (Red - -) N500=10.

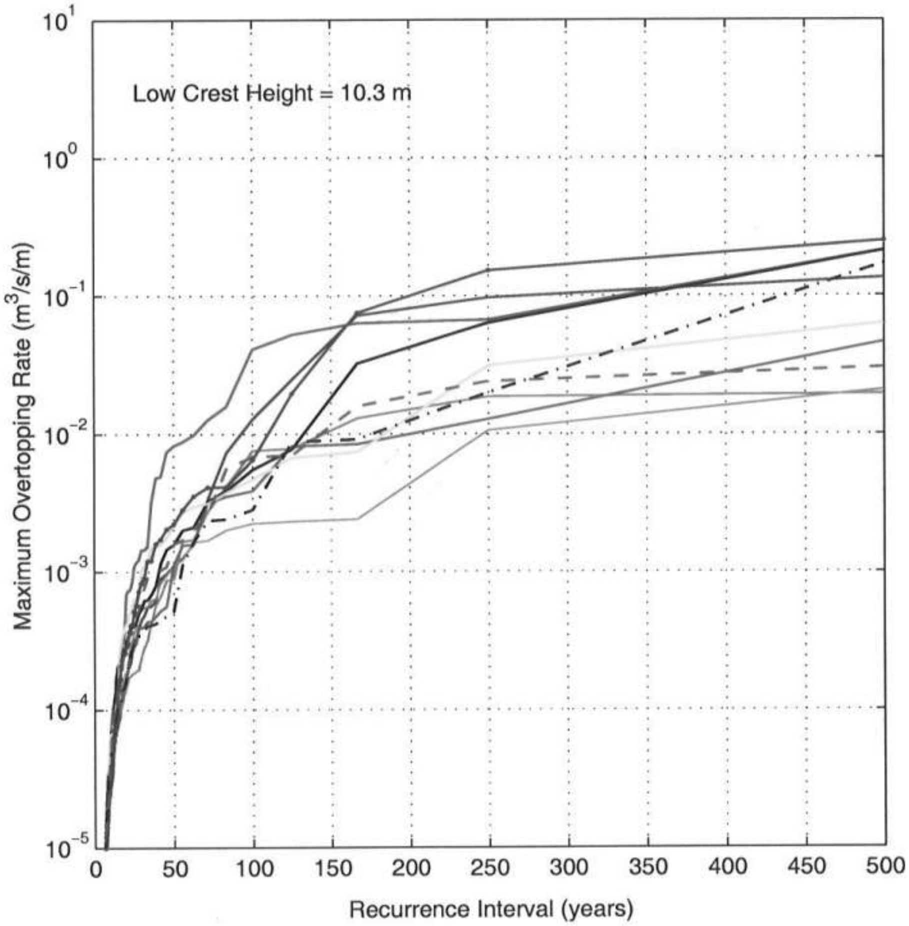

Figure 4.6: Maximum Overtopping Rate for Low Structure Crest on 1/800 Slope at Location of $d = 4$ m as a Function of the Recurrence Interval for Ten 500-yr Simulations: (Blue —) N500=1; (Red —) N500=2; (Green —) N500=3; (Cyan —) N500=4; (Magenta —) N500=5; (Black —) N500=6; (Yellow —) N500=7; (Blue -•-) N500=8; (Black -.-) N500=9; (Red - -) N500=10.

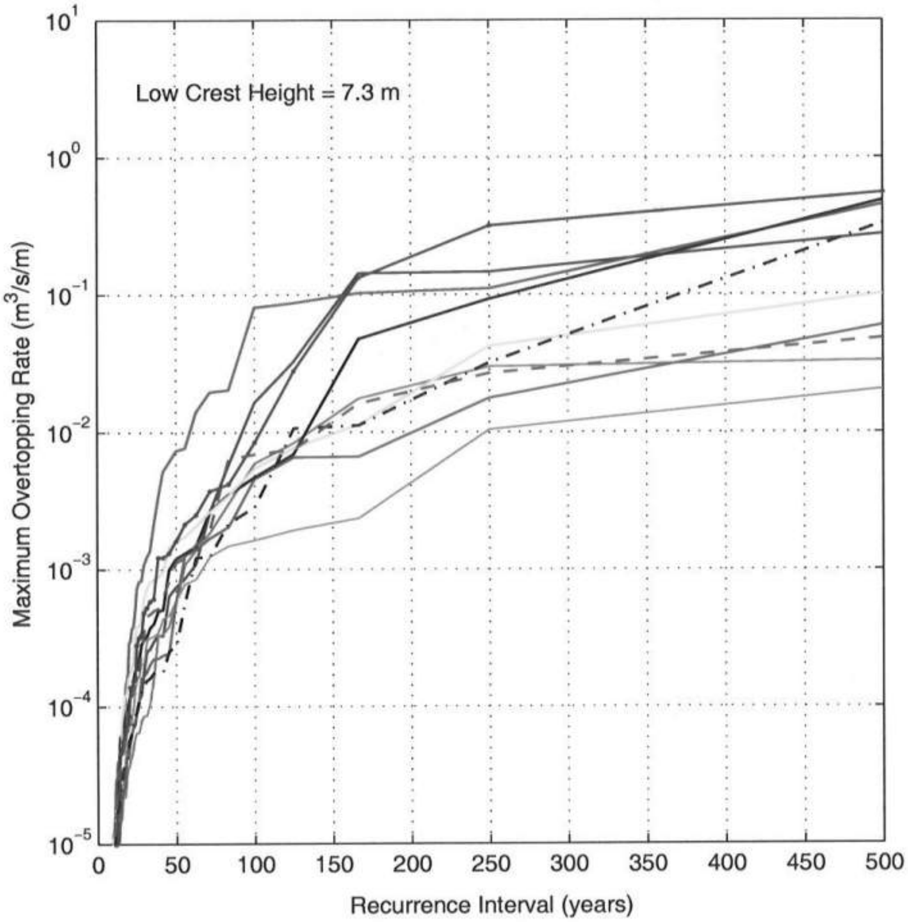

Figure 4.7: Maximum Overtopping Rate for Low Structure Crest on 1/800 Slope at Location of $d = 2$ m as a Function of the Recurrence Interval for Ten 500-yr Simulations: (Blue —) N500=1; (Red —) N500=2; (Green —) N500=3; (Cyan —) N500=4; (Magenta —) N500=5; (Black —) N500=6; (Yellow —) N500=7; (Blue -•-) N500=8; (Black -.-) N500=9; (Red - -) N500=10.

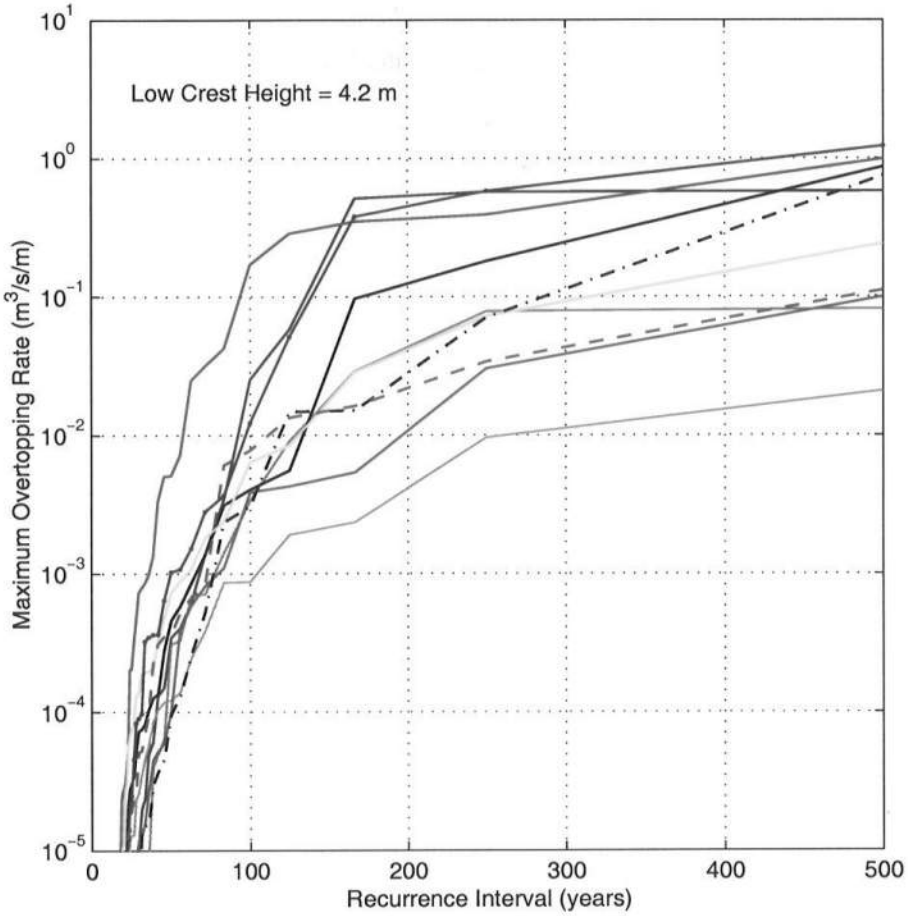

Figure 4.8: Maximum Overtopping Rate for Low Structure Crest on 1/800 Slope at Location of $d = 0$ m as a Function of the Recurrence Interval for Ten 500-yr Simulations: (Blue —) N500=1; (Red —) N500=2; (Green —) N500=3; (Cyan —) N500=4; (Magenta —) N500=5; (Black —) N500=6; (Yellow —) N500=7; (Blue -•-) N500=8; (Black -.-) N500=9; (Red - -) N500=10.

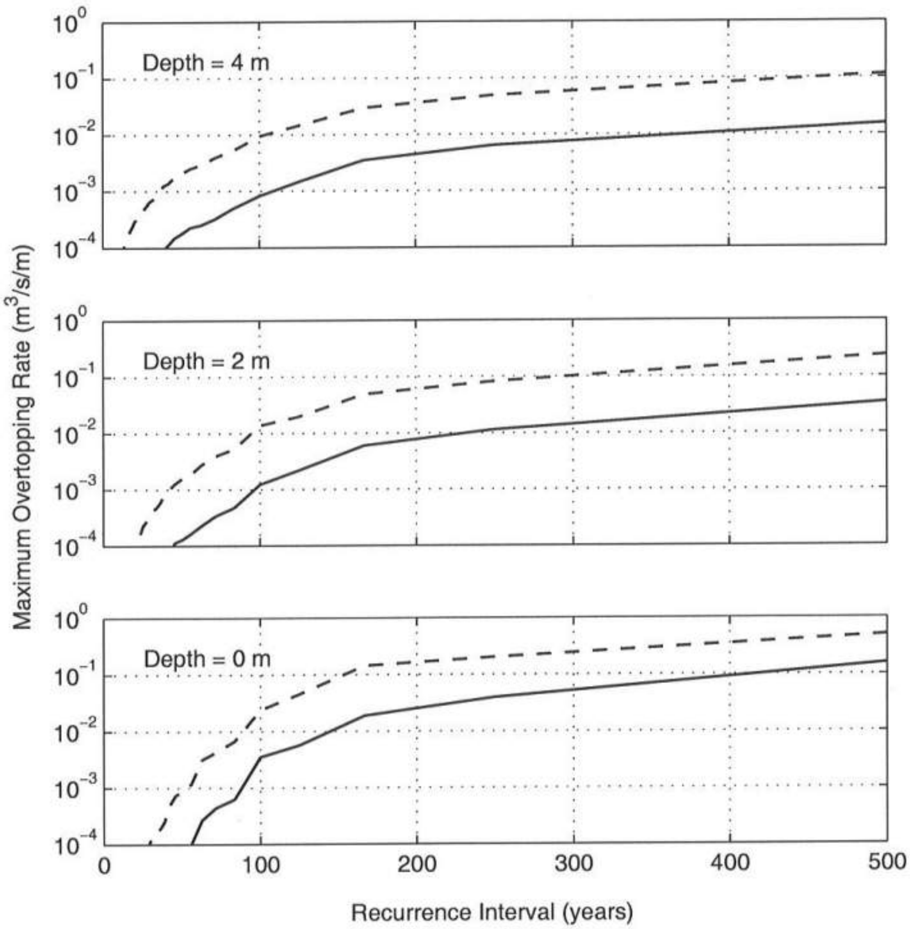

Figure 4.9: Maximum Overtopping Rate Averaged for Ten 500-yr Simulations for High and Low Structure Crests on 1/800 Slope at Locations of $d = 4$, 2 and 0 m: (—) High Crest; (- -) Low Crest.

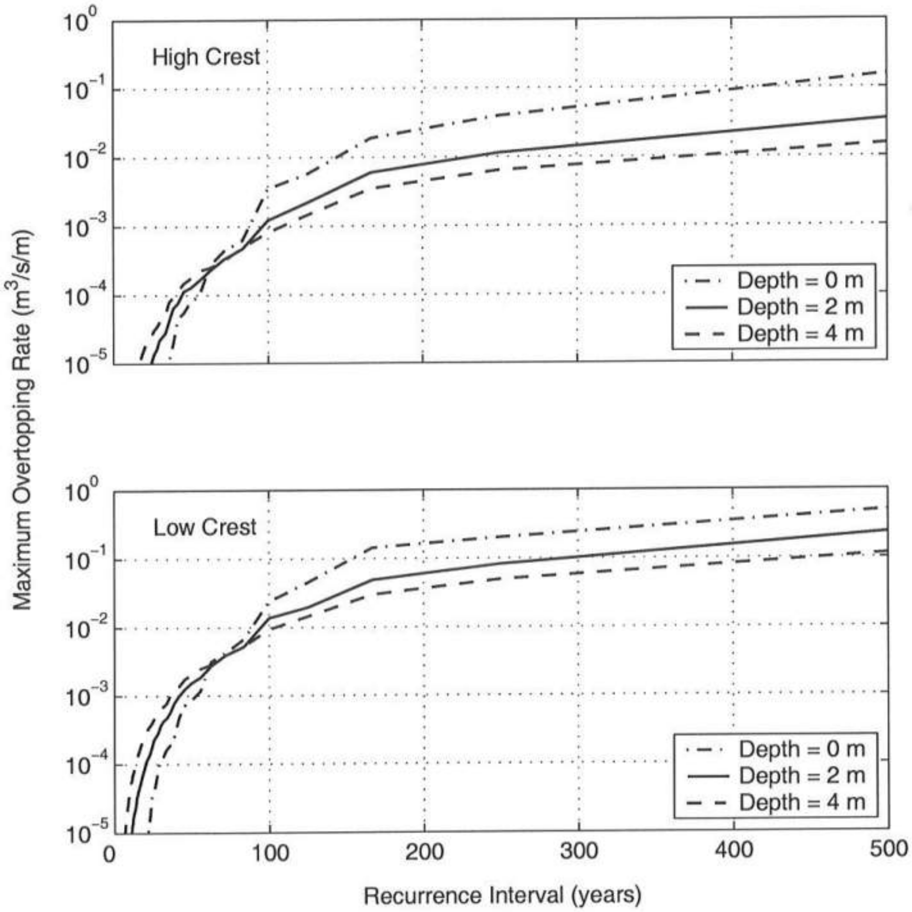

Figure 4.10: Comparison of Maximum Overtopping Rate Averaged for Ten 500-yr Simulations on 1/800 Slope at Locations of $d = 4$, 2 and 0 m for High and Low Structure Crests.

4.3.2 Overtopping Water Volume During Storm and Equivalent Overtopping Duration

The overtopping water volume during each storm in each 500-yr simulation is plotted in Figs. 4.11 to 4.16 for the high and low crest heights, respectively, at locations of depth $d = 4$, 2 and 0 m in the same manner as in Figs. 4.3 to 4.8 for the maximum overtopping rate.

The overtopping water volume averaged for the ten 500-yr simulations is shown in Fig. 4.17 in the same manner as in Fig 4.9 for the corresponding overtopping rate. The overtopping volume for the 100-yr recurrence interval is of the order of 10 and 10^2 m^3 per meter for the high and low crest heights, respectively. Comparison of the lines for $d = 4$, 2 and 0 m for the high and low crest heights is shown in Fig. 4.18 to clarify the effect of the water depth d below the mean sea level.

The overtoping volume during an entire storm is useful in estimating the extent of a flooded area. However, the prediction of the overtopping volume requires the time series of storm tide and wave characteristics during an entire storm which are not available for most engineering projects. As a result, it is convenient to define the equivalent overtopping duration as the ratio between the overtopping volume and the maximum overtopping rate during the peak of a storm. The equivalent overtopping duration calculated using the computed results in Figs. 4.9 and 4.17 is plotted in Fig. 4.19 and approximately 3 hr for both high and low crests at $d = 4$, 2 and 0 m. This is due to the fact that the variations with respect to the recurrence interval in Figs. 4.9 and 4.17 are very similar for the corresponding lines for the same crest height and the depth d. In short, the overtopping water volume may be estimated by multiplying the maximum overtopping rate by approximately 3 hr.

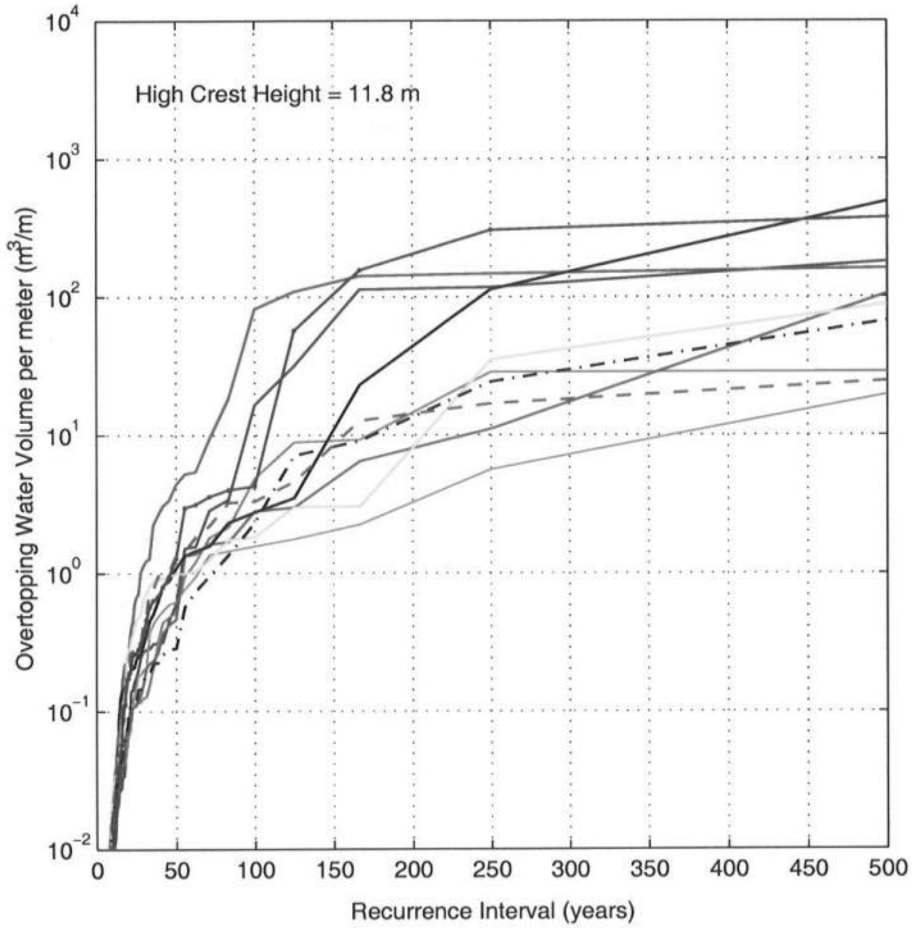

Figure 4.11: Overtopping Water Volume During Storm for High Structure Crest on 1/800 Slope at Location of $d = 4$ m as a Function of the Recurrence Interval for Ten 500-yr Simulations: (Blue —) N500=1; (Red —) N500=2; (Green —) N500=3; (Cyan —) N500=4; (Magenta —) N500=5; (Black —) N500=6; (Yellow —) N500=7; (Blue -•-) N500=8; (Black -.-) N500=9; (Red - -) N500=10.

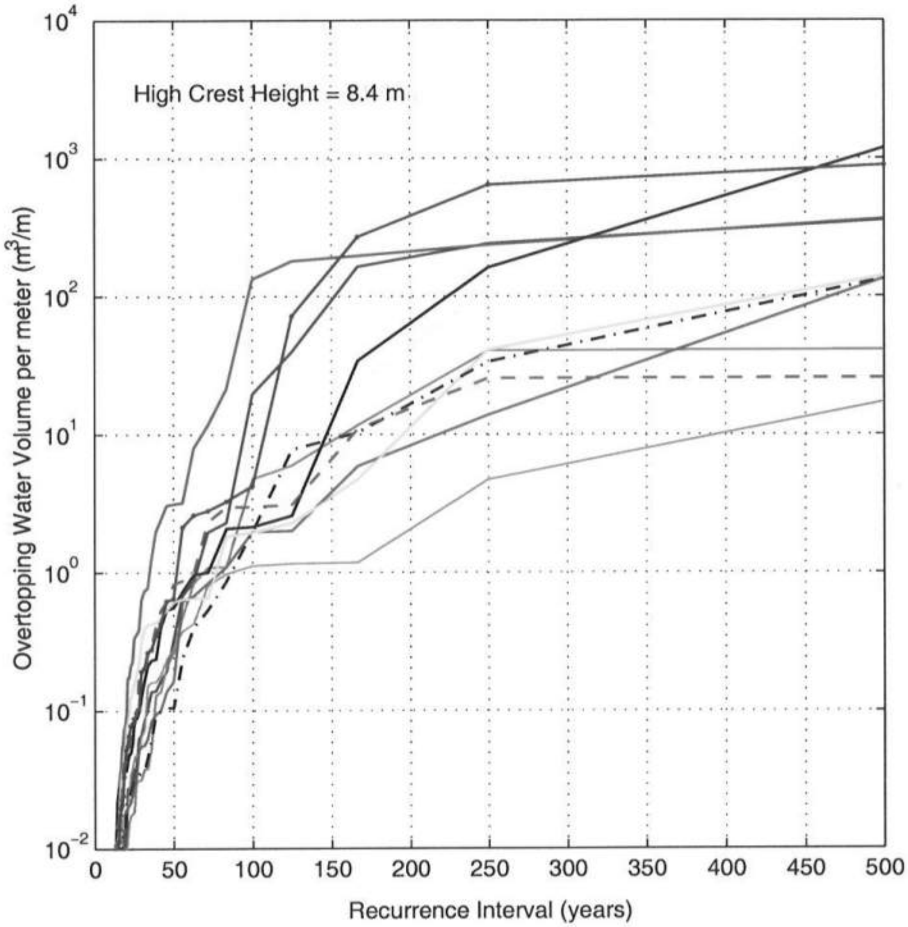

Figure 4.12: Overtopping Water Volume During Storm for High Structure Crest on 1/800 Slope at Location of $d = 2$ m as a Function of the Recurrence Interval for Ten 500-yr Simulations: (Blue —) N500=1; (Red —) N500=2; (Green —) N500=3; (Cyan —) N500=4; (Magenta —) N500=5; (Black —) N500=6; (Yellow —) N500=7; (Blue -•-) N500=8; (Black -.-) N500=9; (Red - -) N500=10.

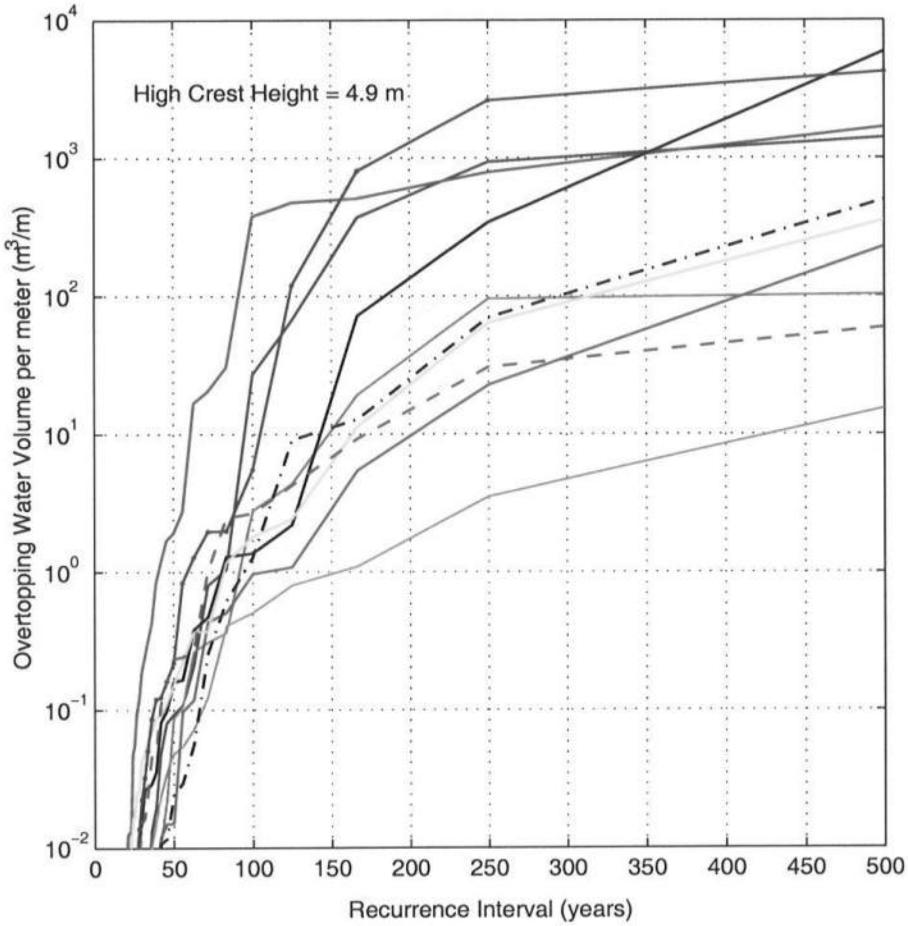

Figure 4.13: Overtopping Water Volume During Storm for High Structure Crest on 1/800 Slope at Location of $d = 0$ m as a Function of the Recurrence Interval for Ten 500-yr Simulations: (Blue —) N500=1; (Red —) N500=2; (Green —) N500=3; (Cyan —) N500=4; (Magenta —) N500=5; (Black —) N500=6; (Yellow —) N500=7; (Blue -•-) N500=8; (Black -.-) N500=9; (Red - -) N500=10.

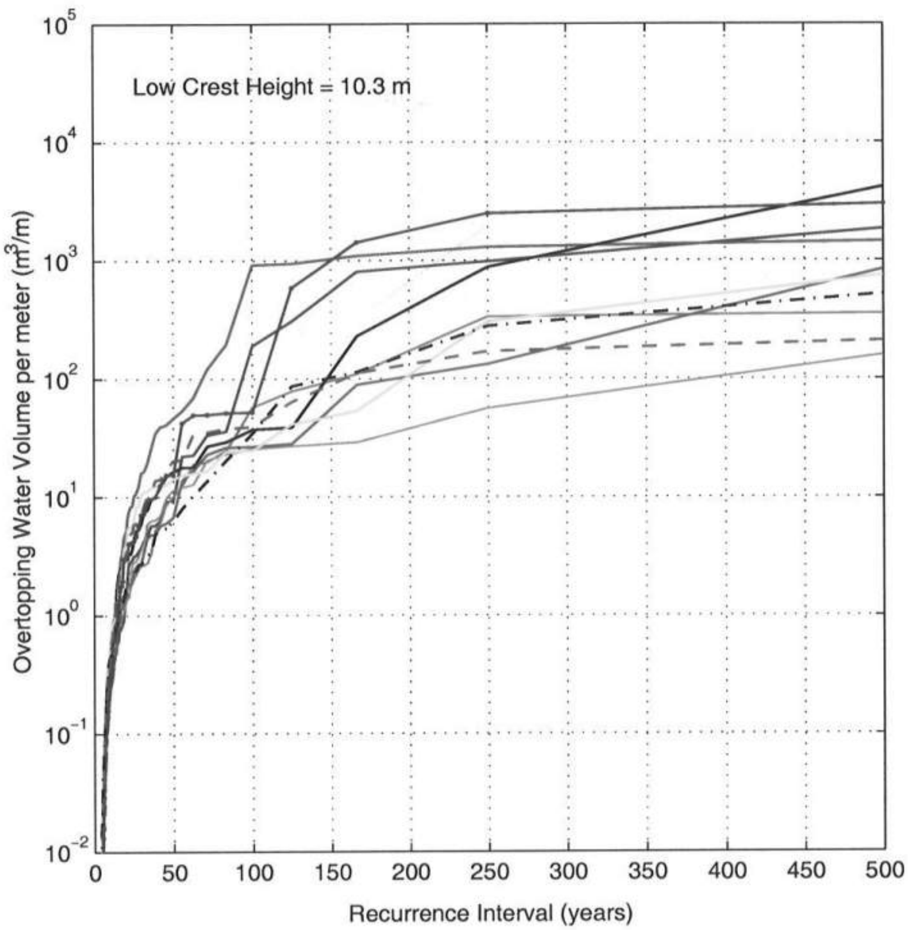

Figure 4.14: Overtopping Water Volume During Storm for Low Structure Crest on 1/800 Slope at Location of $d = 4$ m as a Function of the Recurrence Interval for Ten 500-yr Simulations: (Blue —) N500=1; (Red —) N500=2; (Green —) N500=3; (Cyan —) N500=4; (Magenta —) N500=5; (Black —) N500=6; (Yellow —) N500=7; (Blue -•-) N500=8; (Black -.-) N500=9; (Red - -) N500=10.

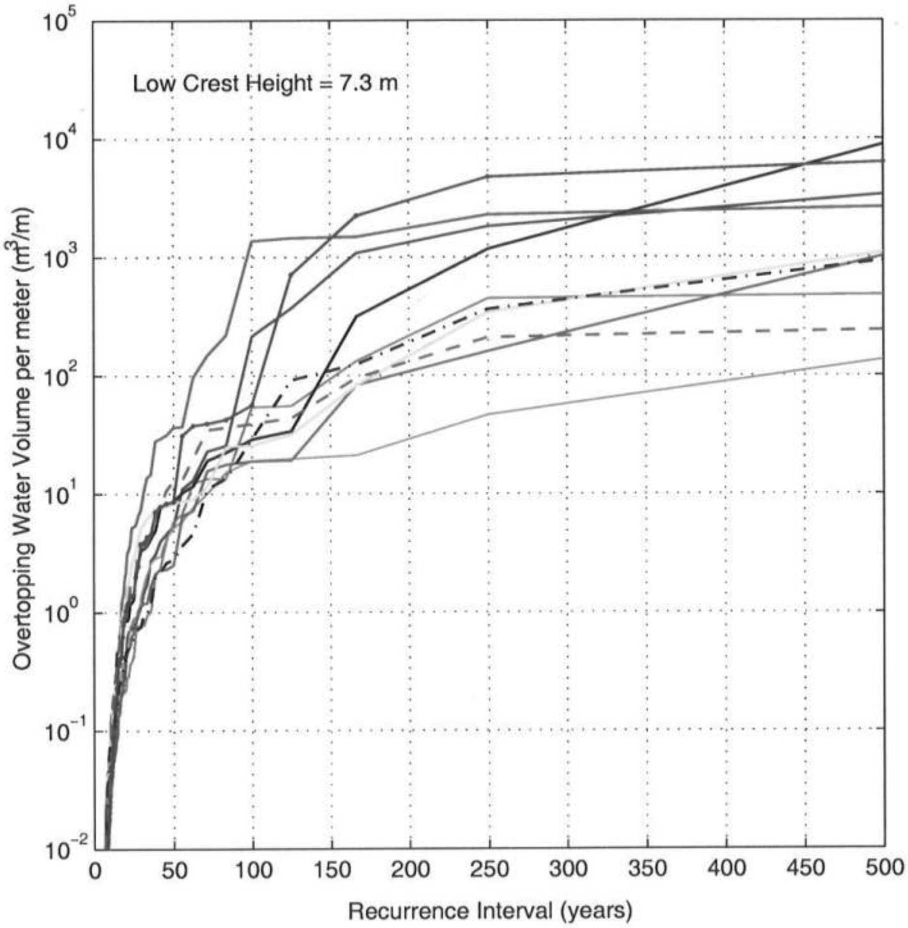

Figure 4.15: Overtopping Water Volume During Storm for Low Structure Crest on 1/800 Slope at Location of $d = 2$ m as a Function of the Recurrence Interval for Ten 500-yr Simulations: (Blue —) N500=1; (Red —) N500=2; (Green —) N500=3; (Cyan —) N500=4; (Magenta —) N500=5; (Black —) N500=6; (Yellow —) N500=7; (Blue -•-) N500=8; (Black -.-) N500=9; (Red - -) N500=10.

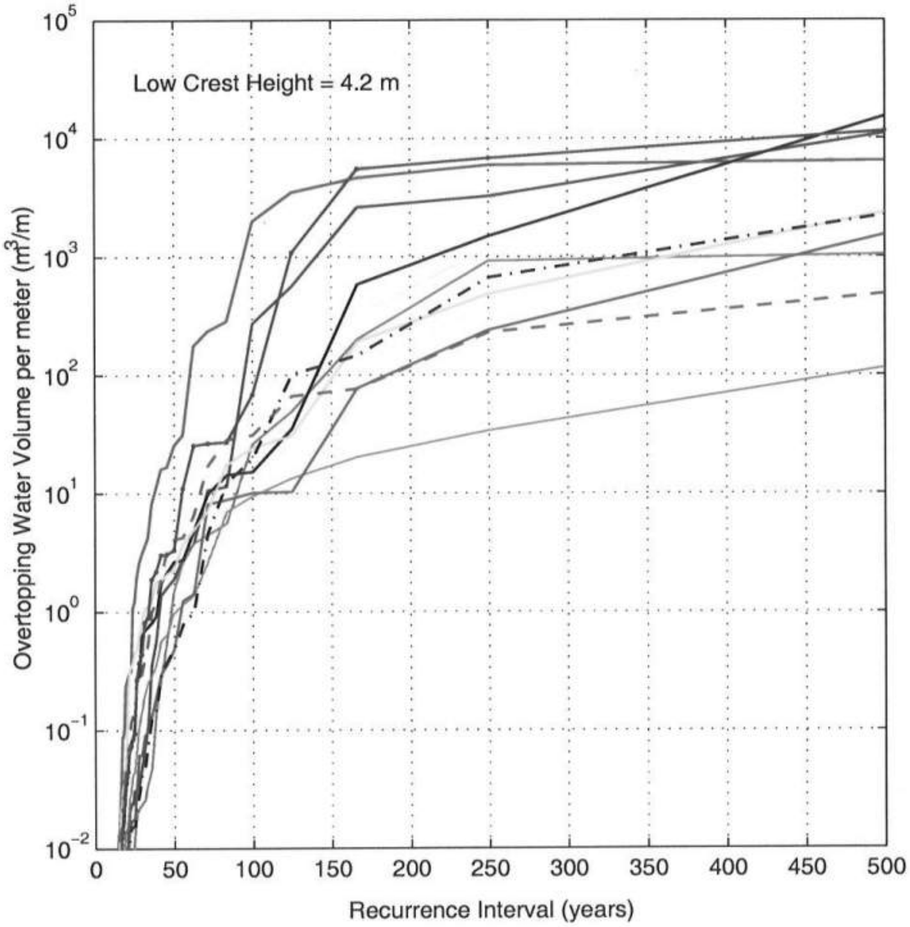

Figure 4.16: Overtopping Water Volume During Storm for Low Structure Crest on 1/800 Slope at Location of $d = 0$ m as a Function of the Recurrence Interval for Ten 500-yr Simulations: (Blue —) N500=1; (Red —) N500=2; (Green —) N500=3; (Cyan —) N500=4; (Magenta —) N500=5; (Black —) N500=6; (Yellow —) N500=7; (Blue -•-) N500=8; (Black -.-) N500=9; (Red - -) N500=10.

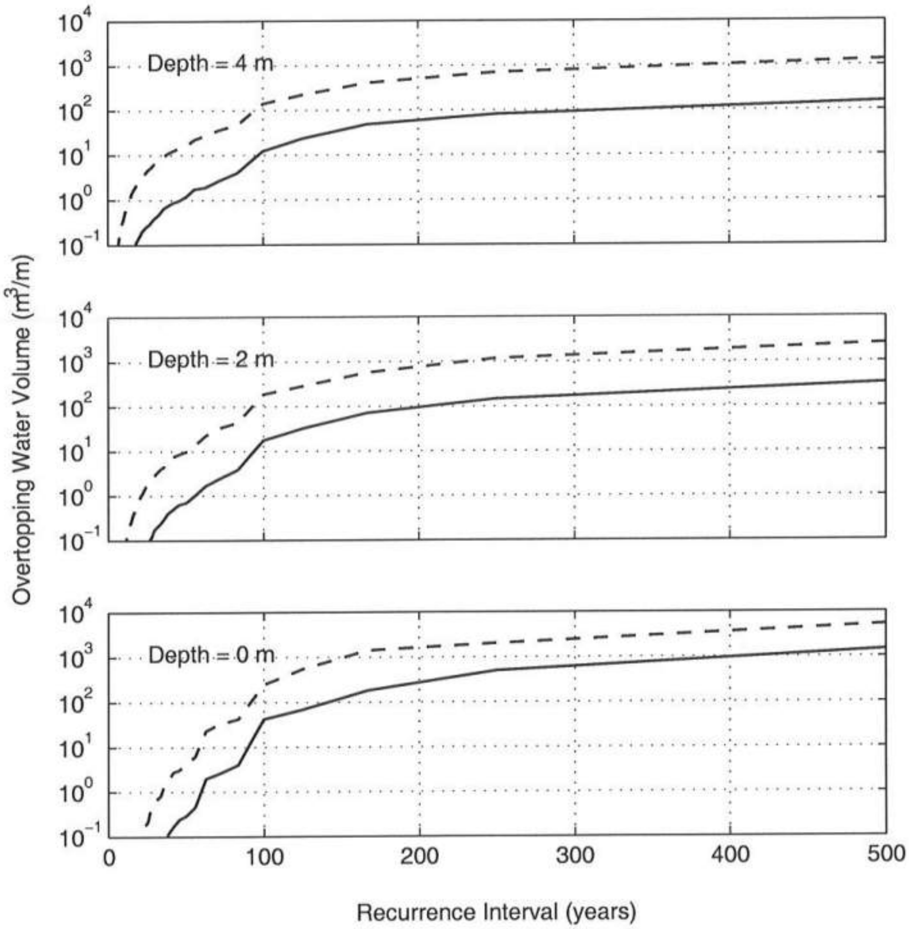

Figure 4.17: Overtopping Water Volume Averaged for Ten 500-yr Simulations for High and Low Structure Crests on 1/800 Slope at Locations of $d =$ 4, 2 and 0 m: (—) High Crest; (- -) Low Crest.

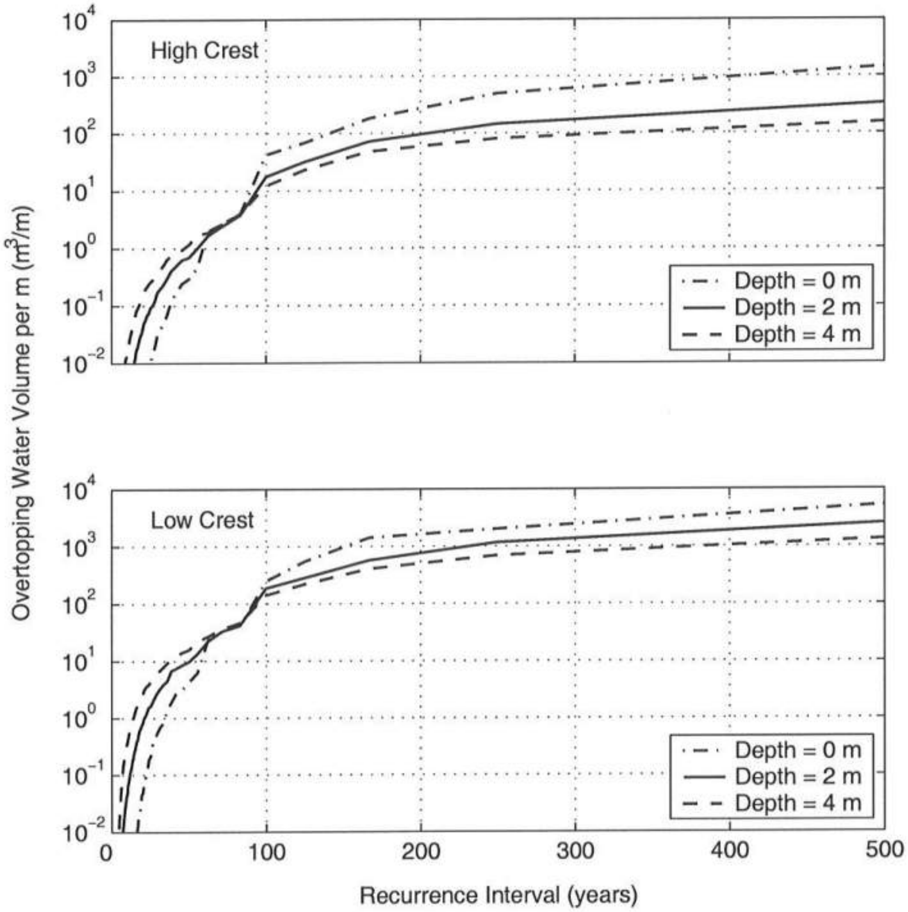

Figure 4.18: Comparison of Overtopping Water Volume Averaged for Ten 500-yr Simulations on 1/800 Slope at Locations of $d = 4$, 2 and 0 m for High and Low Structure Crests.

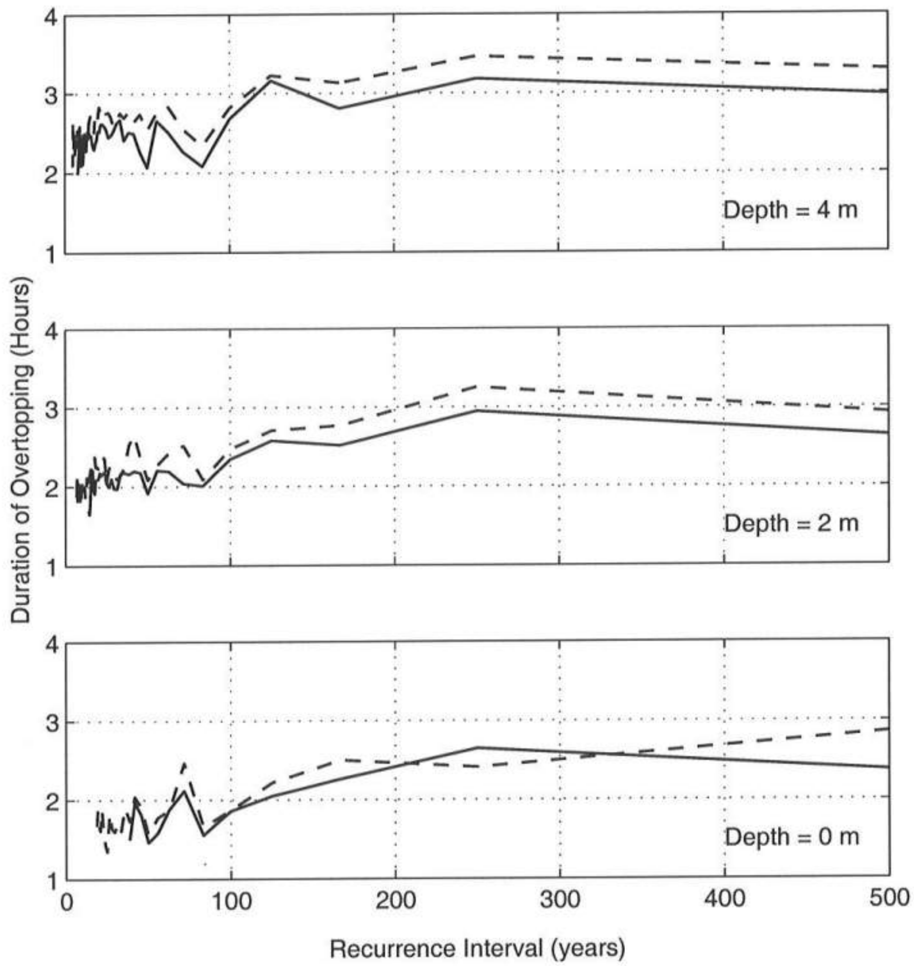

Figure 4.19: Equivalent Overtopping Duration Based on Overtopping Volume and Maximum Overtopping Rate Averaged for Ten 500-yr Simulations for High and Low Structure Crests on 1/800 Slope at Locations of $d =$ 4, 2 and 0 m: (—) High Crest; (- -) Low Crest.

4.4 Computed Results for Bottom Slope of 1/40

The computed results for the case of the 1/40 bottom slope are similar with respect to the maximum overtopping rate, overtopping water volume and equivalent overtopping duration.

4.4.1 Maximum Overtopping Rate

Figs. 4.20 to 4.25 show the maximum overtopping rate for the ten 500-yr simulations for the high and low crest heights in the same manner as Figs. 4.3 to 4.8 for the 1/800 slope. The statistical variability between the ten simulations in Figs. 4.20 to 4.25 remains to be one order of magnitude for both high and low crest heights for the case of the steep slope of 1/40.

The maximum overtopping rate averaged for the ten 500-yr simulations for the 1/40 slope is presented in Fig. 4.26 which is fairly similar to Fig. 4.9 for the 1/800 slope. The difference of one order of magnitude between the high and low crest heights in Fig. 4.26 remains approximately the same for the recurrence interval exceeding about 50 years. The comparison of Figs. 4.9 and 4.26 indicates that the maximum overtopping rate on the 1/800 bottom slope is slightly larger for the recurrence interval exceeding 100 years for both high and low crests at the three locations of depth $d = 4$, 2 and 0 m.

Finally, Fig. 4.27 shows the comparison of the lines for $d = 4$, 2 and 0 m for the high and low crest heights. As is the case with Fig. 4.10 for the slope of 1/800 the maximum overtopping rate increases more rapidly with the increase of the recurrence interval as the water depth d is decreased.

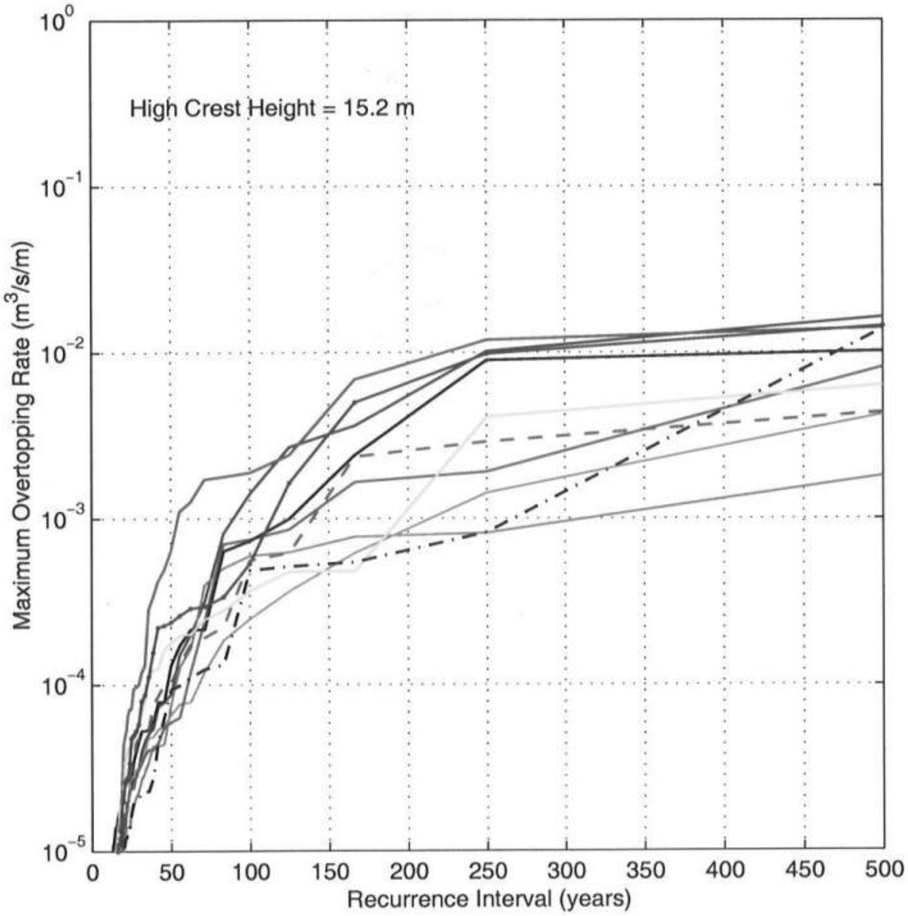

Figure 4.20: Maximum Overtopping Rate for High Structure Crest on 1/40 Slope at Location of d = 4 m as a Function of the Recurrence Interval for Ten 500-yr Simulations: (Blue —) N500=1; (Red —) N500=2; (Green —) N500=3; (Cyan —) N500=4; (Magenta —) N500=5; (Black —) N500=6; (Yellow —) N500=7; (Blue -•-) N500=8; (Black -.-) N500=9; (Red - -) N500=10.

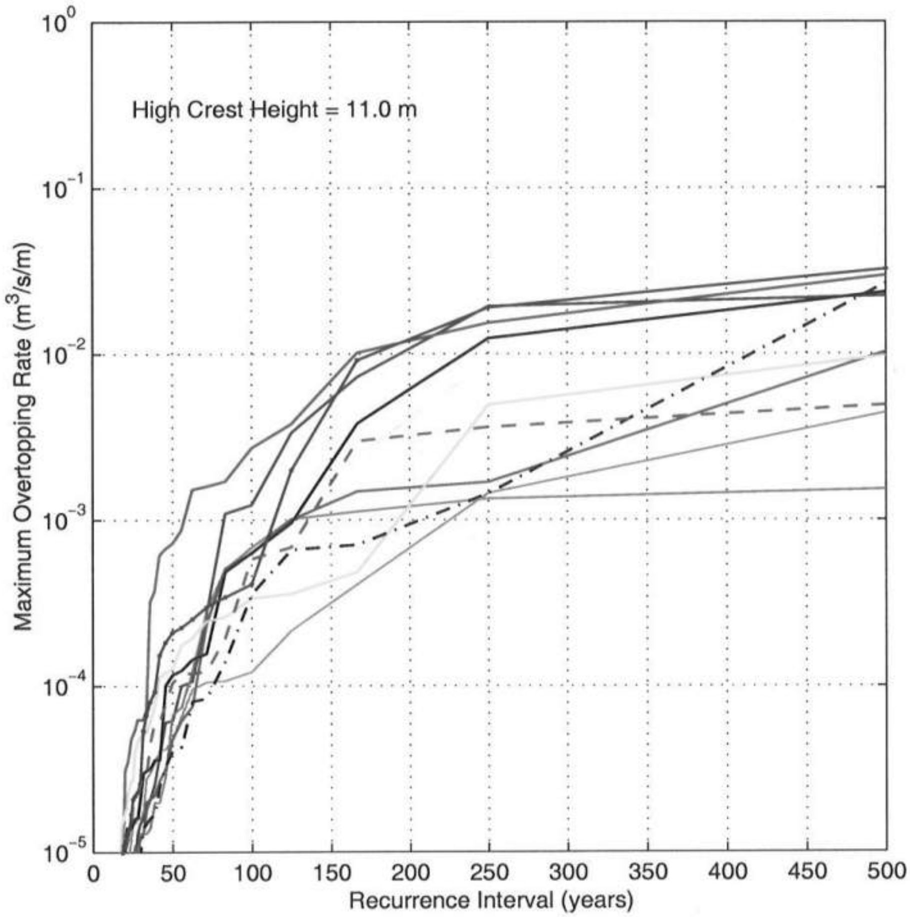

Figure 4.21: Maximum Overtopping Rate for High Structure Crest on 1/40 Slope at Location of d = 2 m as a Function of the Recurrence Interval for Ten 500-yr Simulations: (Blue —) N500=1; (Red —) N500=2; (Green —) N500=3; (Cyan —) N500=4; (Magenta —) N500=5; (Black —) N500=6; (Yellow —) N500=7; (Blue -•-) N500=8; (Black -.-) N500=9; (Red - -) N500=10.

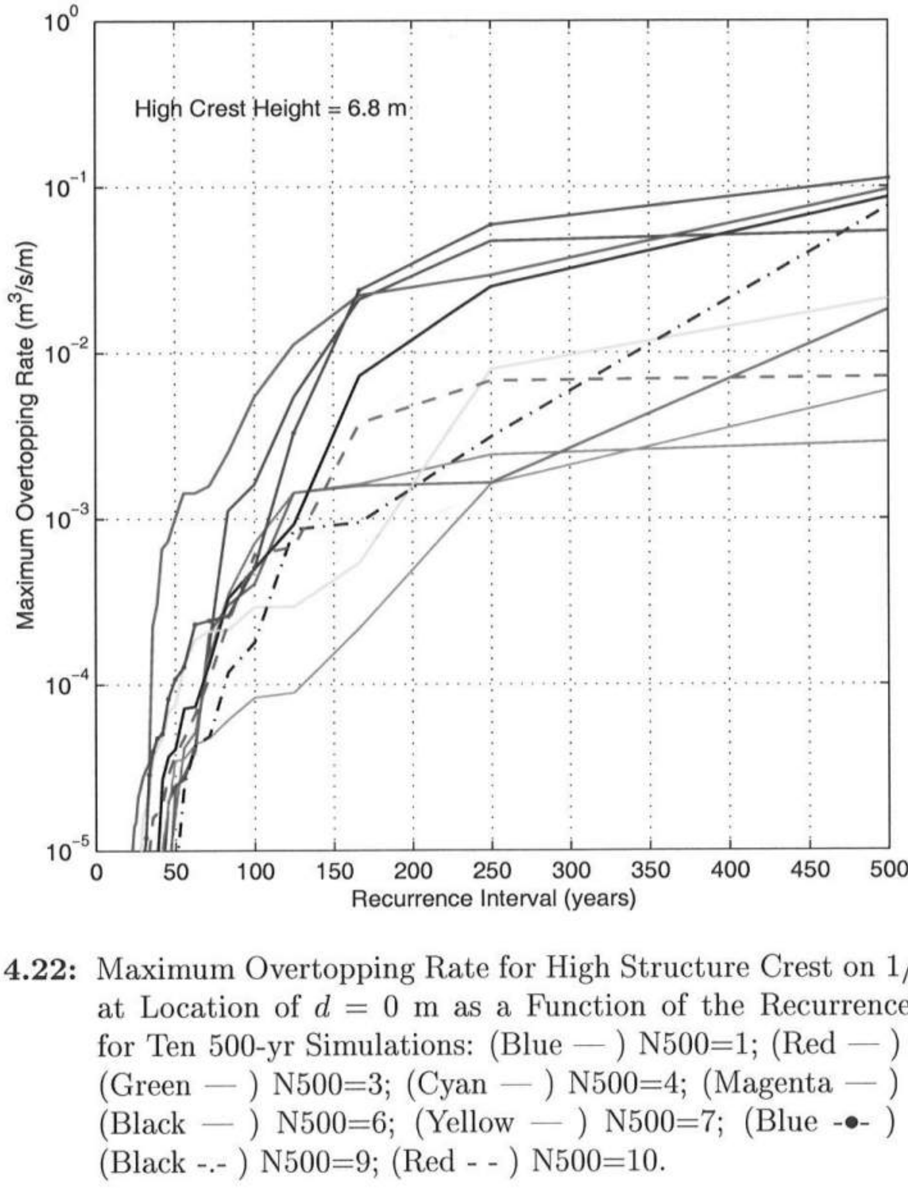

Figure 4.22: Maximum Overtopping Rate for High Structure Crest on 1/40 Slope at Location of $d = 0$ m as a Function of the Recurrence Interval for Ten 500-yr Simulations: (Blue —) N500=1; (Red —) N500=2; (Green —) N500=3; (Cyan —) N500=4; (Magenta —) N500=5; (Black —) N500=6; (Yellow —) N500=7; (Blue -•-) N500=8; (Black -.-) N500=9; (Red - -) N500=10.

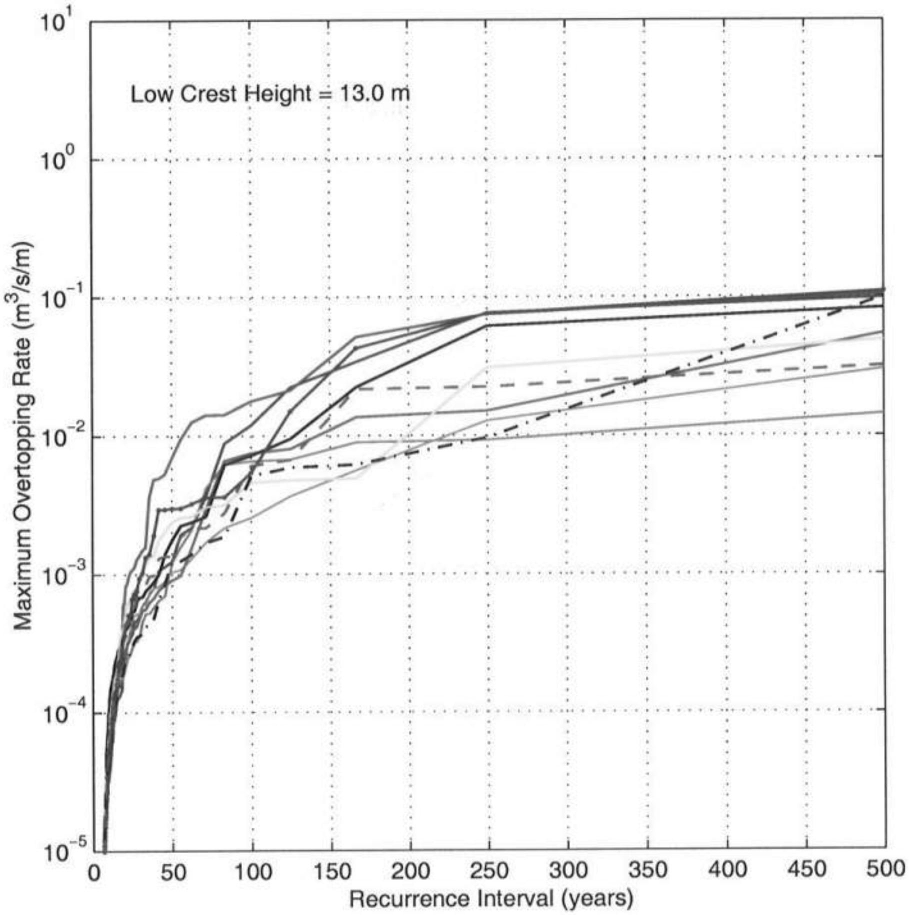

Figure 4.23: Maximum Overtopping Rate for Low Structure Crest on 1/40 Slope at Location of d = 4 m as a Function of the Recurrence Interval for Ten 500-yr Simulations: (Blue —) N500=1; (Red —) N500=2; (Green —) N500=3; (Cyan —) N500=4; (Magenta —) N500=5; (Black —) N500=6; (Yellow —) N500=7; (Blue -•-) N500=8; (Black -.-) N500=9; (Red - -) N500=10.

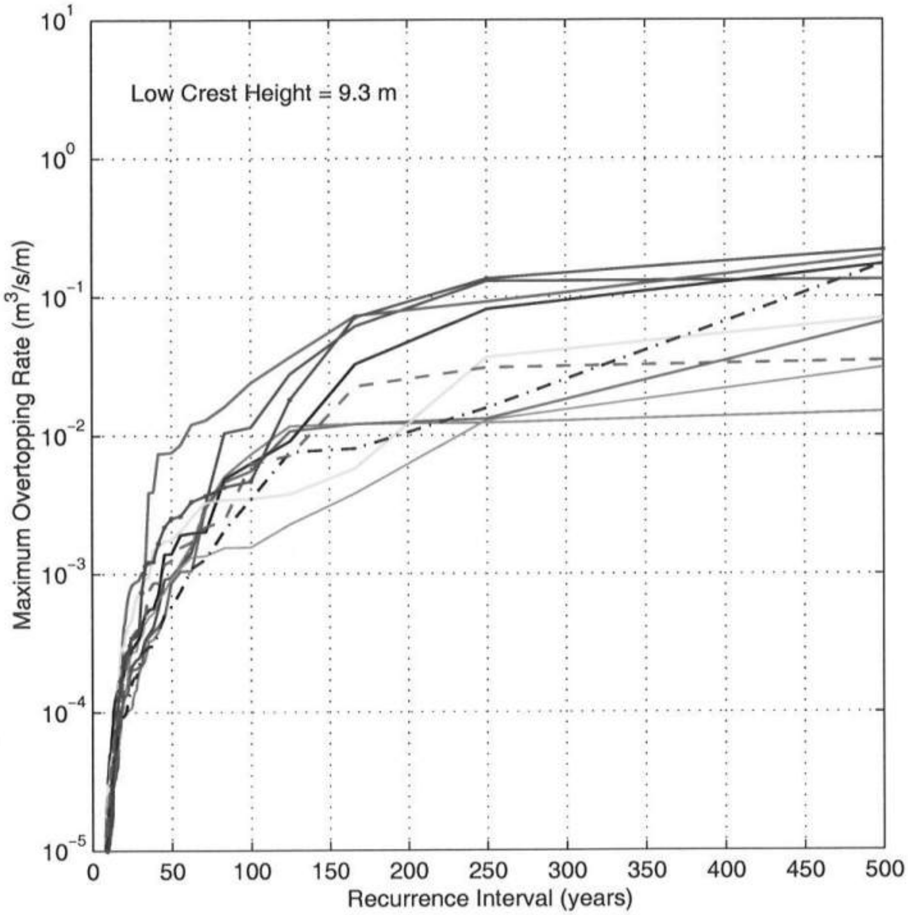

Figure 4.24: Maximum Overtopping Rate for Low Structure Crest on 1/40 Slope at Location of d = 2 m as a Function of the Recurrence Interval for Ten 500-yr Simulations: (Blue —) N500=1; (Red —) N500=2; (Green —) N500=3; (Cyan —) N500=4; (Magenta —) N500=5; (Black —) N500=6; (Yellow —) N500=7; (Blue -•-) N500=8; (Black -.-) N500=9; (Red - -) N500=10.

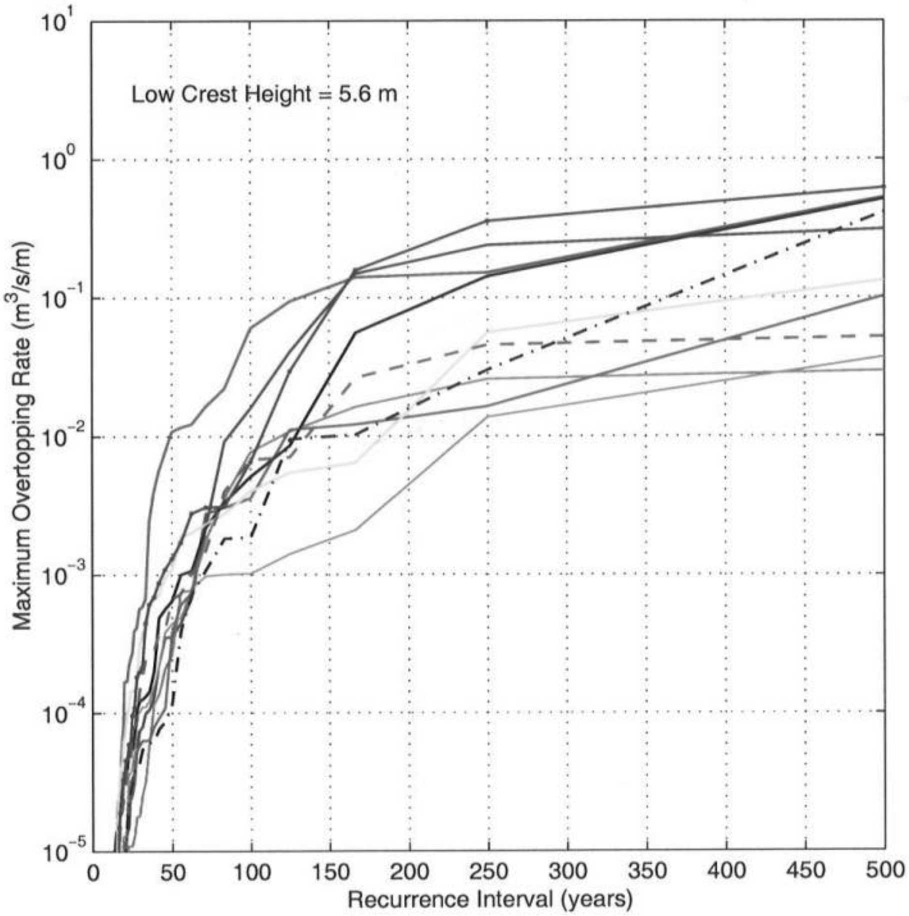

Figure 4.25: Maximum Overtopping Rate for Low Structure Crest on 1/40 Slope at Location of d = 0 m as a Function of the Recurrence Interval for Ten 500-yr Simulations: (Blue —) N500=1; (Red —) N500=2; (Green —) N500=3; (Cyan —) N500=4; (Magenta —) N500=5; (Black —) N500=6; (Yellow —) N500=7; (Blue -•-) N500=8; (Black -.-) N500=9; (Red - -) N500=10.

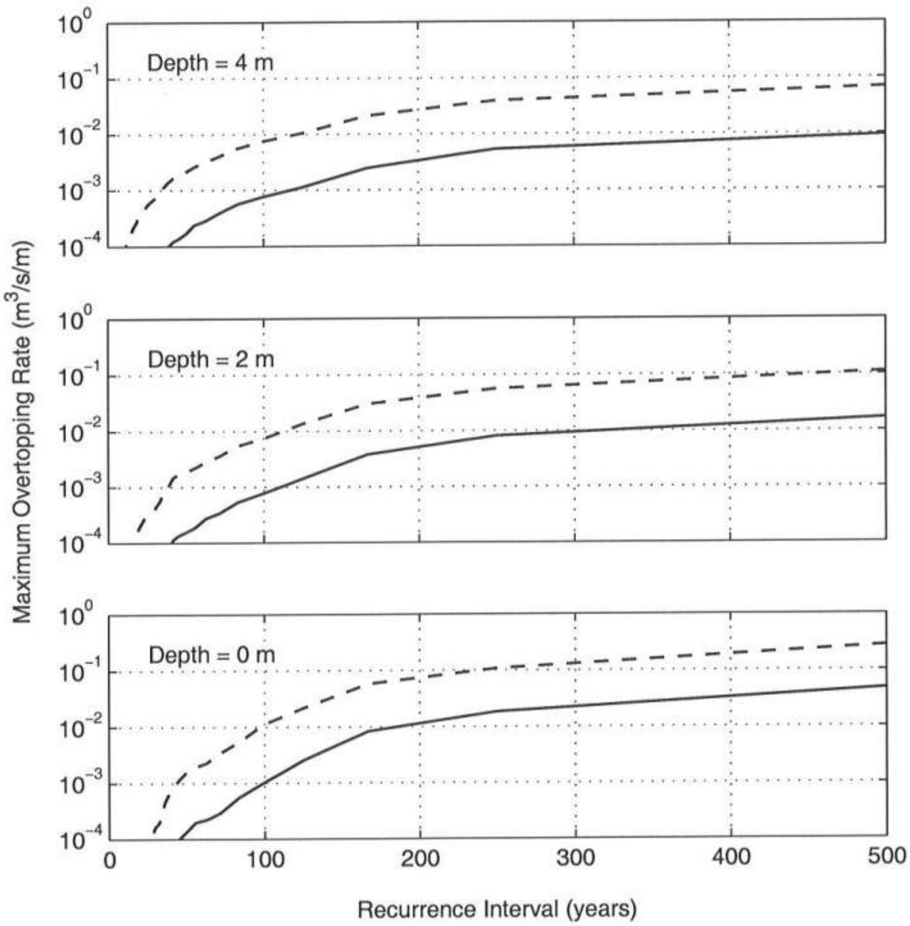

Figure 4.26: Maximum Overtopping Rate Averaged for Ten 500-yr Simulations for High and Low Structure Crests on 1/40 Slope at Locations of d = 4, 2 and 0 m: (—) High Crest; (- -) Low Crest.

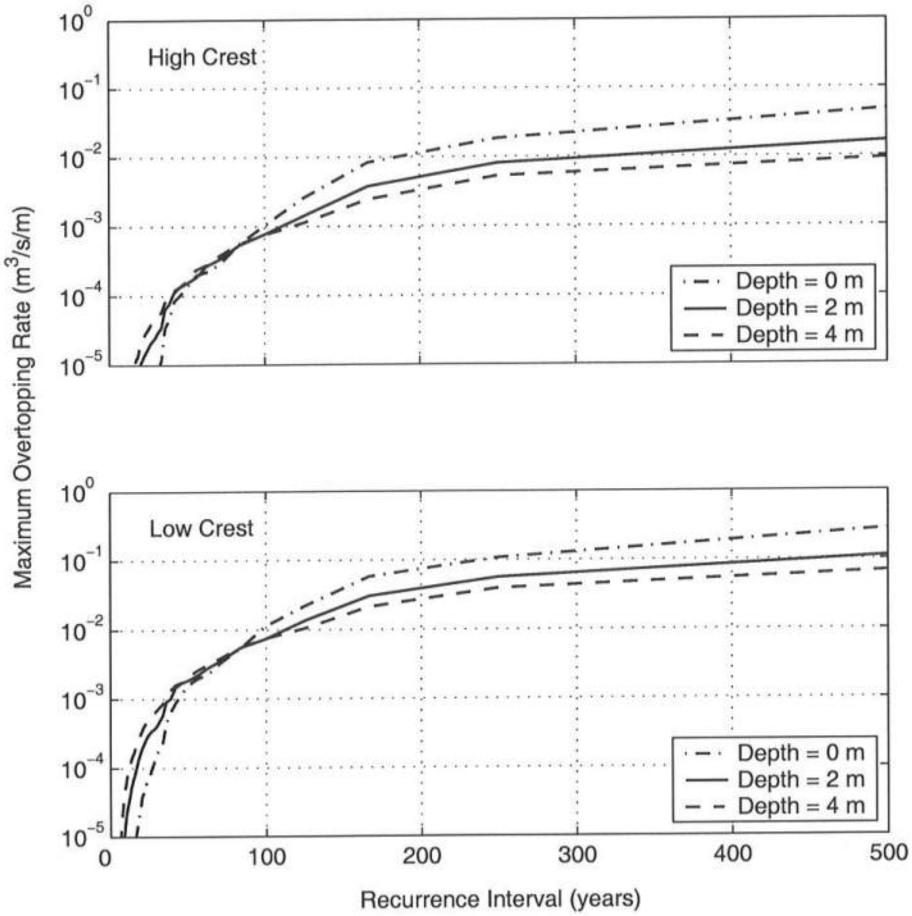

Figure 4.27: Comparison of Maximum Overtopping Rate Averaged for Ten 500-yr Simulations on 1/40 Slope at Locations of $d = 4$, 2 and 0 m for High and Low Structure Crests.

4.4.2 Overtopping Water Volume During Storm and Equivalent Overtopping Duration

For completeness, the overtopping water volume during each storm in each 500-yr simulation on the slope of 1/40 is presented in Figs. 4.28 to 4.33 for the high and low crest heights at the locations of depth $d = 4$, 2 and 0 m. The overtopping water volume averaged for the ten simulations and the comparison for the locations of $d = 4$, 2 and 0 m are illustrated in Figs. 4.34 and 4.35, respectively. The difference of one order of magnitude between the ten simulations and between the high and low crest heights observed in the previous figures is still apparent in these figures.

Fig. 4.34 in comparison to Fig. 4.17 indicates that the overtopping water volume on the 1/800 bottom slope is slightly larger for the recurrence interval exceeding 100 years for both high and low crests at the three locations of depth $d =$ 4, 2 and 0 m.

Finally, Fig. 4.36 shows the equivalent overtopping duration calculated using the computed results in Figs. 4.26 and 4.34. Fig. 4.36 indicates that the equivalent overtopping duration for the 1/40 slope is approximately 3 hr in agreement with the overtopping duration for the 1/800 slope shown in Fig. 4.19. The overtopping volume for hurricanes may hence be estimated approximately by multiplying the maximum overtopping rate at the peak of a storm by 3 hr.

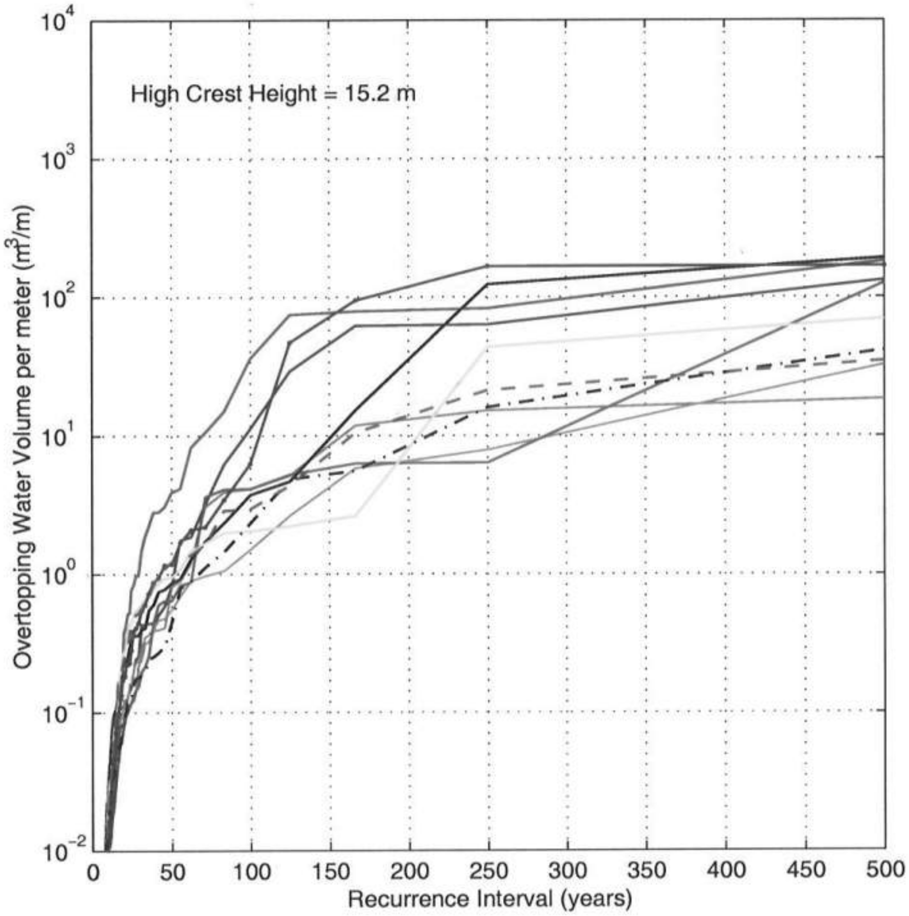

Figure 4.28: Overtopping Water Volume During Storm for High Structure Crest on 1/40 Slope at Location of $d = 4$ m as a Function of the Recurrence Interval for Ten 500-yr Simulations: (Blue —) N500=1; (Red —) N500=2; (Green —) N500=3; (Cyan —) N500=4; (Magenta —) N500=5; (Black —) N500=6; (Yellow —) N500=7; (Blue -•-) N500=8; (Black -.-) N500=9; (Red - -) N500=10.

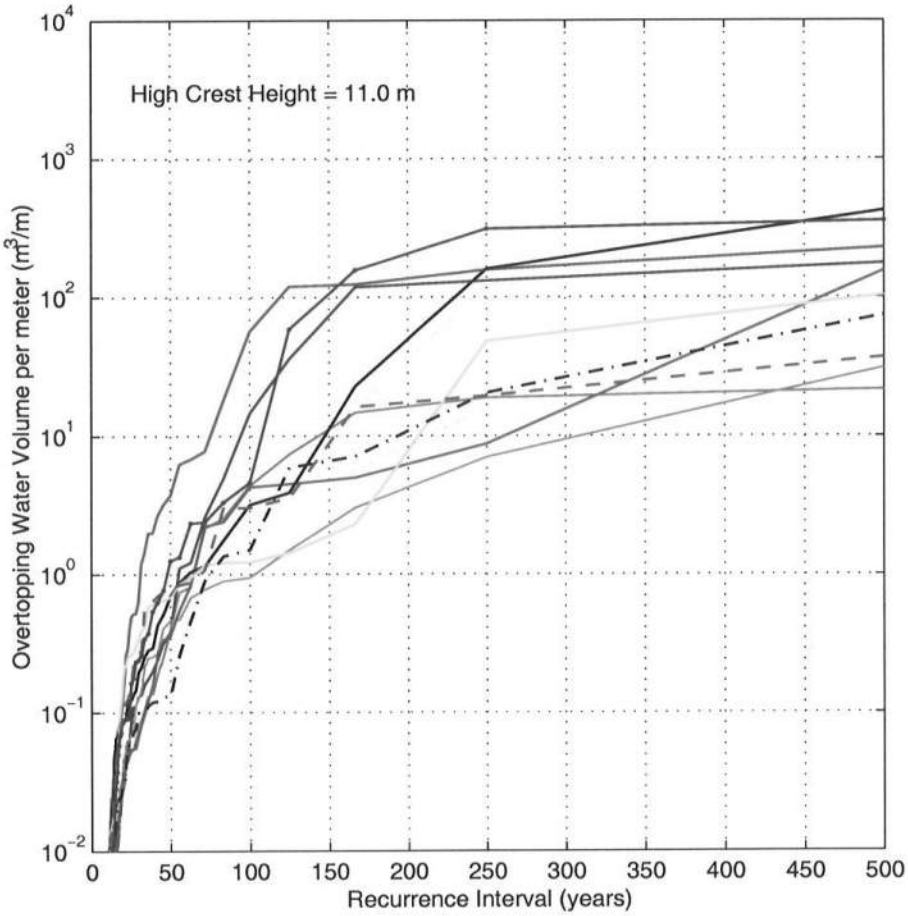

Figure 4.29: Overtopping Water Volume During Storm for High Structure Crest on 1/40 Slope at Location of $d = 2$ m as a Function of the Recurrence Interval for Ten 500-yr Simulations: (Blue —) N500=1; (Red —) N500=2; (Green —) N500=3; (Cyan —) N500=4; (Magenta —) N500=5; (Black —) N500=6; (Yellow —) N500=7; (Blue -•-) N500=8; (Black -.-) N500=9; (Red - -) N500=10.

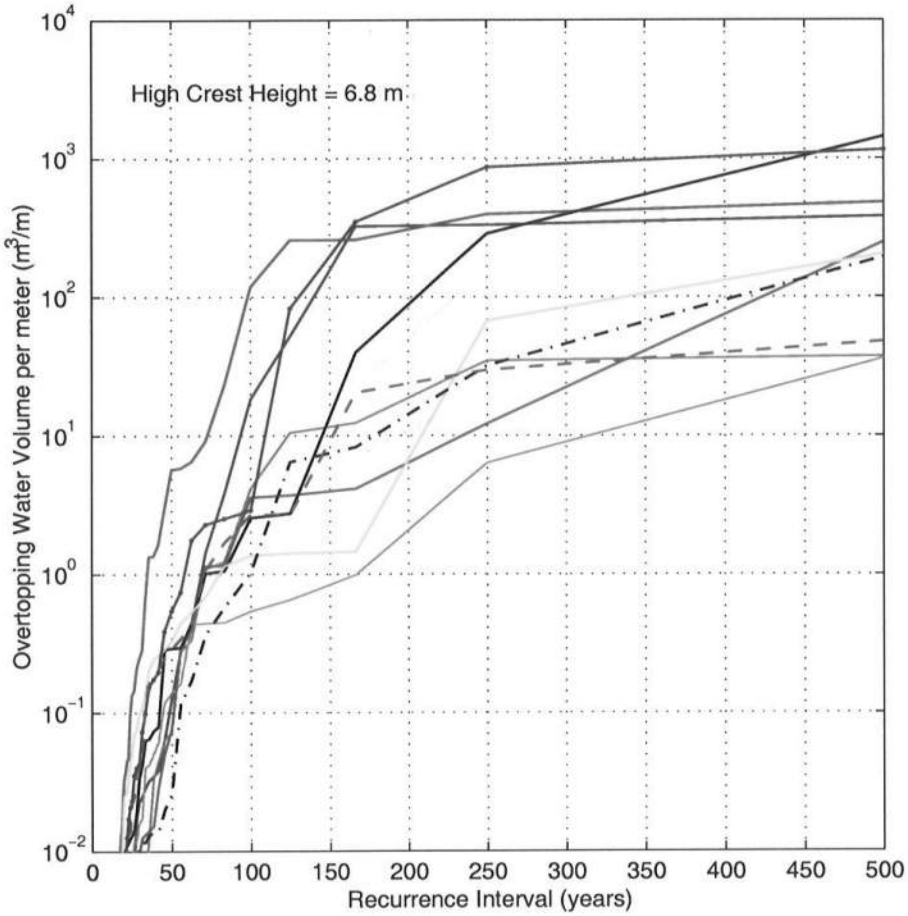

Figure 4.30: Overtopping Water Volume During Storm for High Structure Crest on 1/40 Slope at Location of $d = 0$ m as a Function of the Recurrence Interval for Ten 500-yr Simulations: (Blue —) N500=1; (Red —) N500=2; (Green —) N500=3; (Cyan —) N500=4; (Magenta —) N500=5; (Black —) N500=6; (Yellow —) N500=7; (Blue -•-) N500=8; (Black -.-) N500=9; (Red - -) N500=10.

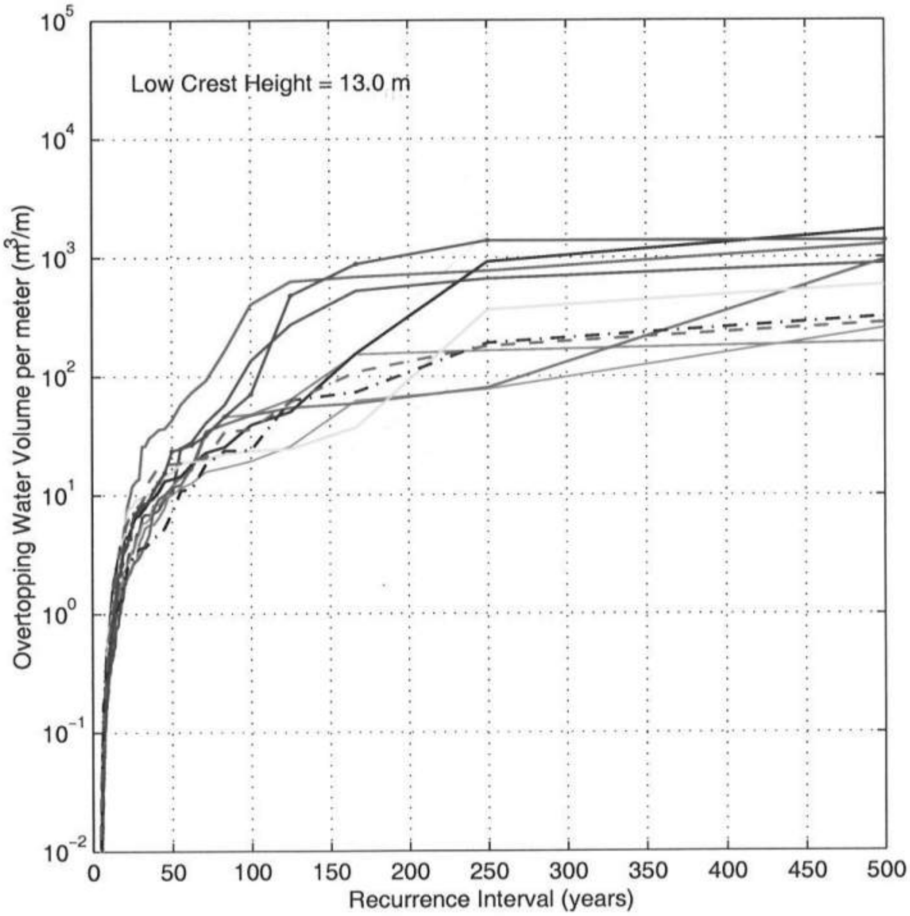

Figure 4.31: Overtopping Water Volume During Storm for Low Structure Crest on 1/40 Slope at Location of $d = 4$ m as a Function of the Recurrence Interval for Ten 500-yr Simulations: (Blue —) N500=1; (Red —) N500=2; (Green —) N500=3; (Cyan —) N500=4; (Magenta —) N500=5; (Black —) N500=6; (Yellow —) N500=7; (Blue -•-) N500=8; (Black -.-) N500=9; (Red - -) N500=10.

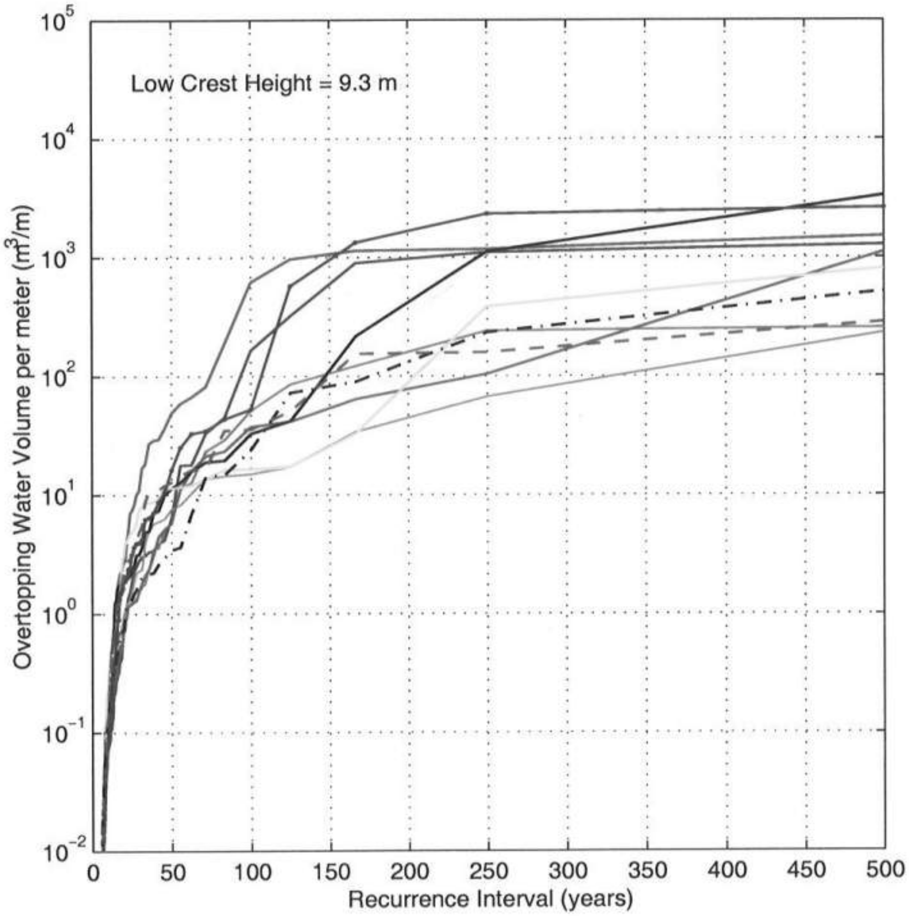

Figure 4.32: Overtopping Water Volume During Storm for Low Structure Crest on 1/40 Slope at Location of $d = 2$ m as a Function of the Recurrence Interval for Ten 500-yr Simulations: (Blue —) N500=1; (Red —) N500=2; (Green —) N500=3; (Cyan —) N500=4; (Magenta —) N500=5; (Black —) N500=6; (Yellow —) N500=7; (Blue -•-) N500=8; (Black -.-) N500=9; (Red - -) N500=10.

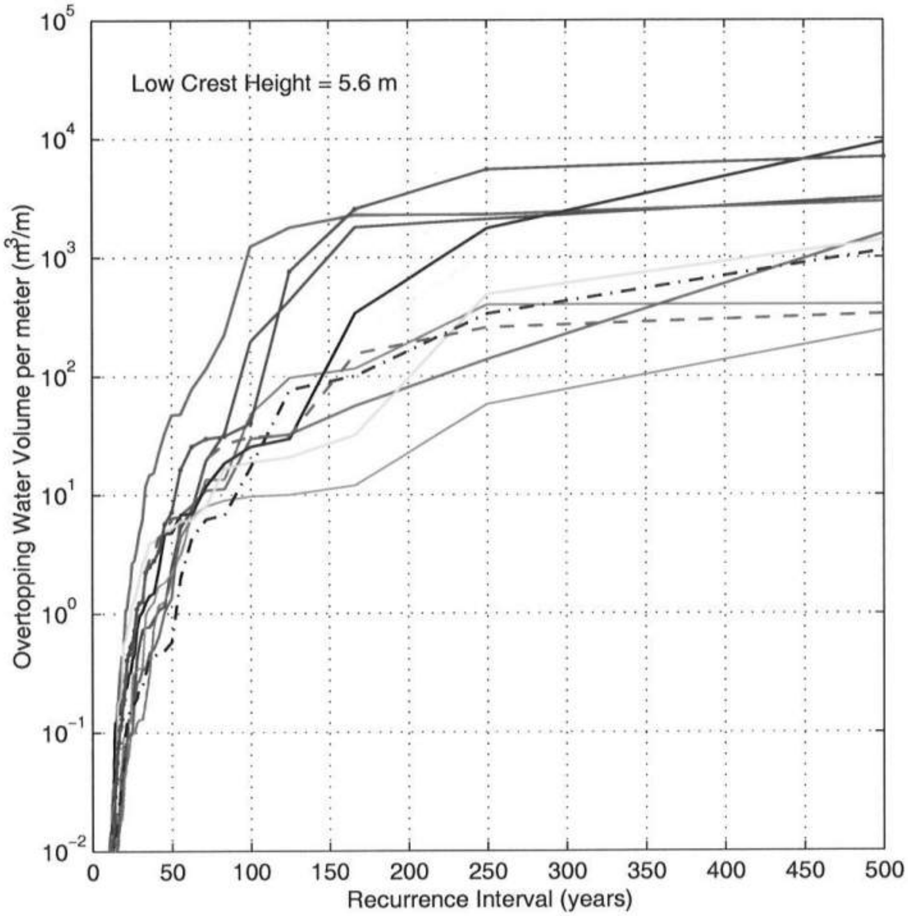

Figure 4.33: Overtopping Water Volume During Storm for Low Structure Crest on 1/40 Slope at Location of $d = 0$ m as a Function of the Recurrence Interval for Ten 500-yr Simulations: (Blue —) N500=1; (Red —) N500=2; (Green —) N500=3; (Cyan —) N500=4; (Magenta —) N500=5; (Black —) N500=6; (Yellow —) N500=7; (Blue -•-) N500=8; (Black -.-) N500=9; (Red - -) N500=10.

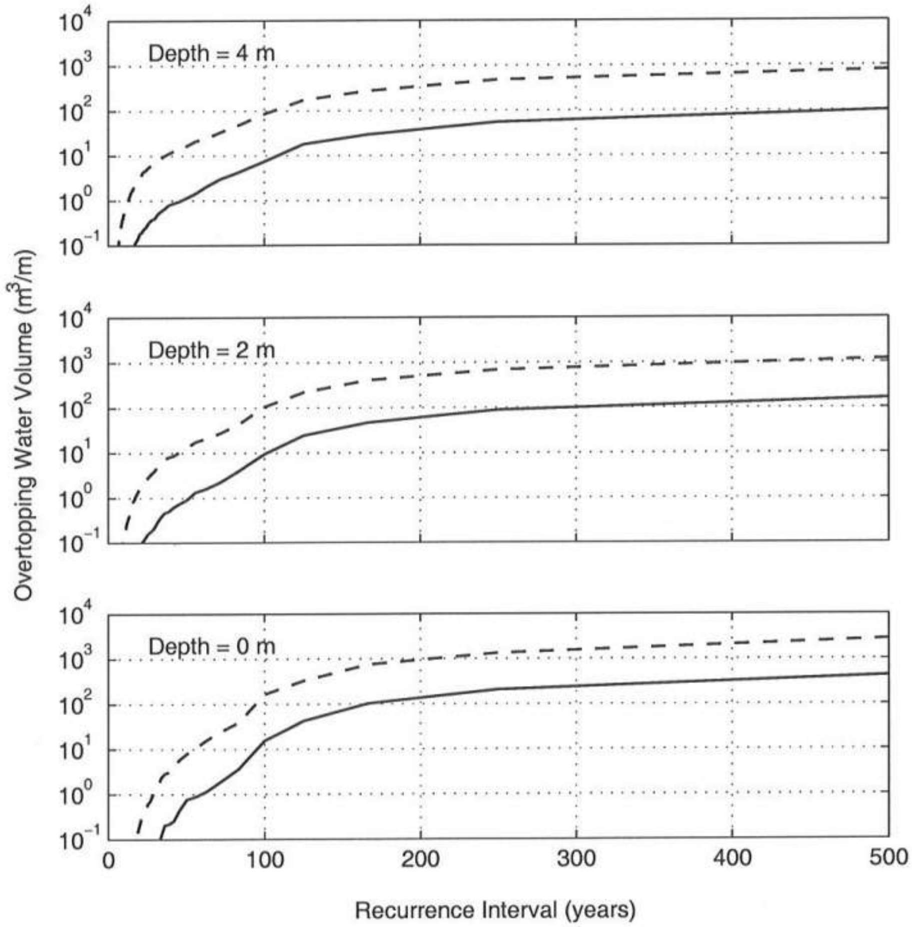

Figure 4.34: Overtopping Water Volume Averaged for Ten 500-yr Simulations for High and Low Structure Crests on 1/40 Slope at Locations of $d = 4$, 2 and 0 m: (—) High Crest; (- -) Low Crest.

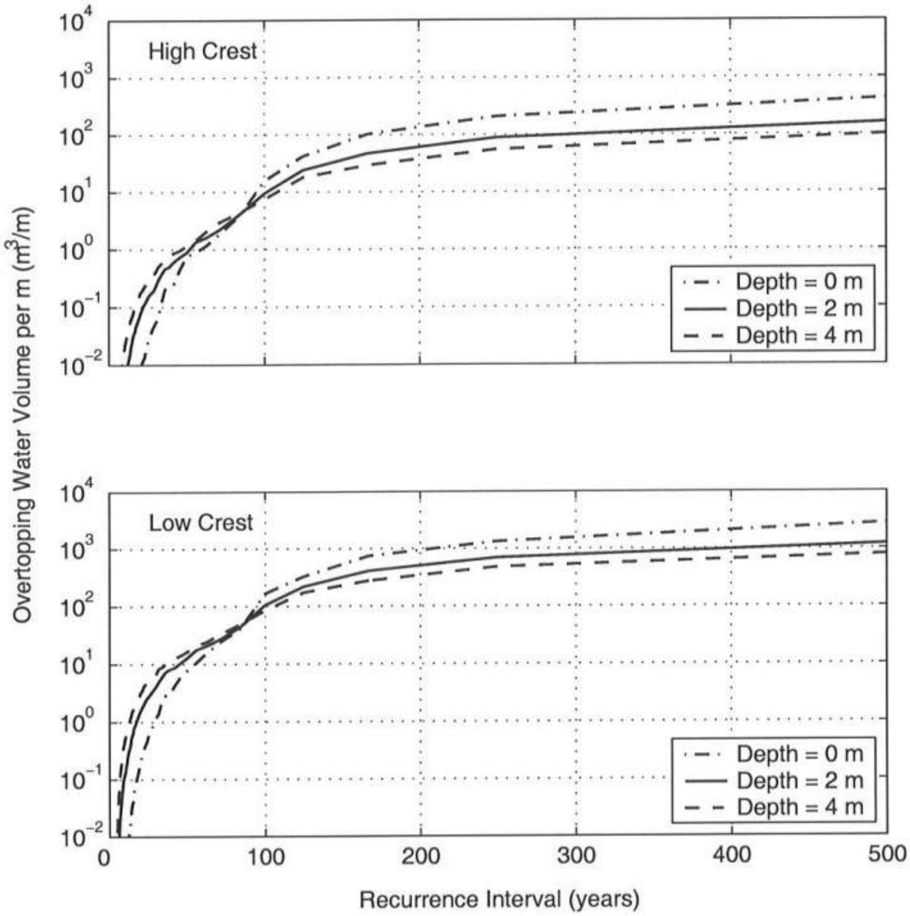

Figure 4.35: Comparison of Overtopping Water Volume Averaged for Ten 500-yr Simulations on 1/40 Slope at Locations of $d = 4$, 2 and 0 m for High and Low Structure Crests.

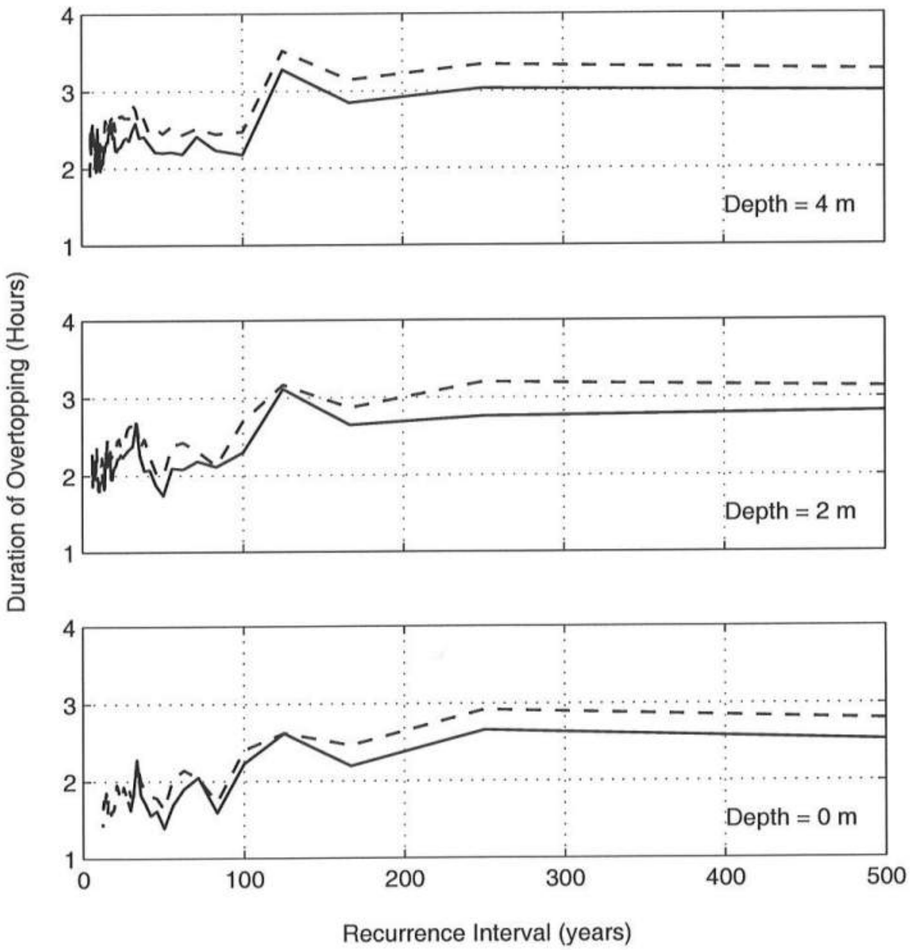

Figure 4.36: Equivalent Overtopping Duration Based on Overtopping Volume and Maximum Overtopping Rate Averaged for Ten 500-yr Simulations for High and Low Structure Crests on 1/40 Slope at Locations of $d = 4$, 2 and 0 m: (—) High Crest; (- -) Low Crest.

Chapter 5

NUMERICAL MODEL DAMAGE: ARMOR LAYER DAMAGE PROGRESSION

Rubble mound breakwater projects require an accurate prediction of damage as part of life cycle analysis. Most breakwater armor stability studies were intended to determine damage for constant wave conditions at the peak of a design storm. The majority of these studies were begun with an undamaged structure and damage measured for a single design wave condition [e.g., Hudson (1959), Shore Protection Manual (1984), Van der Meer (1988a, b)]. The empirical formulas developed in such studies were useful for determining the damage during the peak of a design storm but not for damage progression through several storm events. In summary, the existing techniques are not suited for predicting the future performance of existing structures and estimating the life-cycle costs and maintenance requirements.

To address these unsolved problems, Melby and Kobayashi (1998a, 1998b, 1999, 2000) developed a new empirical model that allowed the prediction of deterioration with time and variability in damage of conventional rubble mound breakwaters exposed to depth-limited breaking waves in sequences of storms with varying wave conditions and water levels.

In this chapter, the armor layer of the rubble mound structure, whose crest height has been designed for minor overtopping in Chapter 4, is first designed against the 100-yr storm and then exposed to the storms generated in the ten 500-yr simulations. The following analysis procedure in the numerical model DAMAGE is similar to that used to assess the performance of the structure against wave overtopping except that damage to the armor layer is cumulative from one storm to the next. As a result, the damage is predicted for the entire sequence of storms in each 500-yr simulation.

5.1 Estimation of Armor Stone Mass against 100-yr Storm

Extensive research on breakwater armor stability has produced many empirical stability formulas. The median mass M_{50} of the armor stone required against the 100-yr storm conditions in Table 4.1 is estimated in this study using the empirical formulas by Hudson (Shore Protection Manual, 1984) and Van der Meer (1988a, b).

5.1.1 Hudson's Formula

The most widely known empirical stability formula was developed by Hudson (1959). The Hudson formula is expressed in the following form

$$M_{50} = \frac{\rho_a H_{10}^3}{K_D \Delta^3 \cot\alpha} \quad ; \quad \Delta = \frac{\rho_a}{\rho} - 1 \tag{5.1}$$

where M_{50} = median mass of individual armor units in the primary cover layer; ρ_a = armor stone density; H_{10} = design wave height at the structure toe; K_D = empirical stability coefficient; ρ_a/ρ = specific gravity of the armor unit material; ρ = sea water density equal to $\rho = 1{,}025\ kg/m^3$; and α = seaside angle of the armor slope relative to horizontal.

Hudson also expressed this equation in a slightly different form as

$$N_s = (K_D \cot\alpha)^{1/3} = \frac{H_{10}}{\Delta D_{n50}} \tag{5.2}$$

where N_s = dimensionless stability number; and D_{n50} = nominal stone diameter given by $D_{n50} = (M_{50}/\rho_a)^{1/3}$.

The Hudson formula was based on regular wave experiments. To apply this formula to irregular waves, the Shore Protection Manual (1984) proposed the use of H_{10} as a representative wave height of irregular waves, where H_{10} is the average height of the highest 1/10 of waves, but the justification of this particular wave height is unclear. As a result, H_{10} is used in equations 5.1 and 5.2

5.1.2 Van der Meer's Formula

Most stability experiments were conducted on undamaged breakwaters in relatively deep water to reduce the number of parameters involved in the resulting empirical formulas. Van der Meer (1988b) conducted 16 tests for depth-limited breaking waves and proposed the use of $H_{2\%}$ for the stability number, where $H_{2\%}$ is the wave height exceeded by 2% of waves. His previous formula for non-breaking waves (Van der Meer 1988a) was extended to depth-limited breaking waves in relatively deep water using $H_{2\%} = 1.40H_s$ based on the Rayleigh distribution for wave heights, where the surf similarity parameter ξ_m based on H_s was not changed.

Based on the limited data for depth-limited breaking waves, he proposed the following two equations for the armor stability,
For plunging waves:

$$\frac{H_{2\%}}{\Delta D_{n50}} = 8.7P^{0.18}\left(\frac{S}{\sqrt{N}}\right)^{0.2}\xi_m^{-0.5} \qquad \text{for } \xi_m < \xi_c = \left(6.2P^{0.31}\sqrt{\tan\alpha}\right)^{\frac{1}{P+0.5}} \tag{5.3}$$

and for surging waves:

$$\frac{H_{2\%}}{\Delta D_{n50}} = 1.4P^{-0.13}\left(\frac{S}{\sqrt{N}}\right)^{0.2}\sqrt{\cot\alpha}\,\xi_m^{P} \qquad \text{for } \xi_m > \xi_c = \left(6.2P^{0.31}\sqrt{\tan\alpha}\right)^{\frac{1}{P+0.5}} \tag{5.4}$$

where $\Delta = (\rho_a/\rho - 1)$ with ρ_a/ρ = specific gravity of the armor unit; D_{n50} = nominal stone diameter based on the median stone mass M_{50} defined as $D_{n50} = (M_{50}/\rho_a)^{1/3}$; P = empirical permeability coefficient; S = dimensionless damage defined as $S = A_e/D_{n50}^2$ with A_e = cross-sectional eroded area; N = number of waves usually in the range N = 1000 - 5000 waves during the storm peak; α = structure slope angle; and ξ_m = surf similarity parameter given by $\xi_m = \tan\alpha/\sqrt{H_s/L_o}$ with H_s = significant wave height, $L_o = gT_m^2/(2\pi H_s)$ and T_m = mean wave period.

Van der Meer noted that $S = 2$ was a good estimate of the initiation of damage and that local failure occured when $S = 8$ for the structure slopes of 1/1.5 and 1/2, where failure was defined as exposure of the underlayer through a hole of D_{n50} diameter. Van der Meer used test durations of N = 1,000 and 3,000, where the number of waves N is defined as $N = t/T_m$ with t = test duration and T_m = mean wave period.

5.1.3 Estimation of the Characteristic Wave Heights in Hudson and Van der Meer Formulas

Design criteria for coastal structures with respect to wave forces, armor stability, wave runup or wave overtopping involve at least one characteristic wave height of the incident waves, typically the significant wave height H_s or a wave height with some low exceedance probability. If the wave heights are Rayleigh-distributed, these heights can all be converted from one into another through known constants, but if the distribution is distorted due to shallow-water breaking, these ratios are not constant and need to be estimated empirically.

The values of H_{10}, $H_{2\%}$ and H_s for the known values of the still water depth d_s, the spectral significant wave height H_{mo}, and the bottom slope are estimated using the wave height distribution on shallow foreshores proposed by Battjes and Groenendijk (2000), where wave setup was not accounted for because their data did not include very shallow water.

As a working hypothesis, they assumed a combination of two Weibull distributions matched at the transitional wave height H_{tr}. A simple parameterisation for this transitional wave height H_{tr} and the root-mean-squared wave height H_{rms} was proposed to develop a predictive model for the local wave height distribution, which uses as input only the local wave energy, depth and bottom slope.

The transitional wave height H_{tr} and root-mean-squared wave height H_{rms} are related to the local depth, bottom slope and wave energy as follows

$$H_{tr} = (0.35 + 5.8 \tan \theta_{bottom}) d_s \tag{5.5}$$

$$H_{rms} = (2.69 + 3.24\sqrt{m_o}/d_s)\sqrt{m_o} \tag{5.6}$$

where θ_{bottom} = bottom slope; m_o = local zero-th spectral moment with $H_{mo} = 4\sqrt{m_o}$; and d_s = still water depth.

By normalizing all wave heights with H_{rms}, only the normalized transitional wave height, $\tilde{H}_{tr} = H_{tr}/H_{rms}$, was involved in the distribution of the normalized wave heights. Using Table 2 in Battjes and Groenendijk (2000), the values of the normalized wave heights, $\tilde{H}_{1/10}$, $\tilde{H}_{2\%}$ and $\tilde{H}_{1/3}$, can be estimated. The dimensional wave heights are then calculated by multiplying their normalized values by H_{rms}.

$$H_s = H_{1/3} = \tilde{H}_{1/3} H_{rms} \tag{5.7}$$

$$H_{10} = H_{1/10} = \tilde{H}_{1/10} H_{rms} \tag{5.8}$$

$$H_{2\%} = \tilde{H}_{2\%} H_{rms} \tag{5.9}$$

where Battjes and Groenendijk (2000) used the notations of $H_{1/3}$ and $H_{1/10}$ instead of H_s and H_{10} used here.

5.2 Damage Progression

5.2.1 Prediction of Damage Increase by Many Storms

The formulas by Hudson and Van der Meer in section 5.1 are limited to constant incident wave conditions and water levels as well as the initial condition of an undamaged structure. These formulas are intended to predict damage during the peak of a single design storm. Therefore, they cannot be used to predict damage development over the life of a structure as is required in a life-cycle cost analysis.

Melby and Kobayashi (1998a, 1998b, 1999, 2000) developed an empirical procedure based on laboratory experiments for traditional rubble mound breakwaters with a 1/2 slope exposed to depth-limited irregular breaking waves in order to determine the damage progression with time for a series of storms.

The damage S_{n+1} at a certain time t_{n+1} is expressed as a function of the damage S_n at time t_n (Melby and Kobayashi 1999, 2000)

$$S_{n+1} = aN_{mo}^5 (N_e + \delta N)^b \qquad \text{for } N_{mo} > N_c \qquad (5.10a)$$

$$S_{n+1} = S_n \qquad \text{for } N_{mo} \leq N_c \qquad (5.10b)$$

with

$$N_{mo} = \frac{H_{mo}}{\Delta D_{n50}} \qquad (5.11)$$

$$N_e = \left(\frac{S_n}{aN_{mo}^5} \right)^{1/b} \qquad (5.12)$$

$$\delta N = \frac{t_{n+1} - t_n}{T_m} \qquad (5.13)$$

where S_{n+1} and S_n = damage at the time levels t_{n+1} and t_n, respectively; N_c = critical stability number described in the following section; N_{mo} = stability number based on the spectral significant wave height H_{mo} during the time interval $t_n \leq t \leq t_{n+1}$; N_e = equivalent number of waves based on the stability number N_{mo} during $t_n \leq t \leq t_{n+1}$ to cause the same damage S_n; δN = incremental number of waves during this interval $t_n \leq t \leq t_{n+1}$ of constant wave conditions, where $(t_{n+1} - t_n)$ = 0.5 hr in this computation, and T_m = mean wave period during $t_n \leq t \leq t_{n+1}$; a, b = empirical coefficients associated with the spectral waves which were calibrated by Melby and Kobayashi (2000) to be $a = 0.011$ and $b = 0.25$.

5.2.2 Critical Stability Number

Smith et al. (1992) proposed a formula for N_c on the basis of the Van der Meer formula. The critical stability number for no damage was estimated as

$$N_c = 0.4\frac{6.2P^{0.18}}{\sqrt{\xi_m}} \qquad \text{for } \xi_m < \xi_c = (6.2P^{0.31}\sqrt{\tan\alpha})^{\frac{1}{P+0.5}} \tag{5.14}$$

$$N_c = 0.4\xi_m^P\frac{\sqrt{\cot\alpha}}{P^{0.13}} \qquad \text{for } \xi_m = \frac{\tan\alpha}{\sqrt{\frac{2\pi H_s}{gT_m^2}}} > \xi_c \tag{5.15}$$

where N_c = critical stability number; P = empirical permeability coefficient; ξ_m = surf similarity parameter; $\tan\alpha$ = structure seaward slope; H_s = significant wave height at the toe of the structure; and T_m = mean peak period.

Their formula predicts N_c as a function of the surf similarity parameter for the permeability coefficient $P = 0.4$ for traditional rubble mound breakwaters and the structure slope of 1/2 considered here.

$$N_c = \frac{2.10}{\sqrt{\xi_m}} \qquad \text{for } \xi_m < \xi_c = 3.77 \tag{5.16}$$

$$N_c = 0.637\xi_m^{0.4} \qquad \text{for } \xi_m > \xi_c = 3.77 \tag{5.17}$$

which is plotted in Fig. 5.1. The critical stability number N_c is not very sensitive to the surf similarity parameter ξ_m for $\xi_m > 2$ and the minimum value is $N_c = 1.08$ at $\xi_m = 3.77$.

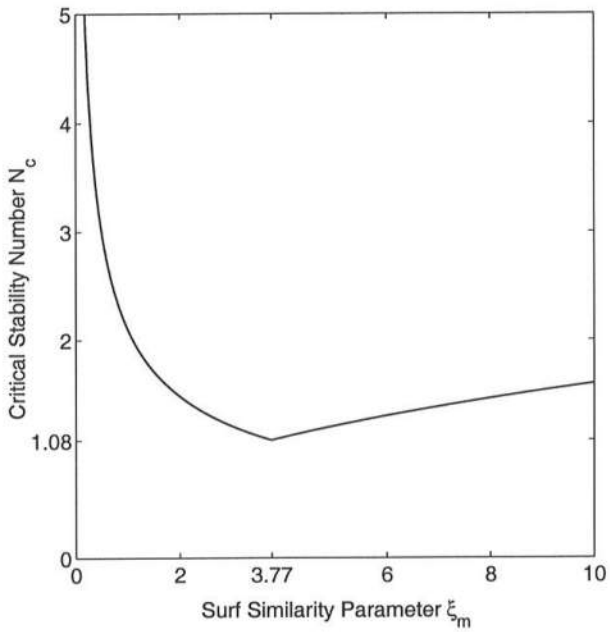

Figure 5.1: Variation of Critical Stability Number as a Function of the Surf Similarity Parameter.

To show that the use of $N_{mo} = N_c = 1.08$ causes very little additional damage to the structure, the damage increment $(S_{n+1} - S_n)$ is computed as a function of the existing damage S_n for δN = 1,000, 3,000 and 5,000 in equation 5.10a where $N_{mo} = N_c$ is used even though N_{mo} must exceed N_c in equation 5.10a.

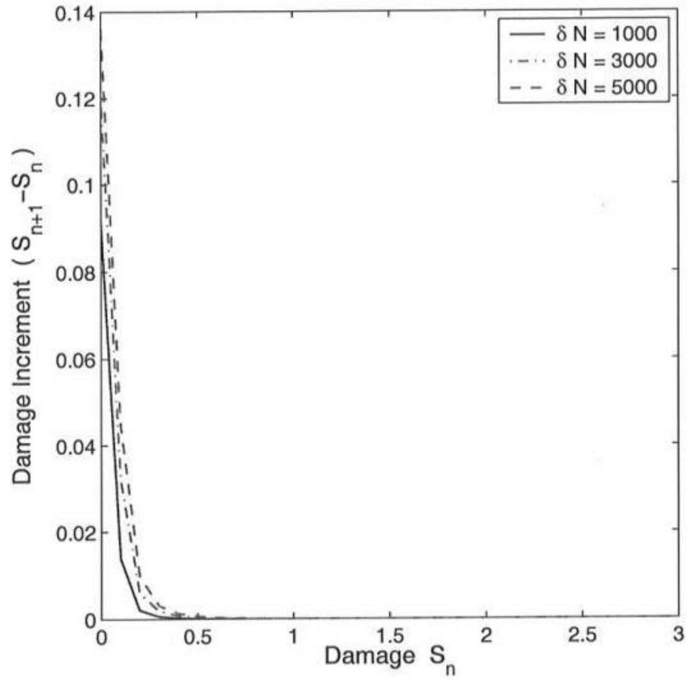

Figure 5.2: Damage Increment $(S_{n+1} - S_n)$ for $N_{mo} = N_c = 1.08$ as a Function of Existing Damage S_n due to Number of Waves δN = 1,000, 3,000 and 5,000.

Therefore, the predicted minimum value of $N_c = 1.08$ for the surf similarity parameter $\xi_m = 3.77$ is adopted in this study because of the uncertainty of the slope in the surf similarity parameter for depth-limited breaking waves as discussed by Melby and Kobayashi (1998a). In any event, the use of $S_{n+1} = S_n$ for $N_{mo} \leq N_c = 1.08$ in equation 5.10*b* may be appropriate to account for no additional damage by relatively small waves.

5.3 Input to DAMAGE

First, the median mass M_{50} of the armor stone is designed against the 100-yr storm conditions listed in Table 4.1. The required median mass is estimated using the empirical formulas by Hudson (Shore Protection Manual 1984) and Van der Meer (1988a,b) together with the wave height distribution on shallow foreshores proposed by Battjes and Groenendijk (2000). The computed results will later be shown in Tables 5.1, 5.2 and 5.3 for the 1/800 bottom slope and in Tables 5.5, 5.6 and 5.7 for the 1/40 bottom slope.

The seaward slope of the armor layer is taken as 1/2. The densities of the armor stone and seawater are assumed to be $\rho_a = 2{,}660$ kg/m^3 and $\rho = 1{,}025$ kg/m^3.

The stability coefficient K_D in Hudson's formula in equations 5.1 and 5.2, varies primarily with the shape of the armor units, roughness of the armor unit surface, and degree of interlocking obtained in placement. For rough angular stones placed randomly in a two-layer thickness and breaking waves on the structure trunk, the recommended value is $K_D = 2$.

The formula of Van der Meer (1988a,b) in equations 5.3 and 5.4 requires additional input. The mean wave period T_m may be estimated as $T_m \simeq T_s/1.2$ (Goda 1985) where T_s is the significant wave period predicted by the offshore hurricane wave model. The permeability coefficient P is taken as $P = 0.4$ for the traditional two-diameter thick armor layer with a filter layer. The damage S defined as the eroded area divided by D_{n50}^2 is taken as $S = 2$ after being exposed to $N = 1{,}000$ waves during the peak of the 100-yr storm for which $T_s = 12.3$ s, $T_m \simeq 10.3$ s and 1000 $T_m \simeq 2.8$ hr.

Second, for the estimated D_{n50} at the location specified by the depth d below MSL on the selected bottom slope, the armor layer with zero damage initially is exposed to the time series of H_{mo} and $T_m \simeq T_s/1.2$ stored at an interval of

0.5 hr at its toe during the entire duration of each storm in each 500-yr simulation. The damage progression is computed using the empirical equation 5.10 with $a = 0.011$ and $b = 0.25$ by Melby and Kobayashi (2000), in which the value of the critical stability number has been assumed to be $N_c = 1.08$. The computed results for the damage progression are described in Sections 5.4 and 5.5 for the 1/800 and 1/40 bottom slopes, respectively.

5.4 Computed Results for Bottom Slope of 1/800

5.4.1 Estimated Armor Stone Mass M_{50}

First, the values of the characteristic wave heights are estimated using the wave height distribution on shallow foreshores proposed by Battjes and Groenendijk (2000) as described in Section 5.1.3. The design wave height for the Hudson formula is H_{10} = average height of the highest 10% of waves. For depth-limited breaking waves, Van der Meer (1988b) suggested the use of $H_{2\%}$ = wave height exceeded by 2% of waves. The estimated values of H_{10} and $H_{2\%}$ at the locations of $d = 4$, 2 and 0 m on the 1/800 slope are shown in Table 5.1. The ratios between H_{10} and H_{mo} and between $H_{2\%}$ and H_{mo} are 1.16 and 1.23, respectively, in comparison with the ratios 1.27 and 1.40 based on the Rayleigh distribution of wave heights.

Table 5.1: Estimated 100-yr Wave Heights H_{10} and $H_{2\%}$ for 1/800 Bottom Slope.

Water depth below MSL, $d(m)$	4.0	2.0	0.0
Still water depth, $d_s(m)$	6.61	4.61	2.61
Significant wave height, $H_{mo}(m)$	3.04	2.27	1.46
Average height of highest 10%, $H_{10}(m)$	3.54	2.64	1.70
H_{10}/H_{mo}	1.16	1.16	1.16
2% exceedance wave height, $H_{2\%}(m)$	3.74	2.78	1.80
$H_{2\%}/H_{mo}$	1.23	1.23	1.23

Second, the required nominal stone diameter D_{n50} and the corresponding stone mass M_{50} are computed using the estimated 100-yr wave heights H_{10} and $H_{2\%}$ in Table 5.1. Table 5.2 lists the stability number defined as $H_{10}/(\Delta D_{n50})$ with $\Delta = (\rho_a - \rho)/\rho = 1.60$ in this computation, the nominal diameter $D_{n50} = (M_{50}/\rho_a)^{1/3}$, and the corresponding stone mass M_{50} based on the Hudson formula. In the same manner, Table 5.3 shows the computed results based on the Van der Meer formula where the surf similarity parameter ξ_m is needed in equations 5.3 and 5.4.

Table 5.2: Estimated Median Stone Mass Using Hudson Formula for 1/800 Bottom Slope.

Water depth below MSL, $d(m)$	4.0	2.0	0.0
Stability number, $H_{10}/\Delta D_{n50}$	1.59	1.59	1.59
Nominal diameter, $D_{n50}(m)$	1.39	1.04	0.67
Median stone mass, $M_{50}(ton)$	7.14	2.99	0.79

Table 5.3: Estimated Median Stone Mass Using Van der Meer Formula for 1/800 Bottom Slope.

Water depth below MSL, $d(m)$	4.0	2.0	0.0
Surf similarity parameter, ξ_m	3.66	4.23	5.28
Stability number, $H_{2\%}/\Delta D_{n50}$	2.22	2.29	2.50
Nominal diameter, $D_{n50}(m)$	1.05	0.76	0.45
Median stone mass, $M_{50}(ton)$	3.08	1.16	0.24

Comparison of the corresponding values of D_{n50} for the given d indicates that the value of D_{n50} in Table 5.2 is always larger than that in Table 5.3. The high (Hudson formula) and low (Van der Meer formula) values of D_{n50} in Tables 5.2 and 5.3 are used in the following computation of damage progression.

5.4.2 Damage Progression Caused by Sequences of Storms

To assess the similarity of the model proposed by Melby and Kobayashi (2000) with the Hudson and Van der Meer formulas, equation 5.10 with $S_n = 0$, $\delta N =$ 1,000 and N_{mo} calculated with the corresponding stone diameter is used to estimate the value of S_{n+1} which may be regarded as the damage during the peak of the 100-yr storm. The calculated damage shown in Table 5.4 is in the range of 0.29–0.30 for the Hudson formula and in the range of 1.20–2.13 for the Van der Meer formula. These estimated damages are based on the maximum wave height H_{mo} during the 100-yr storm with 1,000 waves and do not account for the cummulative damage due to many storms.

Table 5.4: Damage S to Armor Layer Designed by Hudson and Van der Meer Formulas Caused by 100-yr Wave Height and 1,000 Waves for 1/800 Bottom Slope.

		Hudson formula			**Van der Meer formula**		
d (m)	H_{mo} (m)	D_{n50} (m)	N_{mo}	S	D_{n50} (m)	N_{mo}	S
4	3.04	1.39	1.37	0.30	1.05	1.81	1.20
2	2.27	1.04	1.37	0.30	0.76	1.88	1.45
0	1.46	0.67	1.36	0.29	0.45	2.03	2.13

The computation of damage progression starts with $S = 0$ at the beginning of the first storm. The damage progression during the j-th storm with $j = 1$, 2, $\cdots$, J and $J =$ number of storms in each 500-yr simulation is computed using equation 5.10. The damage S_j at the end of the j-th storm and the maximum stability number $(N_{mo})_j$ during this storm are stored. The damage progression for the $(j+1)$-th storm starts with its initial damage of S_j. It is convenient if the damage increment $(S_j - S_{j-1})$ during the j-th storm can be estimated using the maximum stability number $(N_{mo})_j$ alone. The equivalent number N_j of waves is computed using equation 5.10 where S_{n+1}, S_n, N_{mo} and δN are replaced by S_j, S_{j-1}, $(N_{mo})_j$ and N_j, respectively. The computation of N_j is made only if $(N_{mo})_j > N_c$ and $(S_j - S_{j-1}) > 0.0001$.

First, the damage progression of the armor layer with the low and high diameters D_{n50} is presented together with the maximum stability number N_{mo} computed for each storm in each of the ten 500-yr simulations at the locations of $d = 4$, 2 and 0 m. Second, the equivalent number of waves to cause the same damage increment are also obtained for the low and high diameters D_{n50} at locations of $d = 4$, 2 and 0 m for each of the ten 500-yr simulations. Third, the damage progression averaged for the ten 500-yr simulations and the damage progression for each of the ten simulations are plotted together to display the statistical variability about the average damage progression. Lastly, the equivalent number of waves for all the ten 500-yr simulations are also presented to estimate an averaged equivalent number of waves.

Due to the large number of figures resulting from this computation, only examples of the typical results are shown in this section. The rest of the results corresponding to each and average of the ten 500-yr simulations are included in Section 5.6 for completeness.

Fig. 5.3 shows the damage S_j at the end of the j-th storm and the maximum stability number $(N_{mo})_j$ with $j = 1, 2, \cdots, J$ for the low and high values of D_{n50} based on the Van der Meer and Hudson formulas, respectively, for one 500-yr simulation. The computed values of S_j and $(N_{mo})_j$ if $(N_{mo})_j > 1$ are plotted as a function of the time $(500\ j/J)$ in years to allow an easier interpretation, where J is the number of storms in 500 years. The example shown in Fig. 5.3 for the armor layer located at the depth $d = 2$ m below MSL on the 1/800 slope is typical for all the computed results presented in Section 5.6.

The progression of damage to the armor layer is caused episodically by several major storms with large values of $(N_{mo})_j$ but slows down as the damaged armor layer ages because the damage increment for the large value of $(N_{mo})_j$ decreases with time. This trend is important in deciding whether and when the damaged armor layer should be repaired.

Fig. 5.4 shows the damage progression of the armor layer with the low and high D_{n50} at $d = 2$ m on the 1/800 slope for each of the ten 500-yr simulations. The damage progression averaged for the ten simulations shown in a thick line in Fig. 5.4 does not reveal the episodic nature of damage progression. The statistical variability about the average damage progression is a factor of about two. This variability is smaller than what may be expected from the term N_{mo}^5 in equation 5.10 but this relatively small variability is related to the aging effect of the damaged armor layer.

Fig. 5.5 shows the equivalent number N_j of waves to cause the same damage increment using the maximum stability number $(N_{mo})_j$ for the j-th storm at the location of $d = 2$ m on the 1/800 slope. The average value of N_j for the ten 500-yr simulations shown in Fig. 5.5 is 1,238 and 961 for the low and high D_{n50}, respectively.

Fig. 5.6 compares the average damage progression for the armor layer with the low and high D_{n50} at the depth $d = 4$, 2 and 0 m on the 1/800 slope. Except for the low D_{n50} for $d = 0$ m, the predicted average damage for the low D_{n50} is less than about four after the 500-yr simulation. The local failure of the armor layer may occur if the damage exceeds about eight (Van der Meer 1988a, b; Melby and Kobayashi 1998a). The low $D_{n50} = 0.45$ m for $d = 0$ m in Table 5.3 may need to be increased if no repair is planned. The other values of the low D_{n50} in Table 5.3 are sufficient even if the damage variability for the ten simulations is accounted for. On the other hand, all the values of the high D_{n50} in Table 5.2 are more than sufficient and can be reduced slightly.

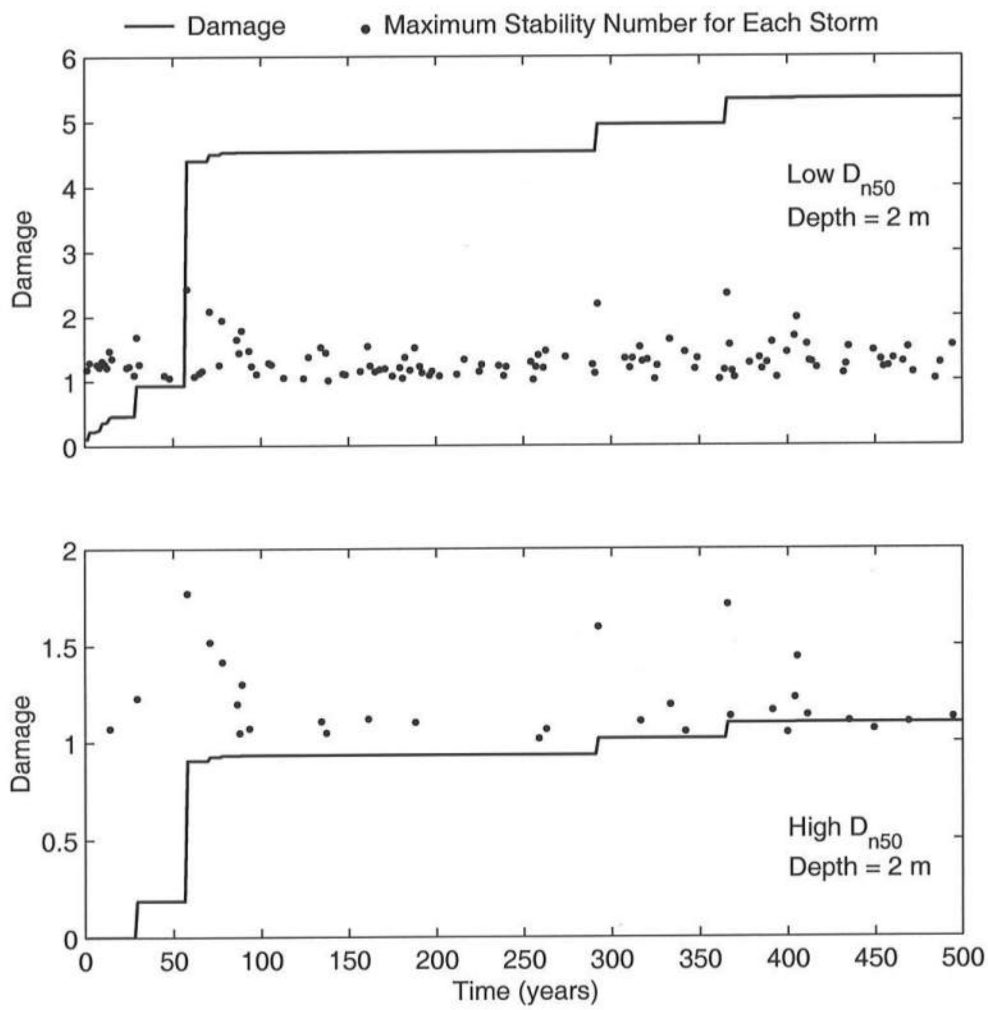

Figure 5.3: Damage Progression of Armor Layer with Low and High Diameters D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/800 Slope at Location of $d = 2$ m for One 500-yr Simulation.

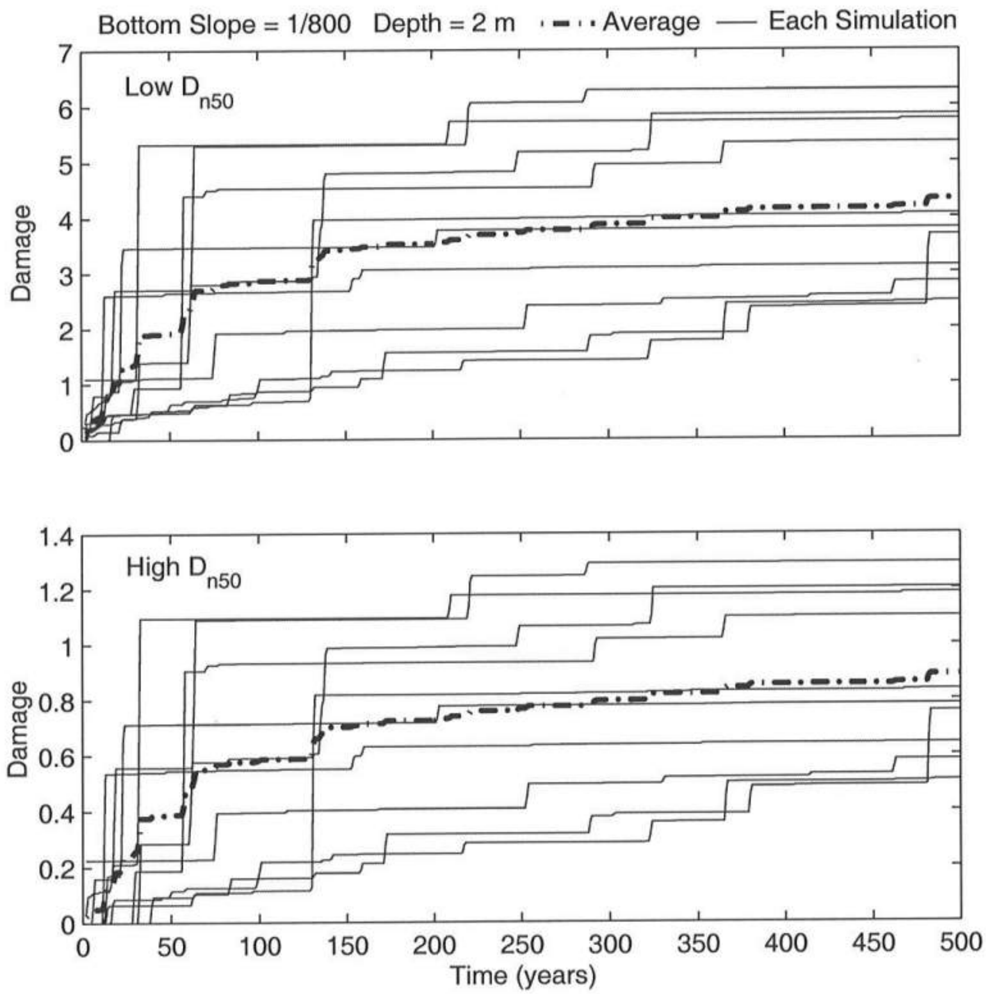

Figure 5.4: Damage Progression of Armor Layer with Low and High Diameters D_{n50} on 1/800 Slope at Location of $d = 2$ m for Each and Average of Ten 500-yr Simulations.

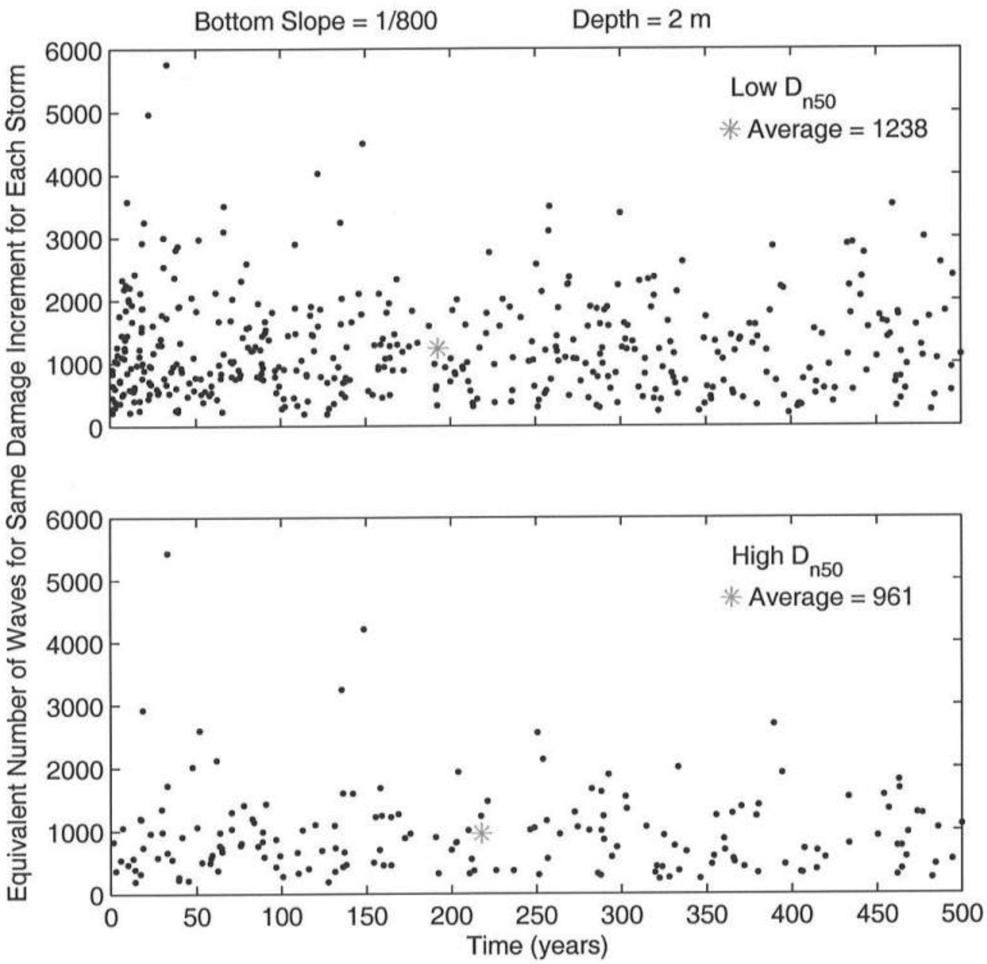

Figure 5.5: Equivalent Number of Waves to Cause the Same Damage Increment for Low and High Diameters D_{n50} on 1/800 Slope at Location of d = 2 m for Ten 500-yr Simulations.

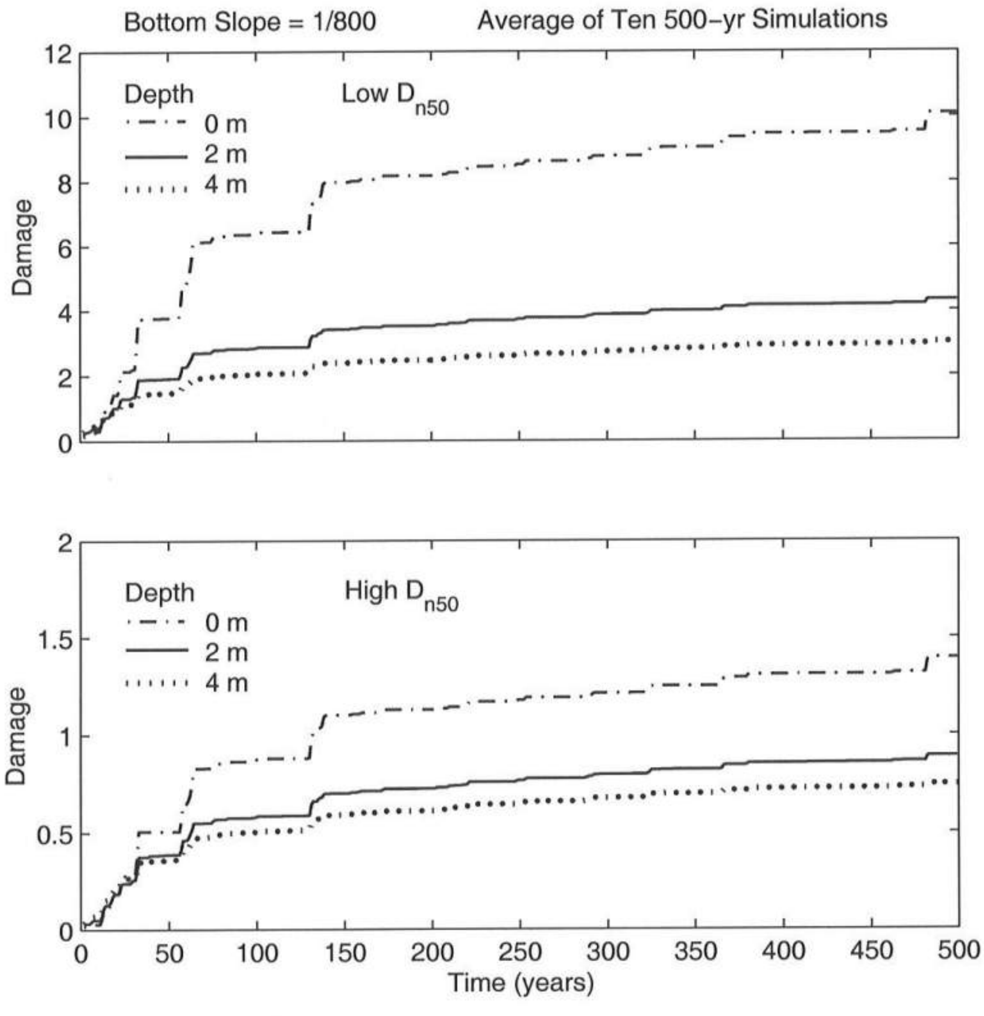

Figure 5.6: Damage Progression of Armor Layer with Low and High Diameters D_{n50} on 1/800 Slope at Locations of $d = 4$, 2 and 0 m Averaged for Ten 500-yr Simulations.

5.5 Computed Results for Bottom Slope of 1/40

5.5.1 Estimated Armor Stone Mass M_{50}

The estimated values of H_{10} and $H_{2\%}$ for the 100-yr storm at the locations of $d = 4$, 2 and 0 m on the 1/40 slope are shown in Table 5.5 in the same manner as in Table 5.1 for the 1/800 slope. The ratios between H_{10} and H_{mo} and between $H_{2\%}$ and H_{mo} are shown to be in the narrow range of 1.22–1.30 and 1.29–1.37, respectively.

Table 5.5: Estimated 100-yr Wave Heights H_{10} and $H_{2\%}$ for 1/40 Bottom Slope.

Water depth below MSL, $d(m)$	4.0	2.0	0.0
Still water depth, $d_s(m)$	6.61	4.61	2.61
Significant wave height, $H_{mo}(m)$	5.05	3.93	2.75
Average height of highest 10%, $H_{10}(m)$	6.17	4.89	3.57
H_{10}/H_{mo}	1.22	1.24	1.30
2% exceedance wave height, $H_{2\%}(m)$	6.52	5.17	3.77
$H_{2\%}/H_{mo}$	1.29	1.32	1.37

Table 5.6 lists the computed results of the stability number, the nominal diameter D_{n50} and the corresponding stone mass based on the Hudson formula for the 1/40 slope. Also, the computed results based on Van der Meer formula are shown in Table 5.7. Once again, the comparison between these two tables indicates the high value of D_{n50} for the Hudson formula in Table 5.6 relative to the low value of D_{n50} for the Van der Meer formula in Table 5.7.

Tables 5.8 and 5.9 are presented here to provide an easy comparison of the values based on the Hudson and Van der Meer formulas for the 1/800 and 1/40 slopes.

Table 5.6: Estimated Median Stone Mass Using Hudson Formula for 1/40 Bottom Slope.

Water depth below MSL, $d(m)$	4.0	2.0	0.0
Stability number, $H_{10}/\Delta D_{n50}$	1.59	1.59	1.59
Nominal diameter, $D_{n50}(m)$	2.43	1.92	1.40
Median stone mass, $M_{50}(ton)$	38.2	18.8	7.30

Table 5.7: Estimated Median Stone Mass Using Van der Meer Formula for 1/40 Bottom Slope.

Water depth below MSL, $d(m)$	4.0	2.0	0.0
Surf similarity parameter, ξ_m	2.84	3.22	3.85
Stability number, $H_{2\%}/\Delta D_{n50}$	2.52	2.37	2.20
Nominal diameter, $D_{n50}(m)$	1.62	1.36	1.07
Median stone mass, $M_{50}(ton)$	11.3	6.69	3.26

Table 5.8: Armor Stone Nominal Diameter D_{n50} Designed for 100-yr Storm Using Hudson Formula.

Bottom Slope	1/800			1/40		
Depth d(m) below MSL	4.0	2.0	0.0	4.0	2.0	0.0
Wave Height H_{10}(m)	3.54	2.64	1.70	6.17	4.89	3.57
Diameter D_{n50}(m)	1.39	1.04	0.67	2.43	1.92	1.40
Stability Number N_{mo}	1.37	1.37	1.36	1.30	1.28	1.23

Table 5.9: Armor Stone Nominal Diameter D_{n50} Designed for 100-yr Storm Using Van der Meer Formula.

Bottom Slope	1/800			1/40		
Depth d (m) below MSL	4.0	2.0	0.0	4.0	2.0	0.0
Wave Height $H_{2\%}$(m)	3.74	2.78	1.80	6.52	5.17	3.77
Diameter D_{n50}(m)	1.05	0.76	0.45	1.62	1.36	1.07
Stability Number N_{mo}	1.81	1.88	2.03	1.95	1.80	1.61

5.5.2 Damage Progression Caused by Sequences of Storms

The formula proposed by Melby and Kobayashi (2000) is compared with the Hudson and Van der Meer formulas for the peak of the design storm on the 1/40 slope. Equation 5.10 is used to estimate the value of S_{n+1} in the same manner as in Table 5.4. The calculated damage shown in Table 5.10 is in the range of 0.17–0.23 for the Hudson formula and in the range of 0.67–1.74 for the Van der Meer formula.

Table 5.10: Damage S to Armor Layer Designed by Hudson and Van der Meer Formulas Caused by 100-yr Wave Height and 1,000 Waves for 1/40 Bottom Slope.

		Hudson formula			**Van der Meer formula**		
d (m)	H_{mo} (m)	D_{n50} (m)	N_{mo}	S	D_{n50} (m)	N_{mo}	S
4	5.05	2.43	1.30	0.23	1.62	1.95	1.74
2	3.93	1.92	1.28	0.21	1.36	1.80	1.17
0	2.75	1.40	1.23	0.17	1.07	1.61	0.67

Examples of the typical results for the 1/40 bottom slope are presented in the same manner as for the 1/800 slope. All the figures are presented in Section 5.7 for completeness.

Fig. 5.7 shows the damage S_j at the end of the j-th storm and the maximum stability number $(N_{mo})_j$ for the low and high values of D_{n50} based on the Van der Meer and Hudson formulas, respectively, for one 500-yr simulation. The example shown in Fig. 5.7 is similar to that shown in Fig. 5.3. The progression of damage to the armor layer is episodic in both figures.

Fig. 5.8 shows the damage progression of the armor layer with the low and high D_{n50} at $d = 2$ m on the 1/40 slope for each of the ten 500-yr simulations in the same manner as Fig. 5.4. The statistical variability about the average damage progression remains to be a factor of about two and small due to the aging effect of the damaged armor layer.

Fig. 5.9 shows the equivalent number N_j of waves to cause the same damage increment using the maximum stability number $(N_{mo})_j$ for the j-th storm at the location of $d = 2$ m on the 1/40 slope in the same manner as Fig. 5.5. The average value of N_j for the ten 500-yr simulations shown in Fig. 5.5 is 993 and 725 for the low and high D_{n50}, respectively. As a whole, the computed equivalent number of waves is of the order of 1,000 for the depth $d = 4$, 2 and 0 m and the 1/800 and 1/40 slopes. Consequently, the use of 1,000 waves during the peak of a hurricane storm is reasonable if the time series of H_{mo} during the entire storm is not available.

Finally, Fig. 5.10 compares the averaged damage progression for the armor layer with the low and high D_{n50} at the depth $d = 4$, 2 and 0 m on the 1/40 slope. The computed damage progression for the 1/40 slope is similar to that shown in Fig. 5.6 except that the damage for $d = 0$ m is about 40% of that shown in Fig. 5.6 for the 1/800 slope. Considering that local failure of the armor layer may occur if the damage exceeds about eight, all the values of the low D_{n50} in Table 5.7 are sufficient even if the damage variability is accounted for. On the other hand, the values of the high D_{n50} in Table 5.6 are conservative and can be reduced if such large stones are not available.

The virtual performance of the armor layer presented here is a mere example but gives a better insight into what may happen to the armor layer under a long-term sequence of hurricane storms.

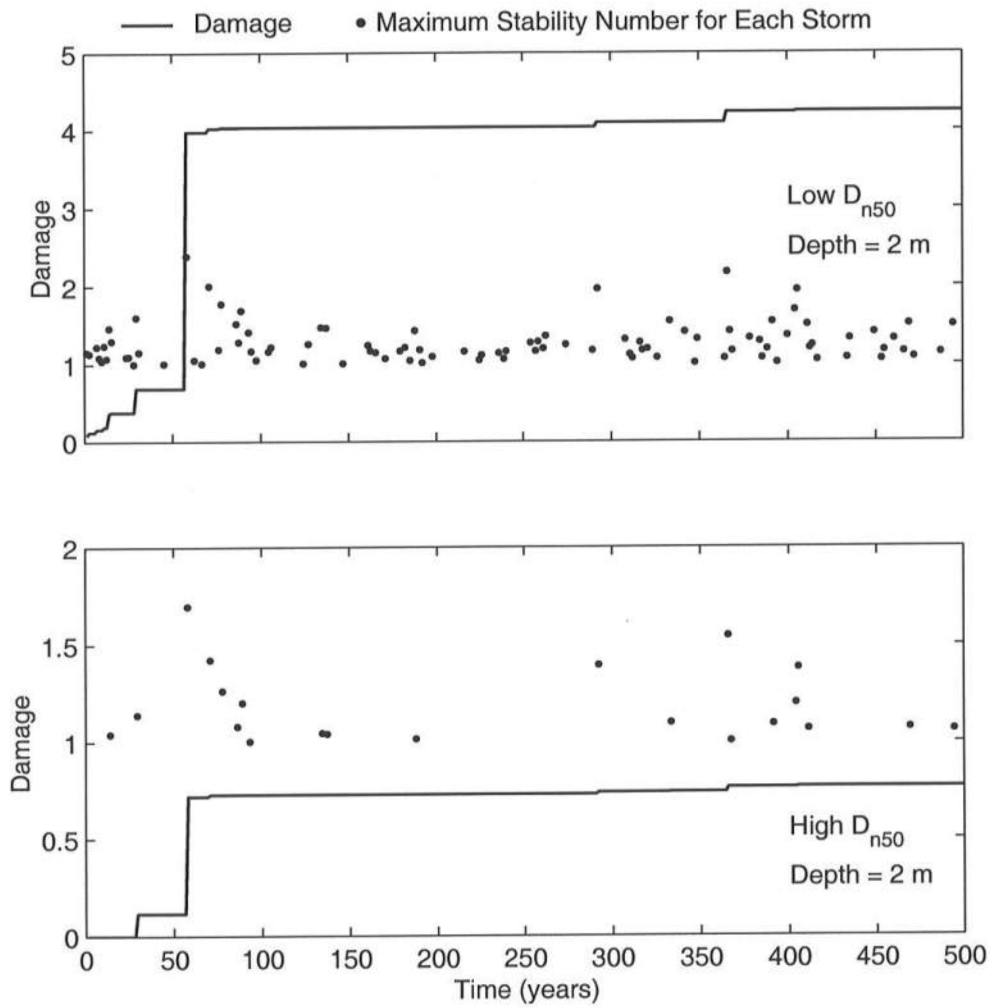

Figure 5.7: Damage Progression of Armor Layer with Low and High Diameters D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/40 Slope at Location of $d = 2$ m for One 500-yr Simulation.

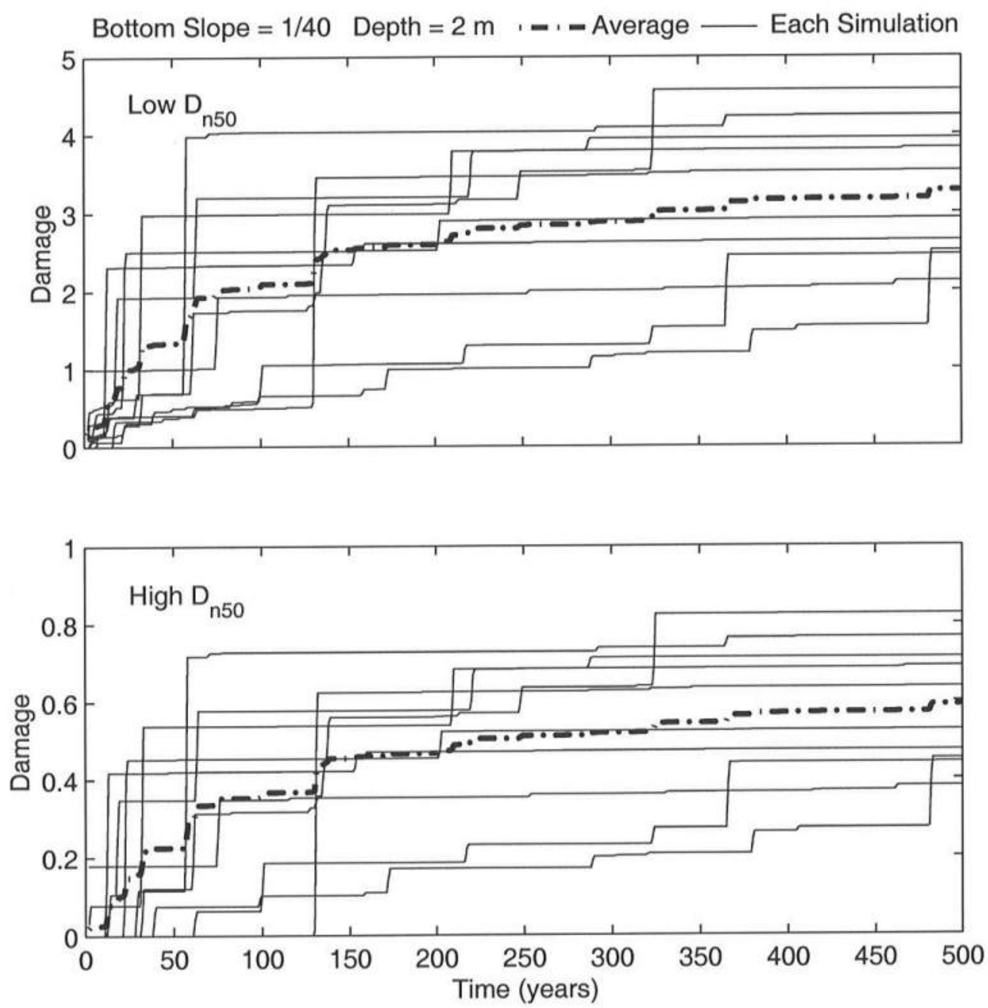

Figure 5.8: Damage Progression of Armor Layer with Low and High Diameters D_{n50} on 1/40 Slope at Location of d = 2 m for Each and Average of Ten 500-yr Simulations.

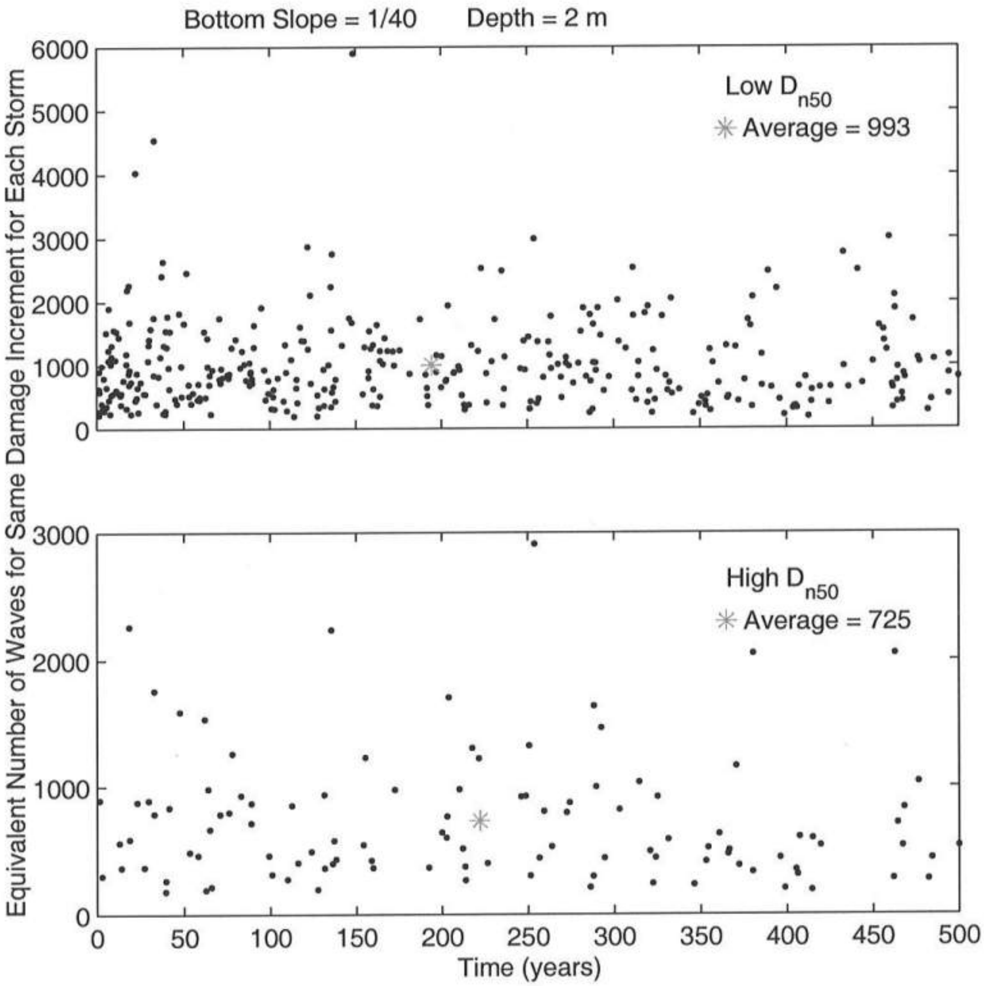

Figure 5.9: Equivalent Number of Waves to Cause the Same Damage Increment for Low and High Diameters D_{n50} on $1/40$ Slope at Location of $d = 2$ m for Ten 500-yr Simulations.

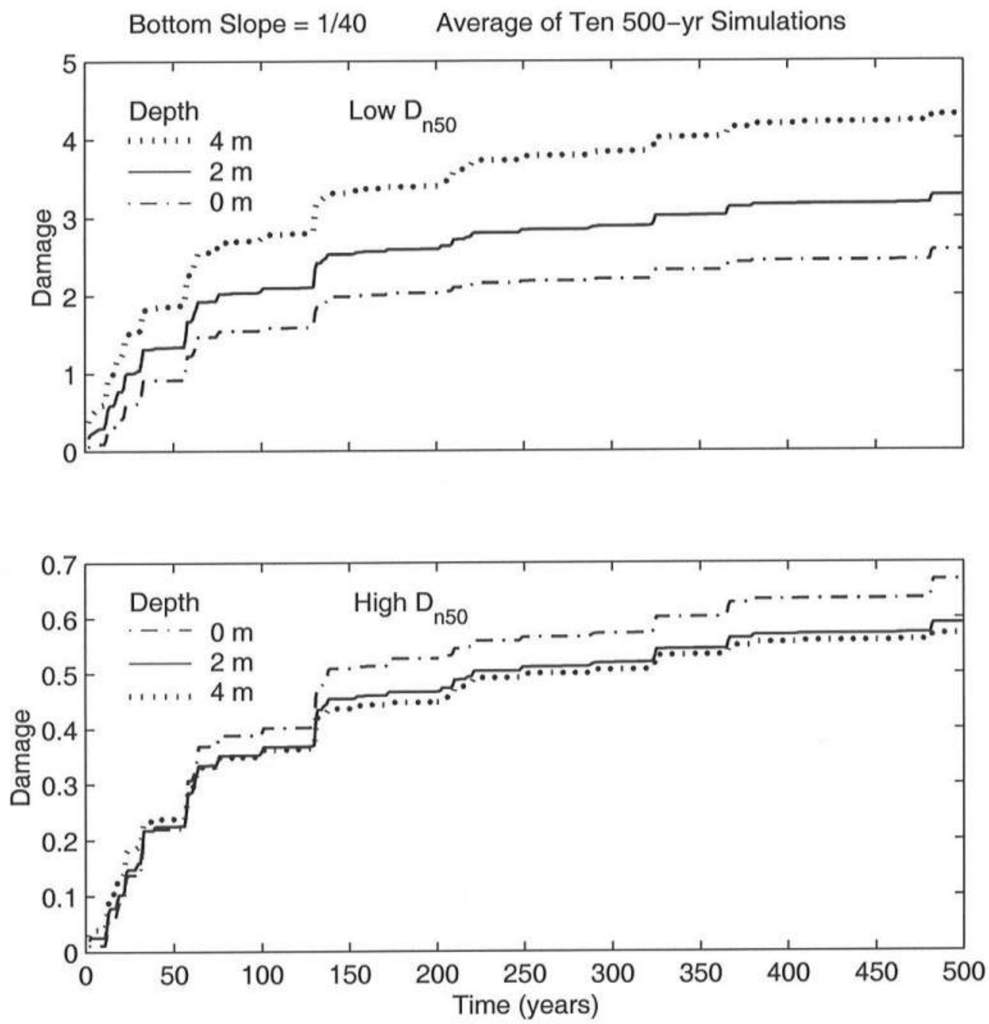

Figure 5.10: Damage Progression of Armor Layer with Low and High Diameters D_{n50} on 1/40 Slope at Locations of $d = 4$, 2 and 0 m Averaged for Ten 500-yr Simulations.

5.6 Individual Figures for Bottom Slope of $1/800$

For each of the ten 500-yr simulations (indicated by the integer $N500 = $ 1-10), three figures related to the damage progression are presented first. The summary of the ten 500-yr simulations at the depth $d = 4$, 2 and 0 m is then presented using six figures.

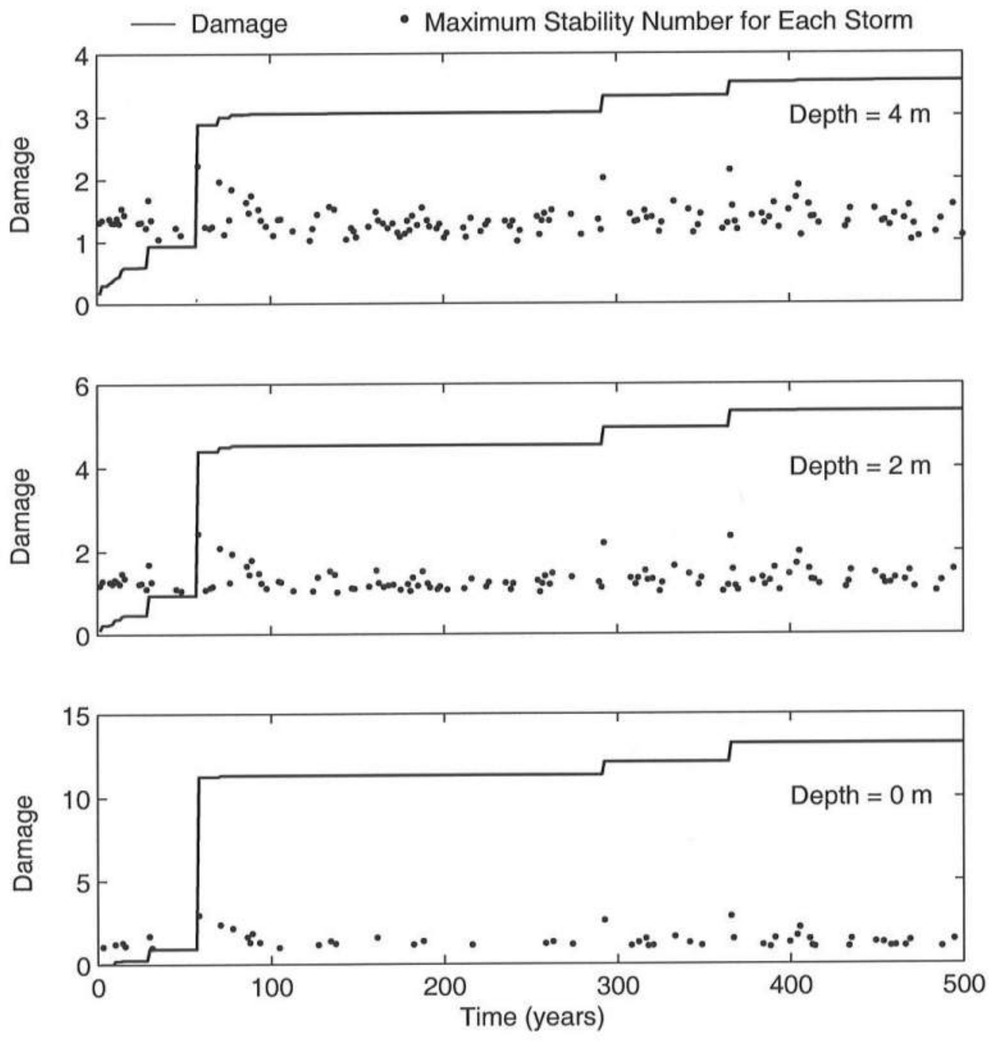

Figure 5.11: Damage Progression of Armor Layer with Low Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/800 Slope at Locations of $d = 4$, 2 and 0 m for $N500 = 1$.

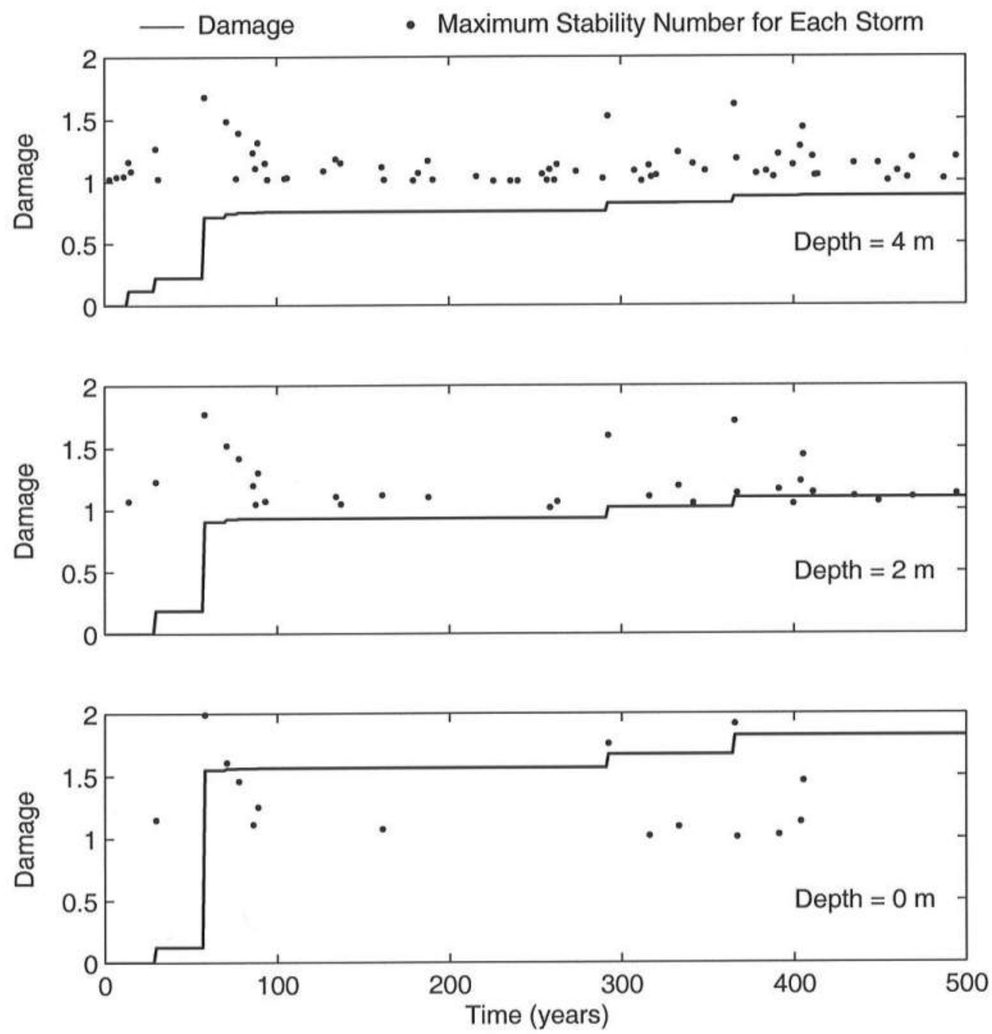

Figure 5.12: Damage Progression of Armor Layer with High Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/800 Slope at Locations of $d = 4$, 2 and 0 m for $N500 = 1$.

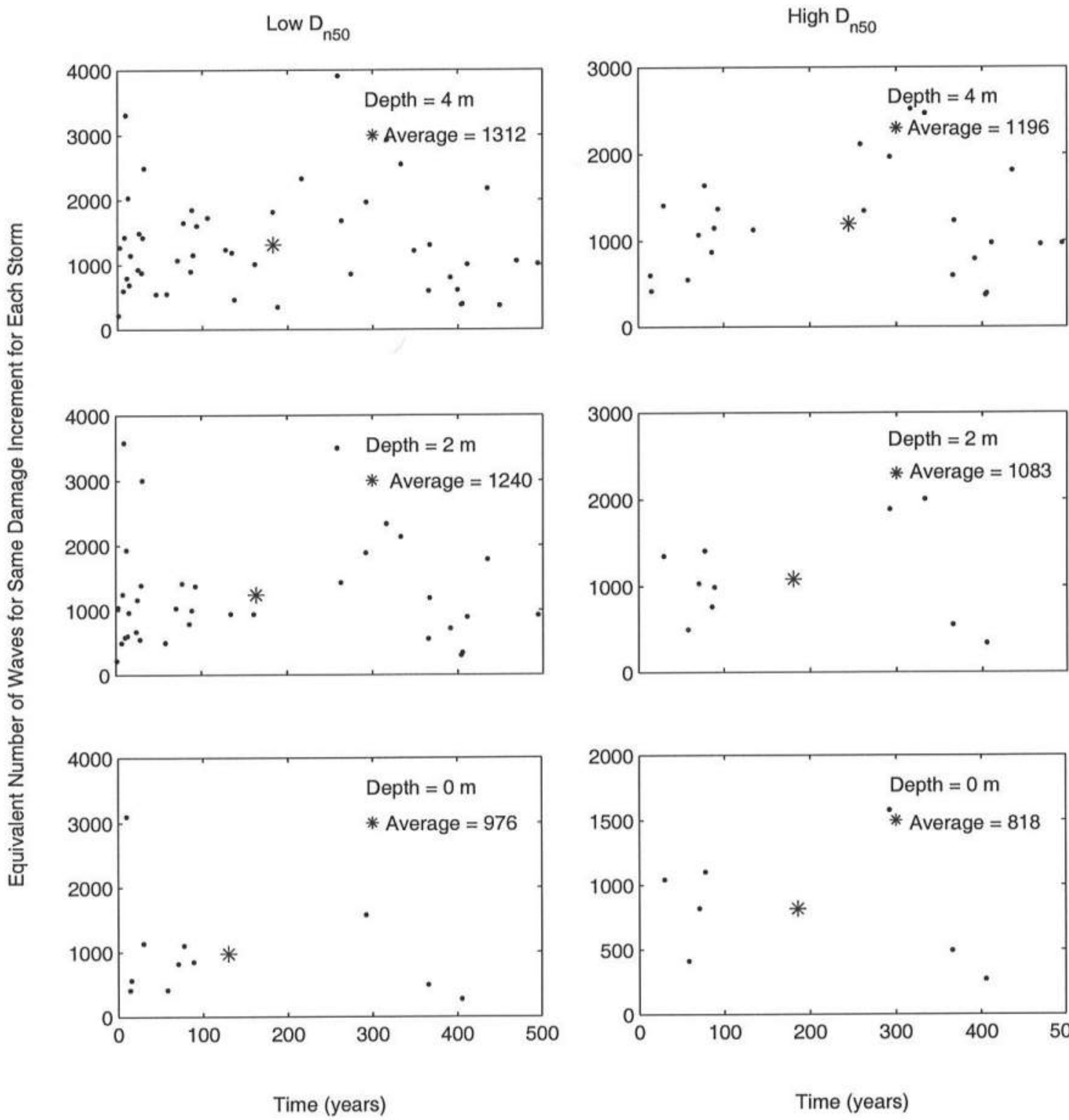

Figure 5.13: Equivalent Number of Waves to Cause the Same Damage Increment for Low and High Diameters D_{n50} on 1/800 Slope at Locations of d = 4, 2 and 0 m for $N500 = 1$.

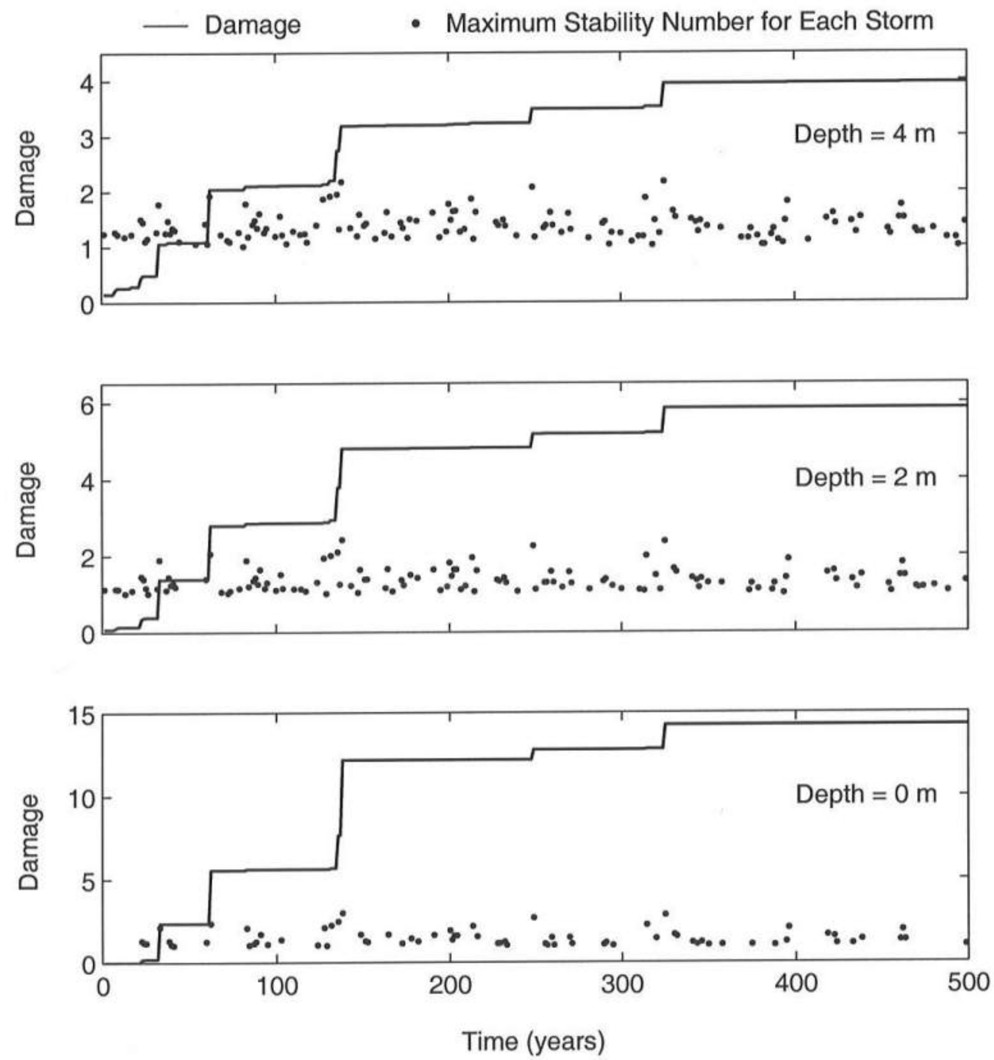

Figure 5.14: Damage Progression of Armor Layer with Low Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/800 Slope at Locations of d = 4, 2 and 0 m for $N500 = 2$.

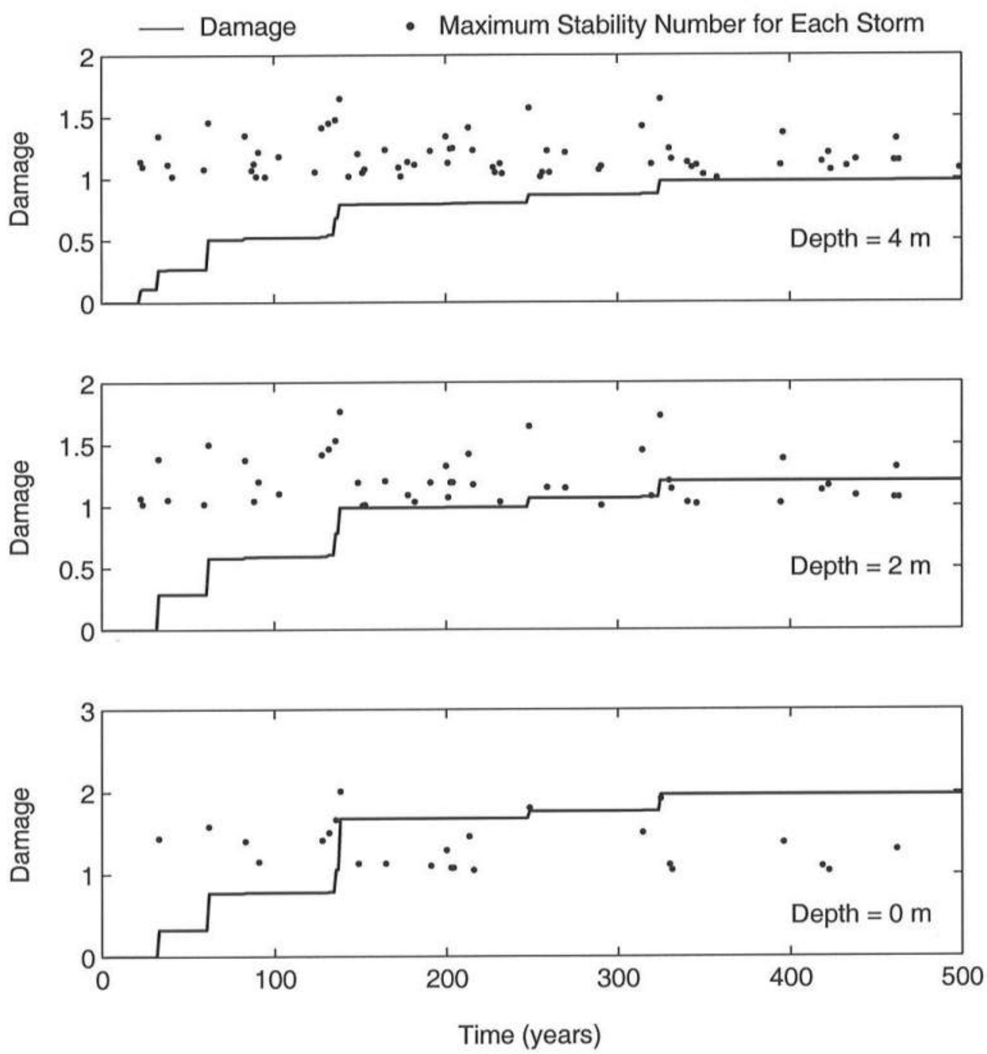

Figure 5.15: Damage Progression of Armor Layer with High Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/800 Slope at Locations of $d = 4$, 2 and 0 m for $N500 = 2$.

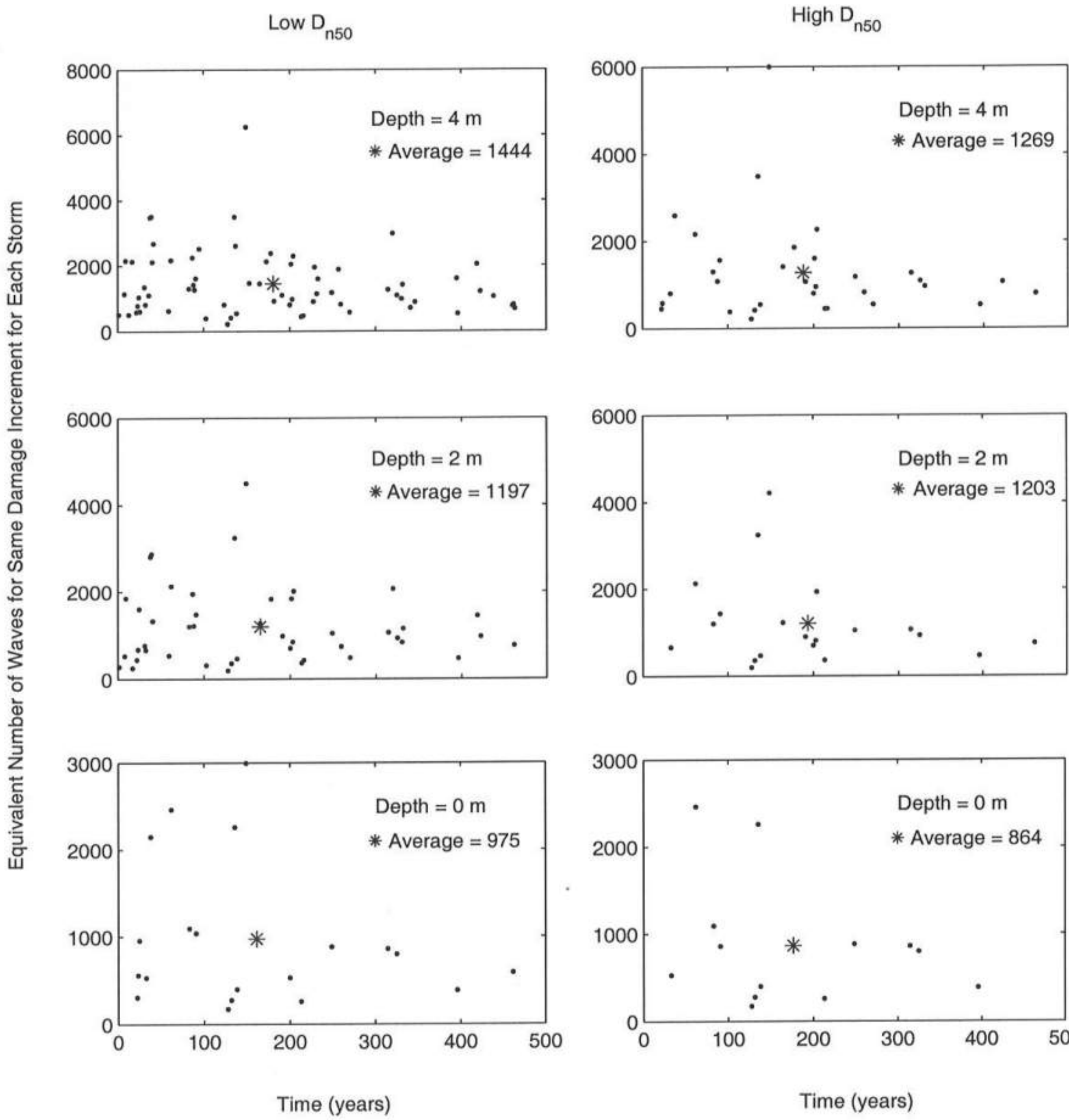

Figure 5.16: Equivalent Number of Waves to Cause the Same Damage Increment for Low and High Diameters D_{n50} on 1/800 Slope at Locations of d = 4, 2 and 0 m for $N500 = 2$.

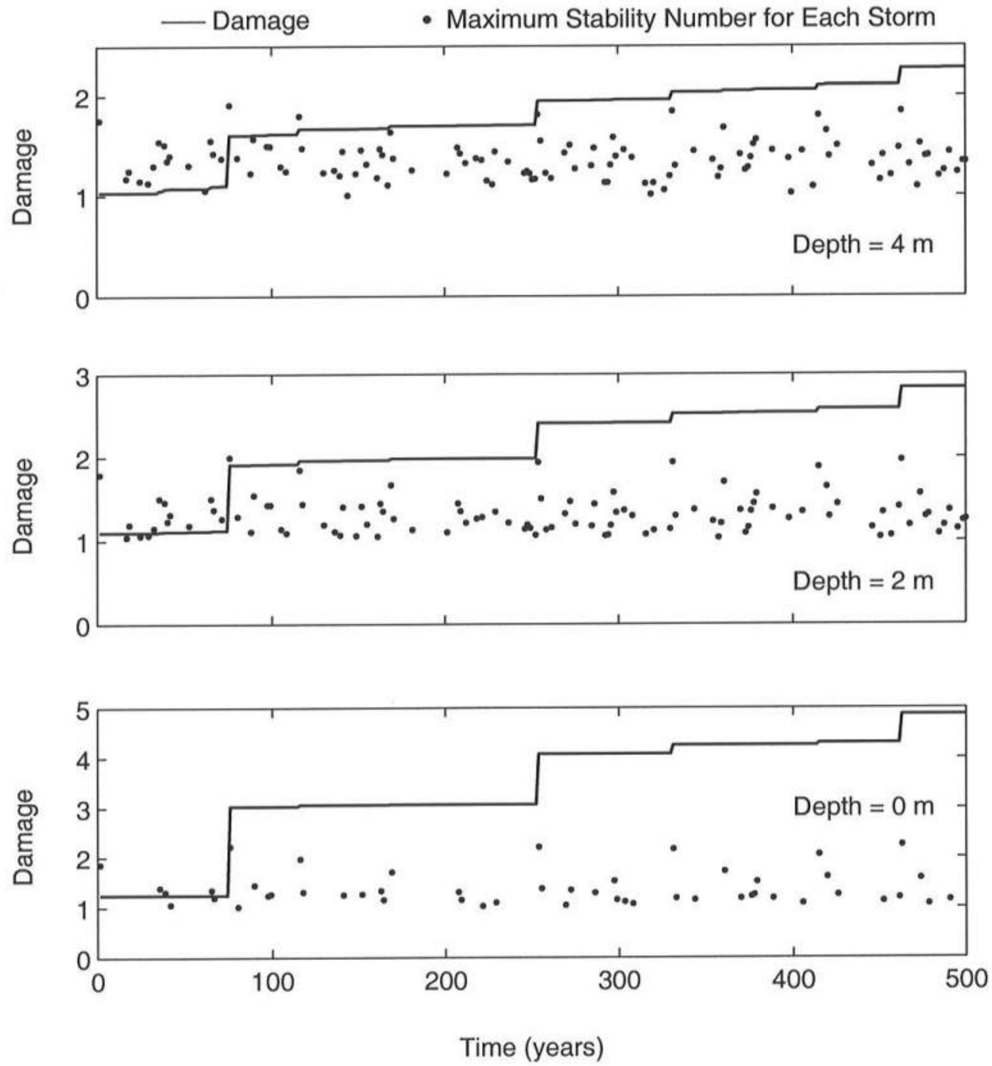

Figure 5.17: Damage Progression of Armor Layer with Low Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/800 Slope at Locations of $d = 4$, 2 and 0 m for $N500 = 3$.

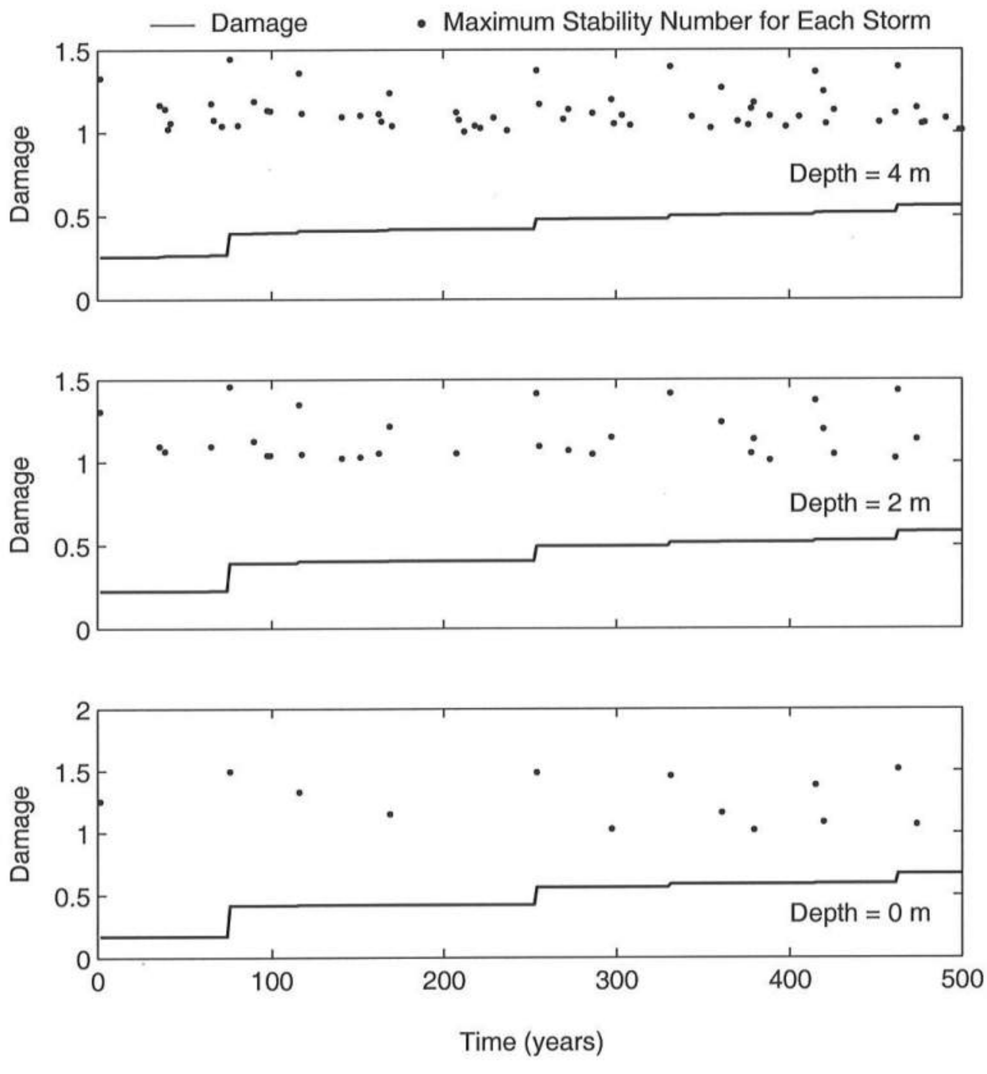

Figure 5.18: Damage Progression of Armor Layer with High Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/800 Slope at Locations of $d = 4$, 2 and 0 m for $N500 = 3$.

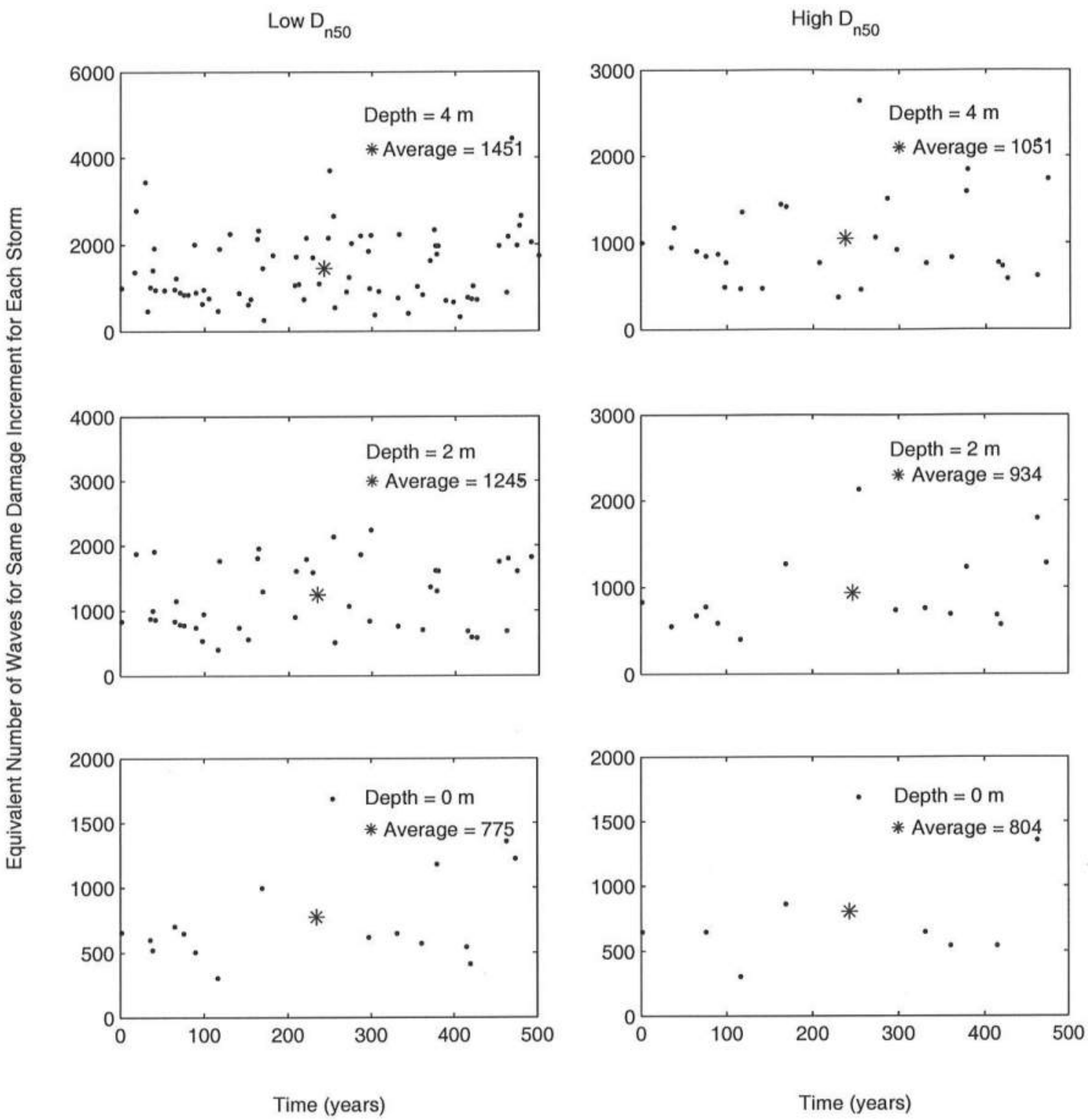

Figure 5.19: Equivalent Number of Waves to Cause the Same Damage Increment for Low and High Diameters D_{n50} on 1/800 Slope at Locations of d = 4, 2 and 0 m for $N500 = 3$.

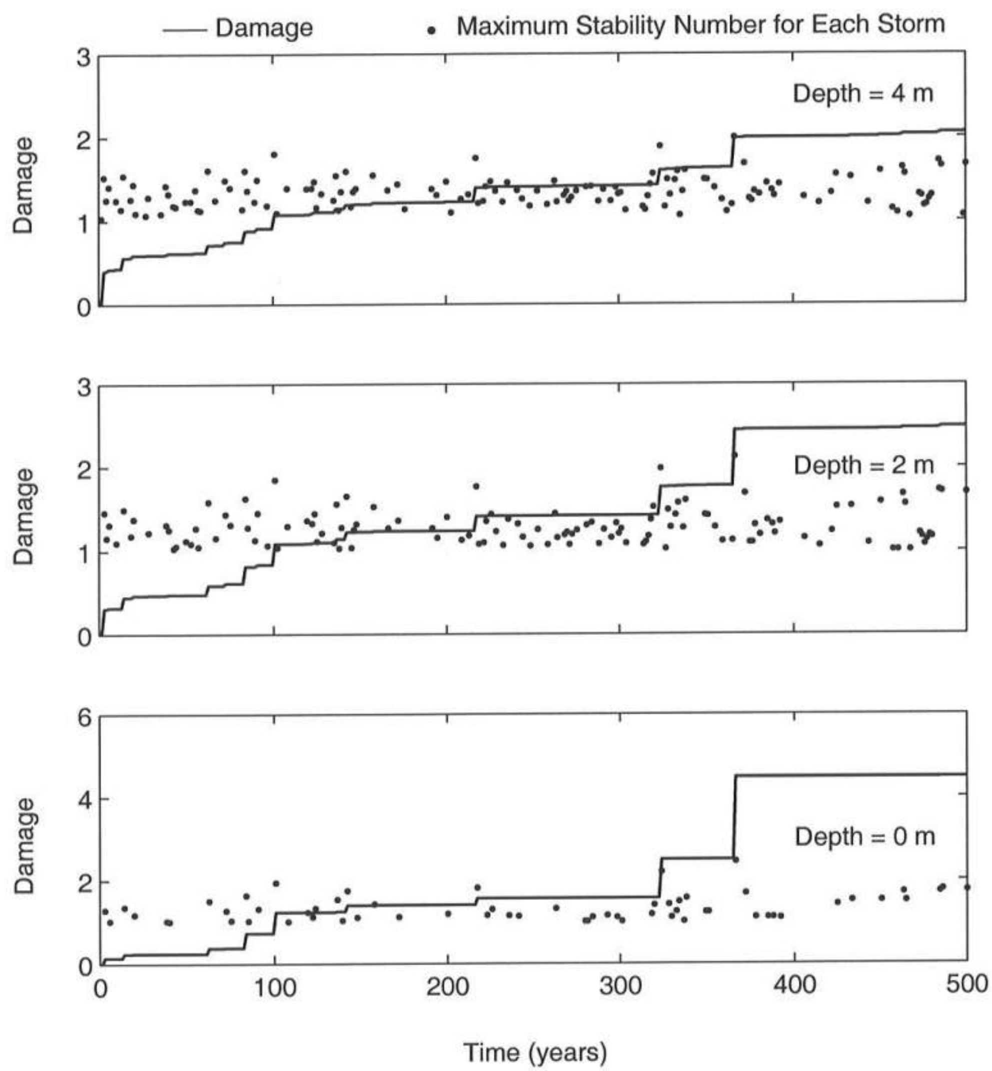

Figure 5.20: Damage Progression of Armor Layer with Low Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/800 Slope at Locations of $d = 4$, 2 and 0 m for $N500 = 4$.

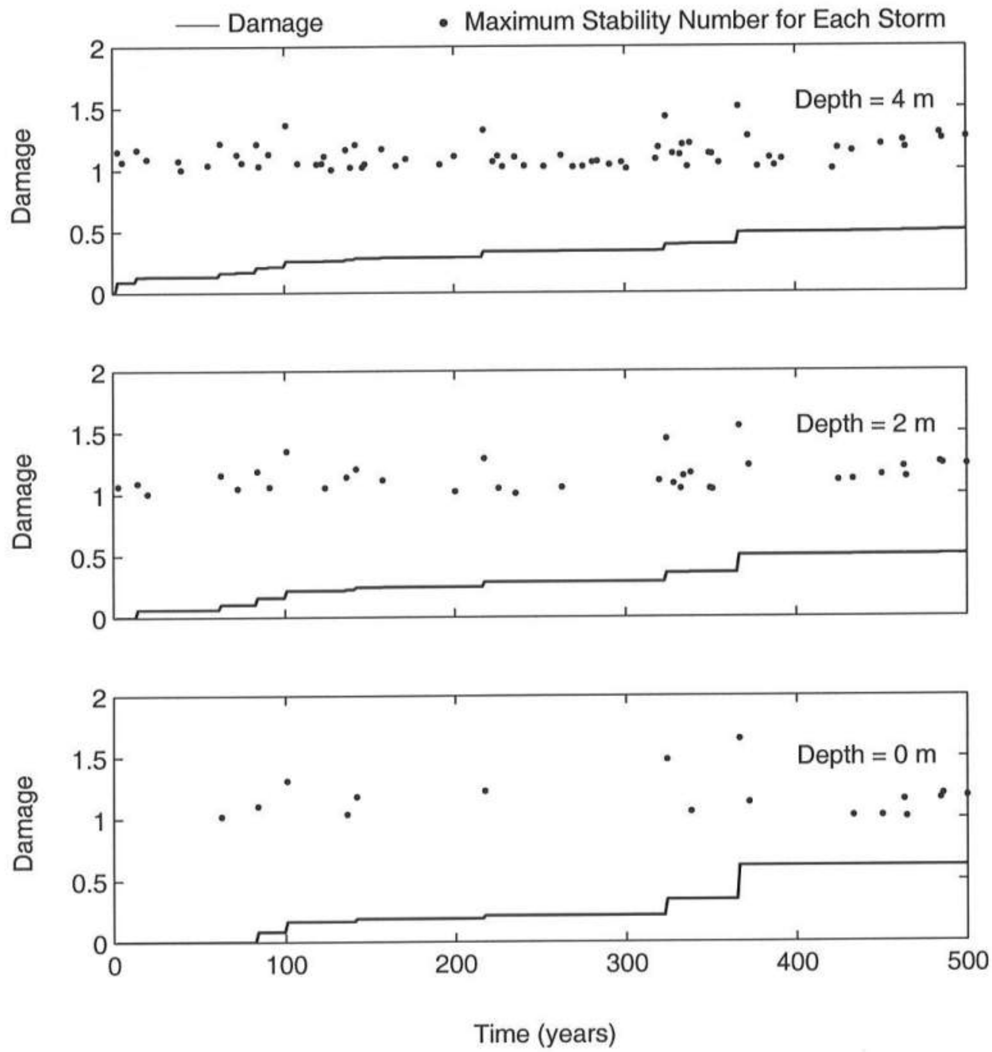

Figure 5.21: Damage Progression of Armor Layer with High Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/800 Slope at Locations of d = 4, 2 and 0 m for $N500$ = 4.

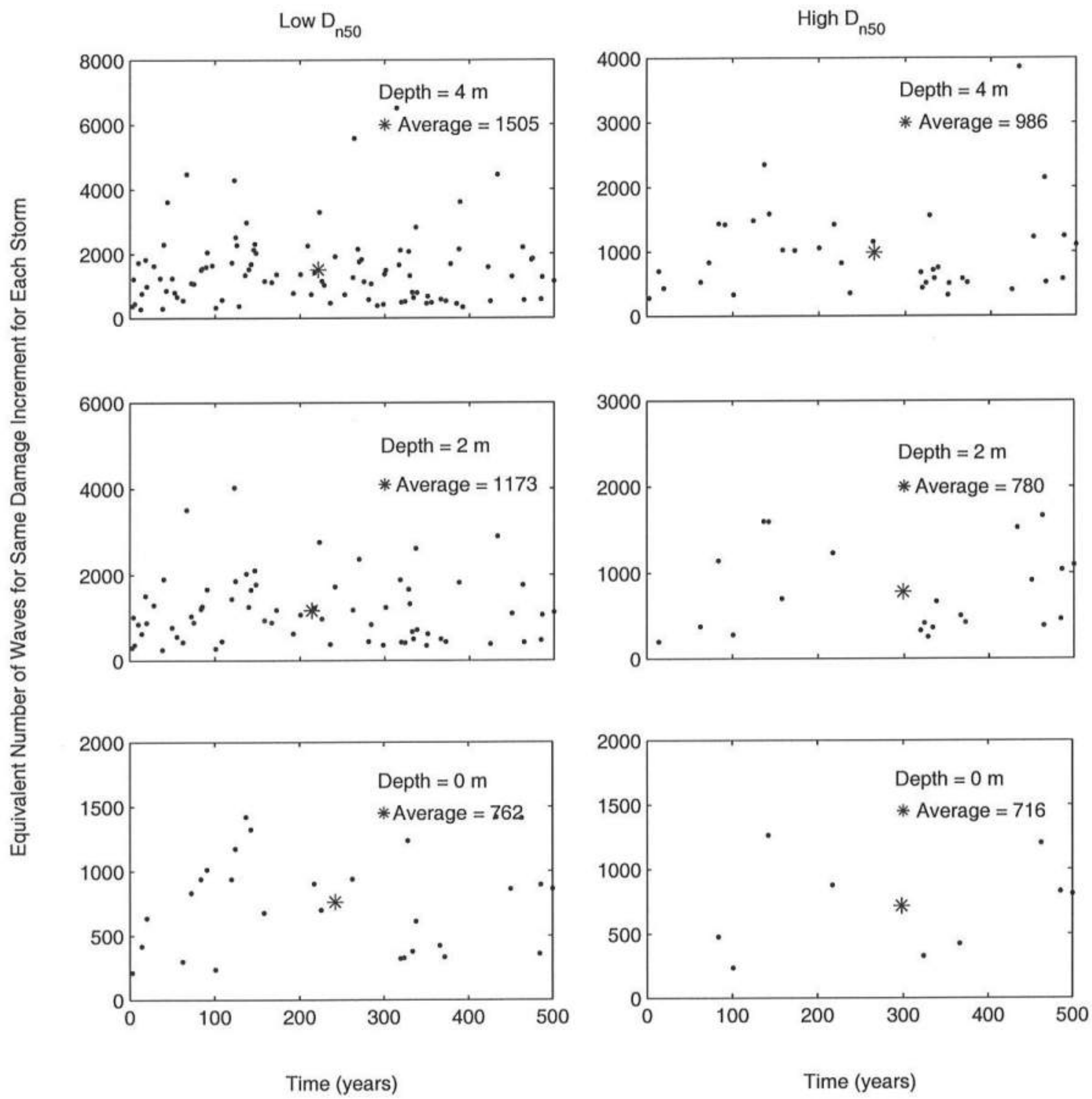

Figure 5.22: Equivalent Number of Waves to Cause the Same Damage Increment for Low and High Diameters D_{n50} on 1/800 Slope at Locations of d = 4, 2 and 0 m for $N500$ = 4.

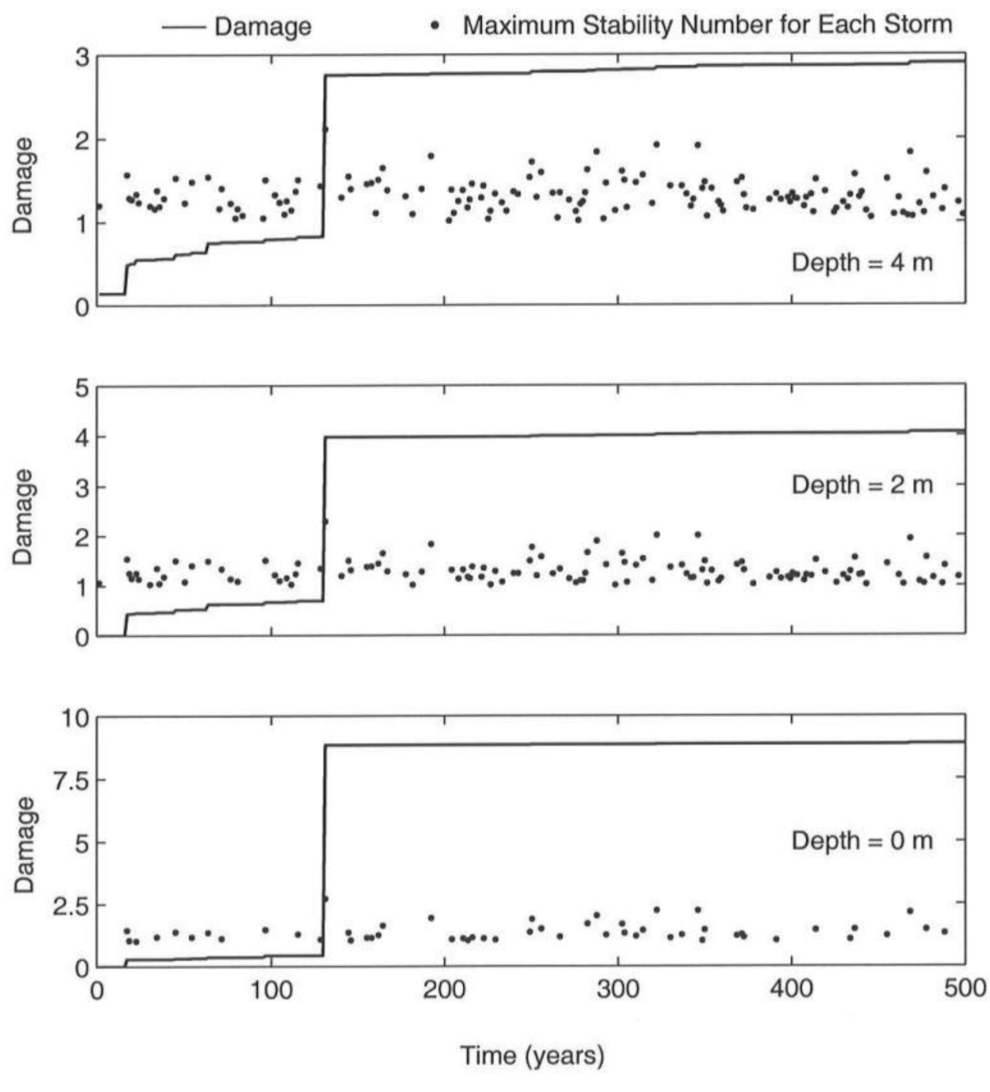

Figure 5.23: Damage Progression of Armor Layer with Low Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/800 Slope at Locations of $d = 4$, 2 and 0 m for $N500 = 5$.

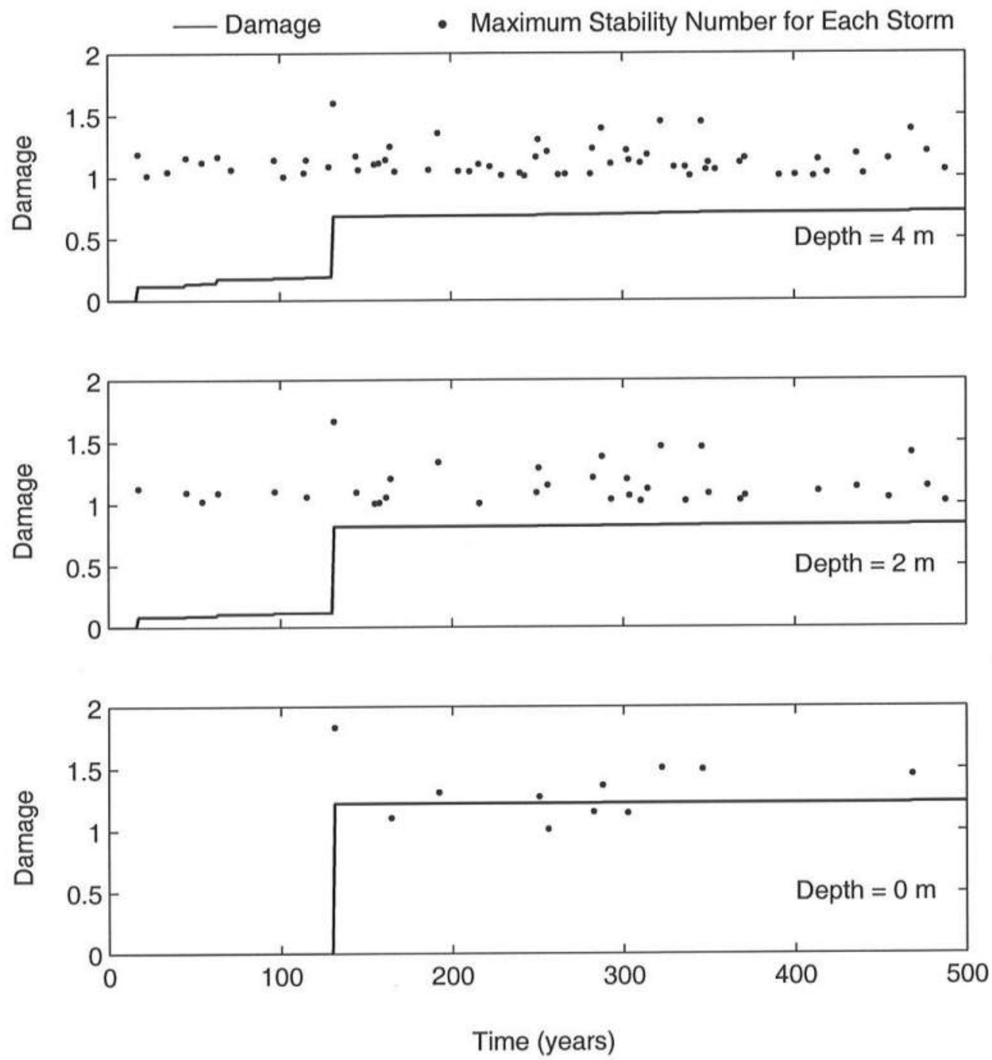

Figure 5.24: Damage Progression of Armor Layer with High Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/800 Slope at Locations of $d = 4$, 2 and 0 m for $N500 = 5$.

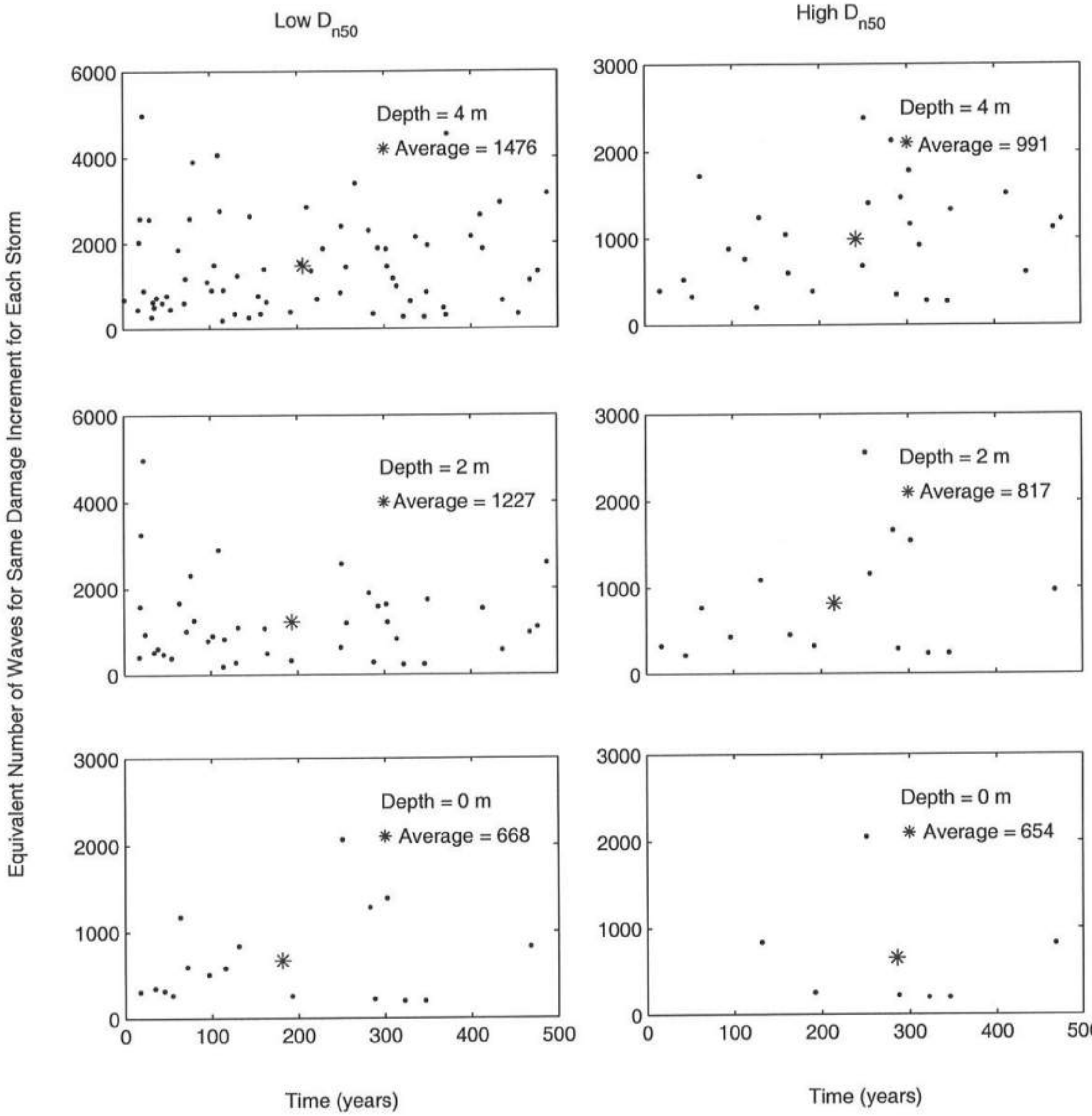

Figure 5.25: Equivalent Number of Waves to Cause the Same Damage Increment for Low and High Diameters D_{n50} on 1/800 Slope at Locations of d = 4, 2 and 0 m for $N500 = 5$.

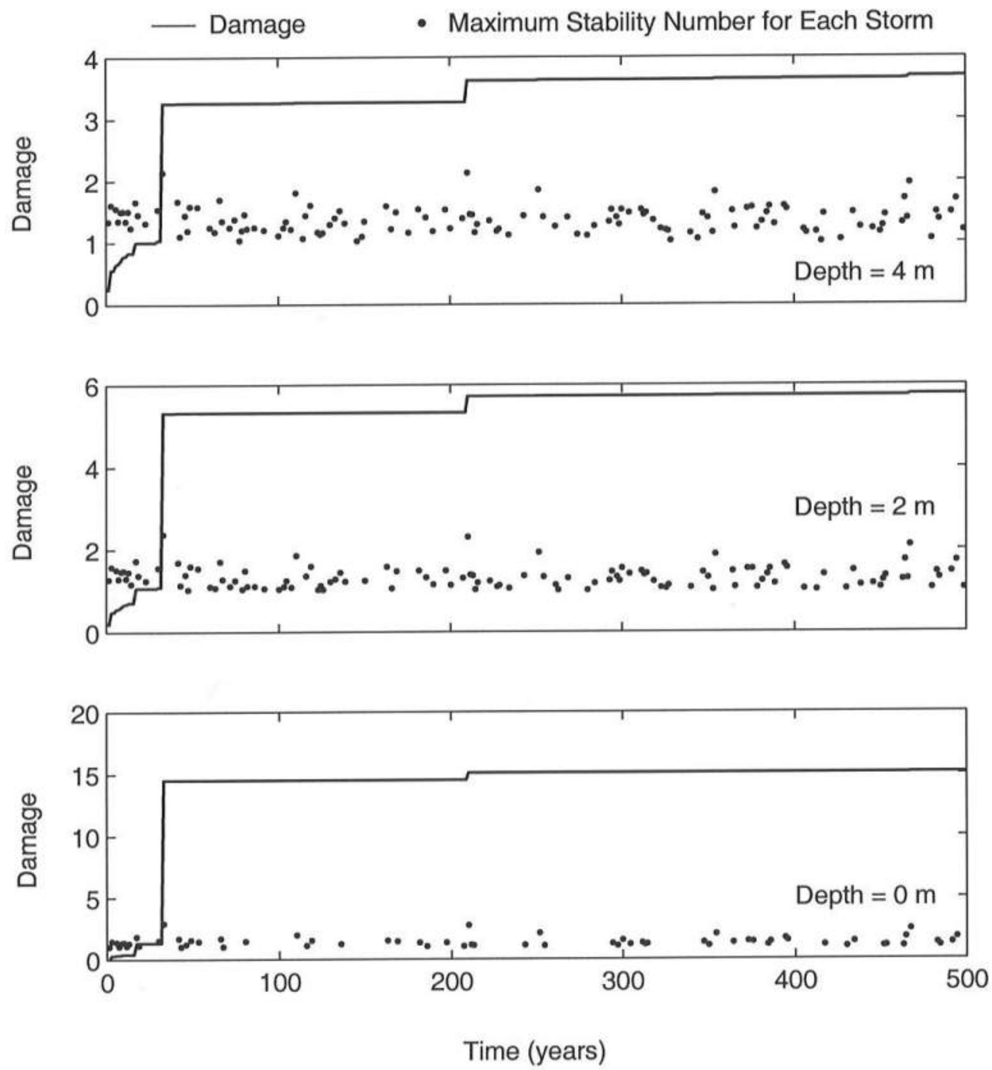

Figure 5.26: Damage Progression of Armor Layer with Low Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/800 Slope at Locations of $d = 4$, 2 and 0 m for $N500 = 6$.

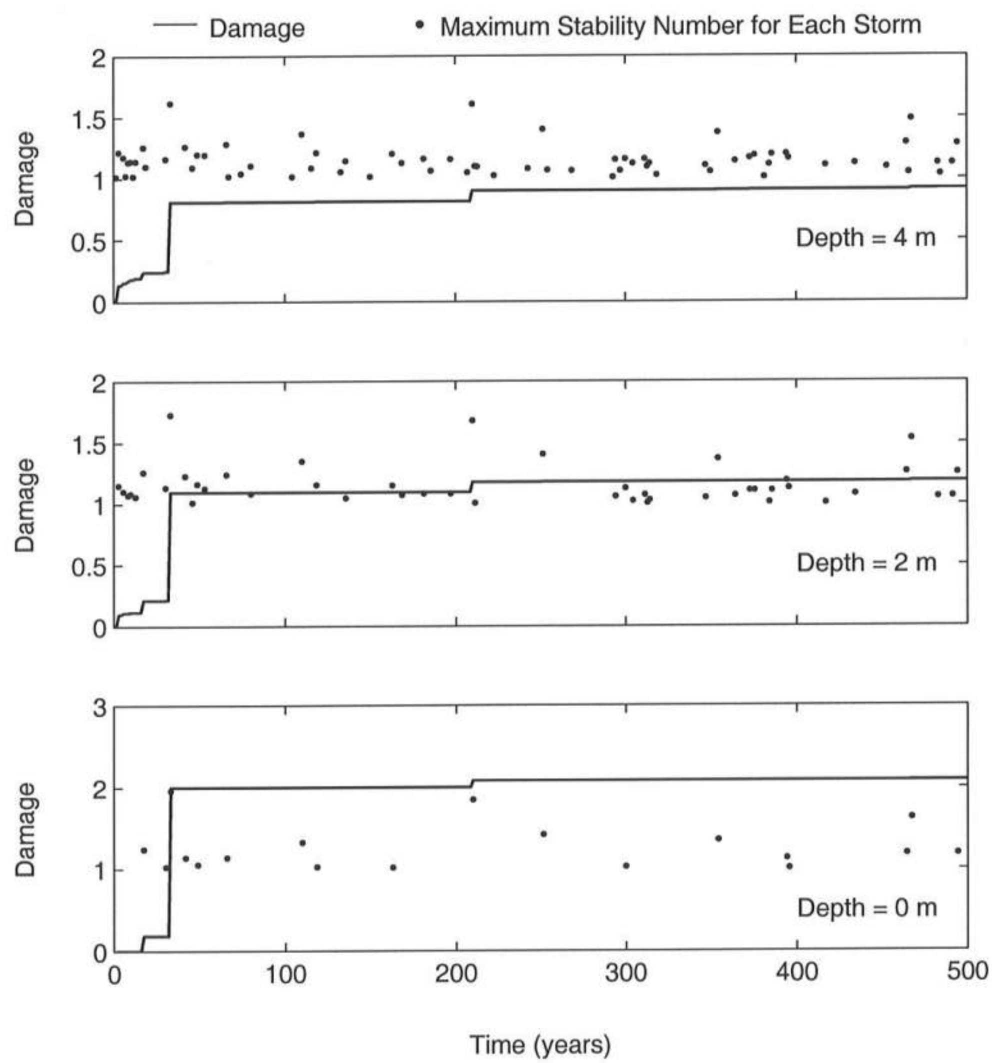

Figure 5.27: Damage Progression of Armor Layer with High Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/800 Slope at Locations of d = 4, 2 and 0 m for $N500$ = 6.

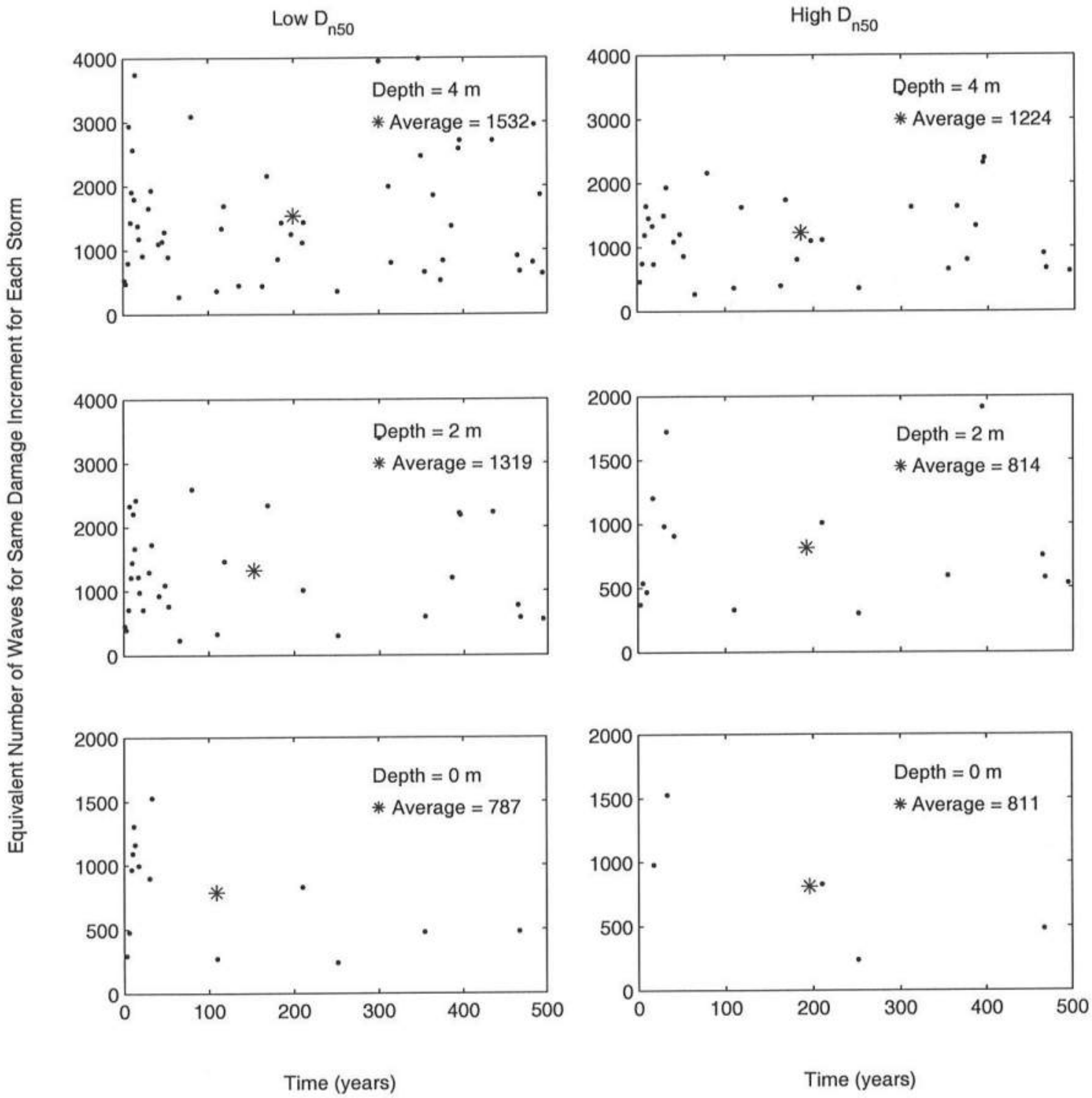

Figure 5.28: Equivalent Number of Waves to Cause the Same Damage Increment for Low and High Diameters D_{n50} on 1/800 Slope at Locations of d = 4, 2 and 0 m for $N500 = 6$.

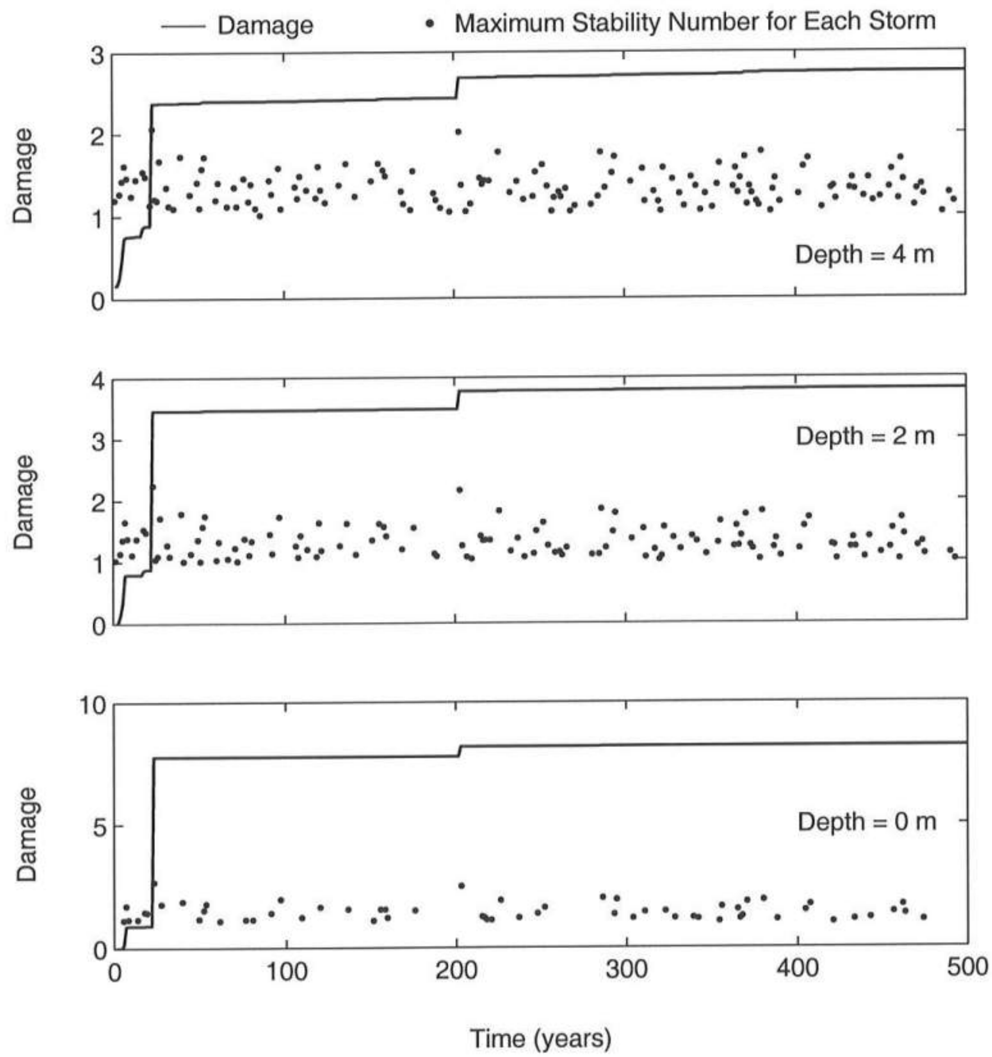

Figure 5.29: Damage Progression of Armor Layer with Low Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/800 Slope at Locations of $d = 4$, 2 and 0 m for $N500 = 7$.

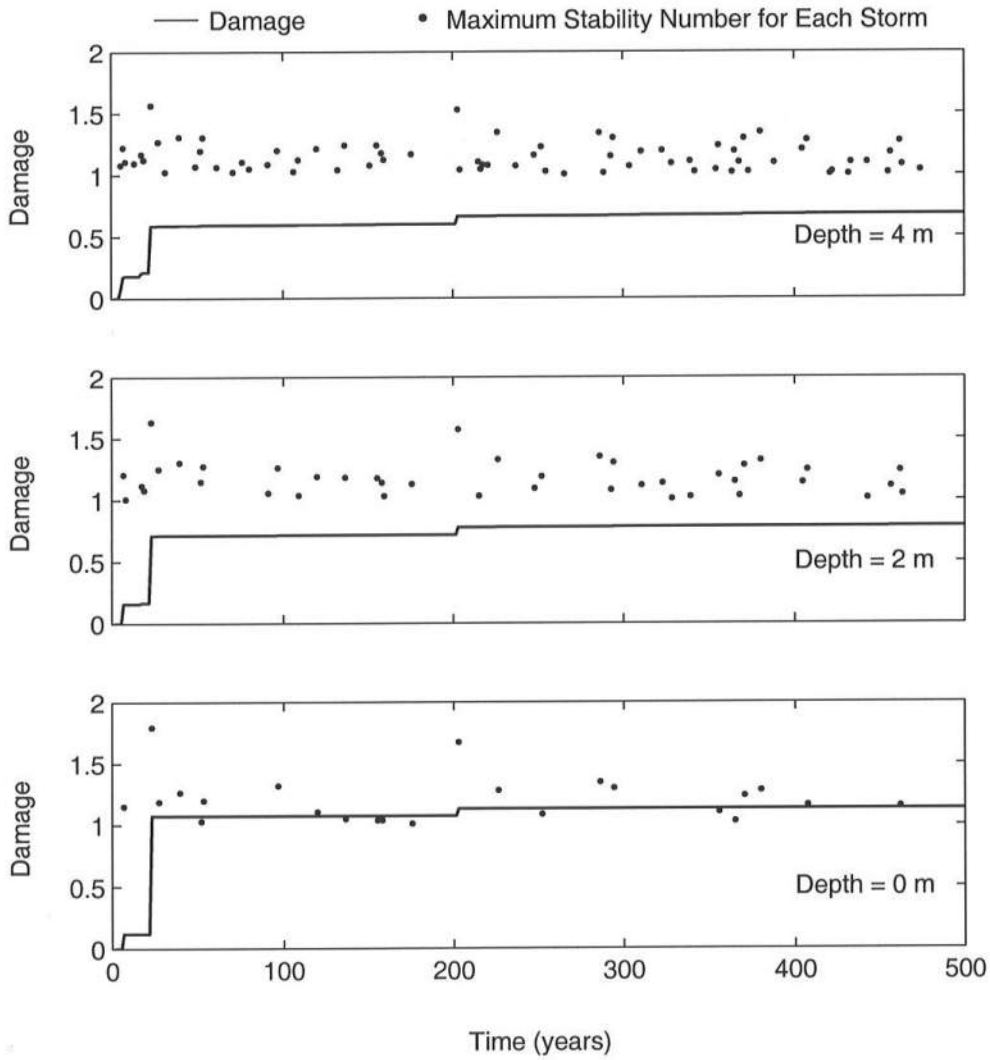

Figure 5.30: Damage Progression of Armor Layer with High Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/800 Slope at Locations of d = 4, 2 and 0 m for $N500$ = 7.

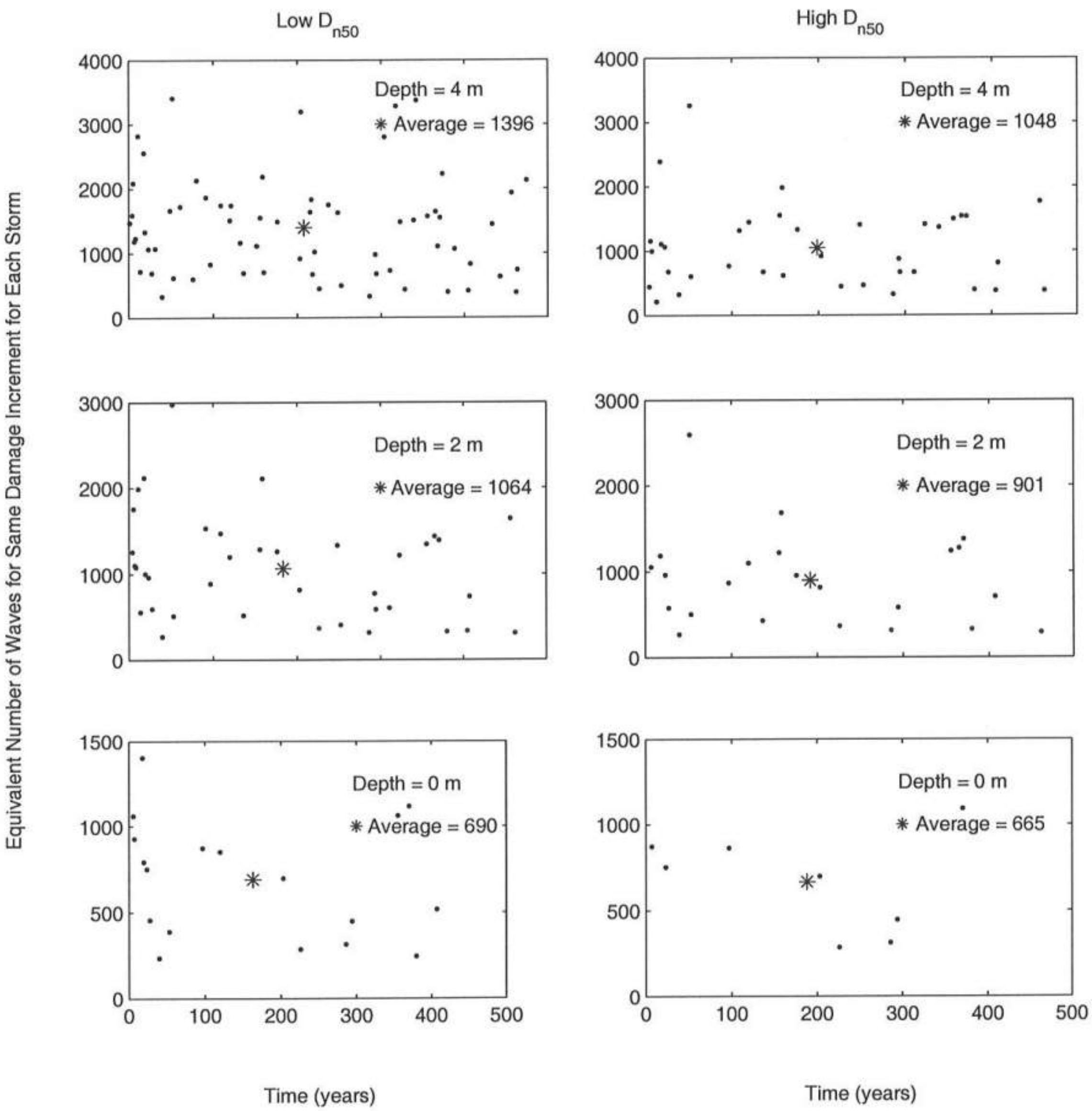

Figure 5.31: Equivalent Number of Waves to Cause the Same Damage Increment for Low and High Diameters D_{n50} on 1/800 Slope at Locations of d = 4, 2 and 0 m for $N500 = 7$.

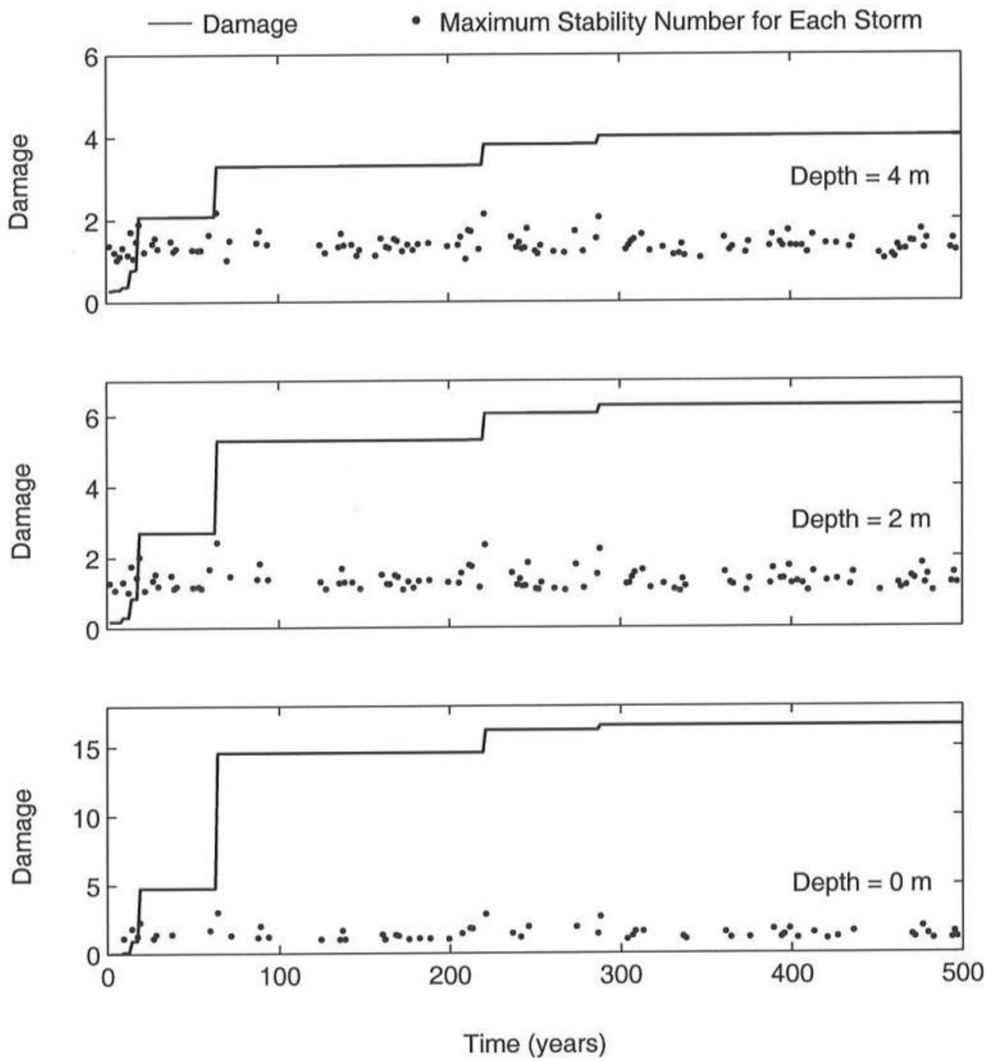

Figure 5.32: Damage Progression of Armor Layer with Low Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/800 Slope at Locations of d = 4, 2 and 0 m for $N500 = 8$.

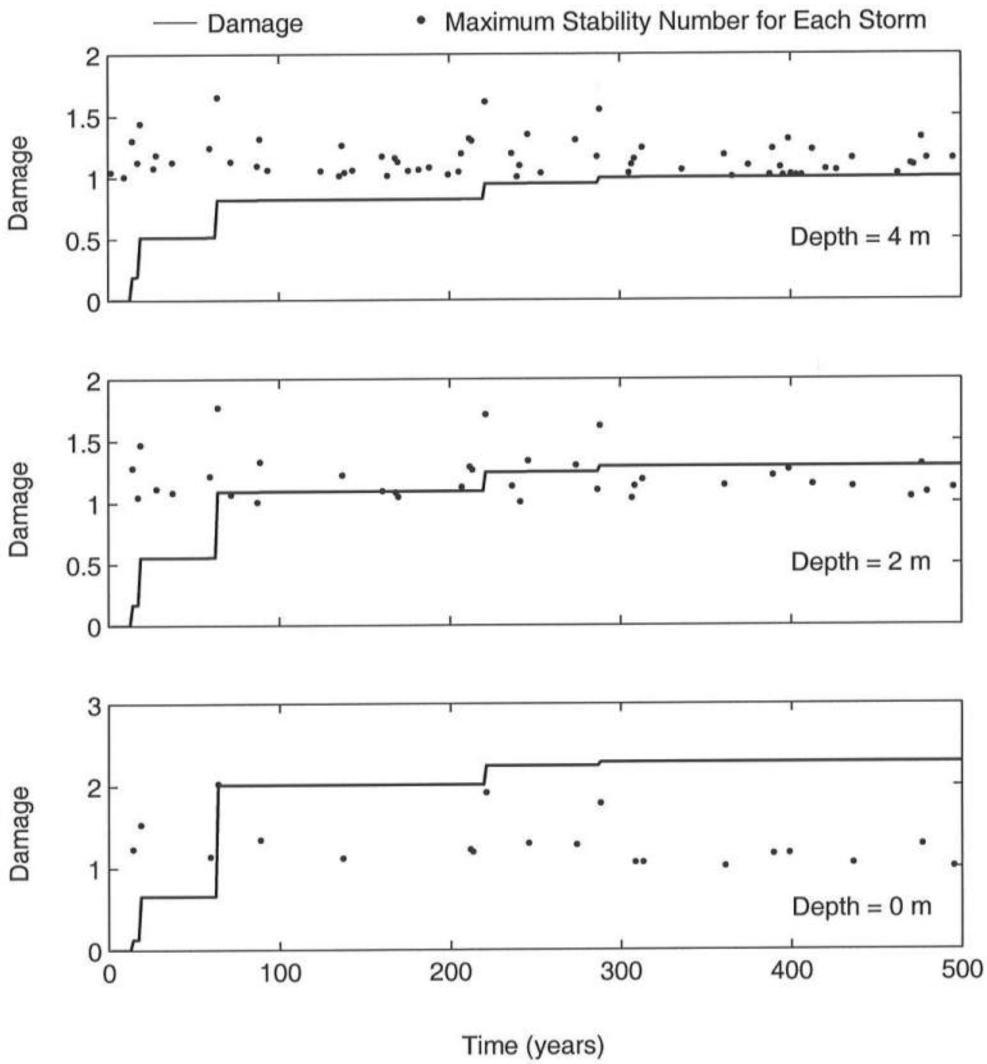

Figure 5.33: Damage Progression of Armor Layer with High Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/800 Slope at Locations of $d = 4$, 2 and 0 m for $N500 = 8$.

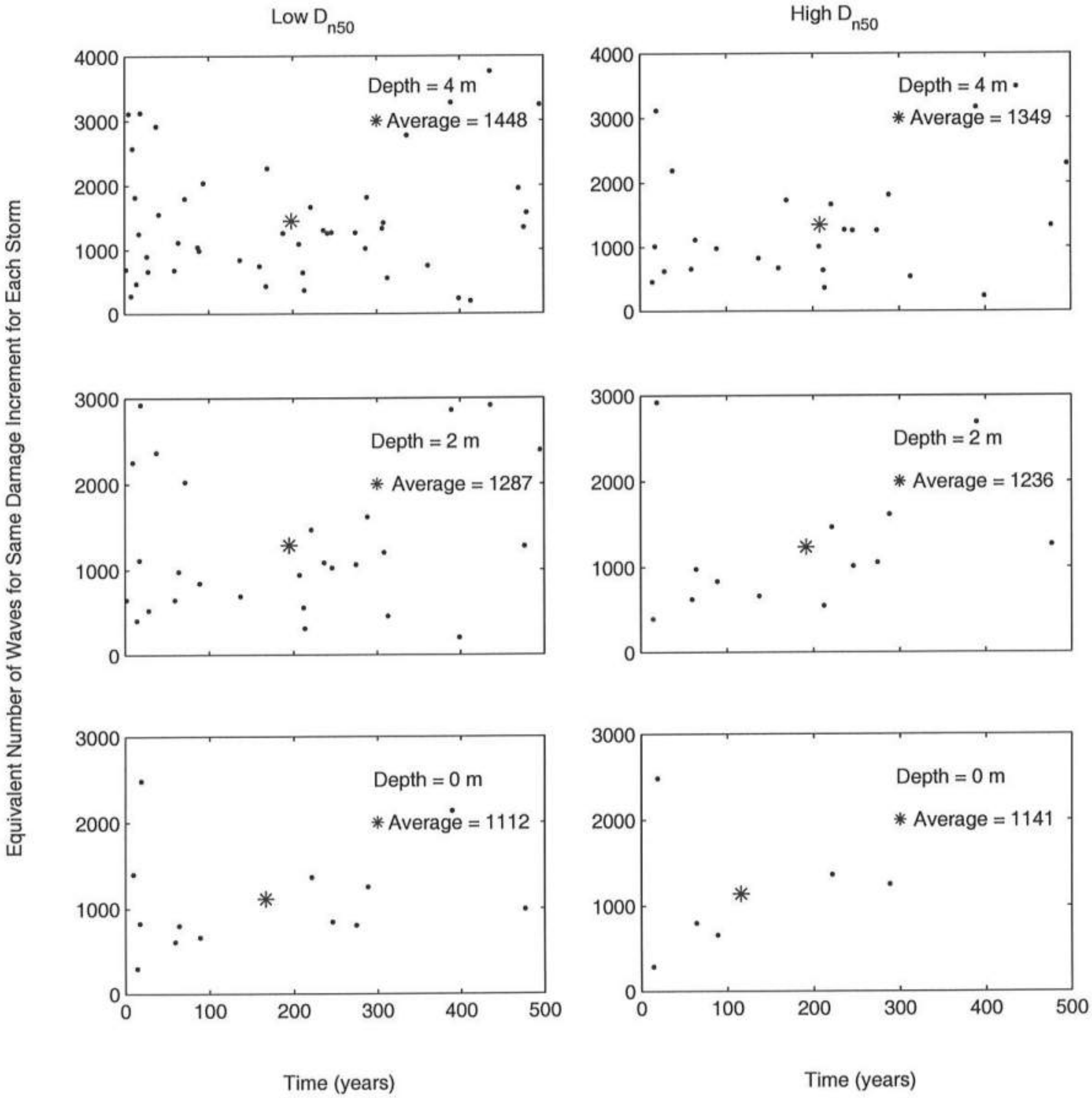

Figure 5.34: Equivalent Number of Waves to Cause the Same Damage Increment for Low and High Diameters D_{n50} on 1/800 Slope at Locations of d = 4, 2 and 0 m for $N500$ = 8.

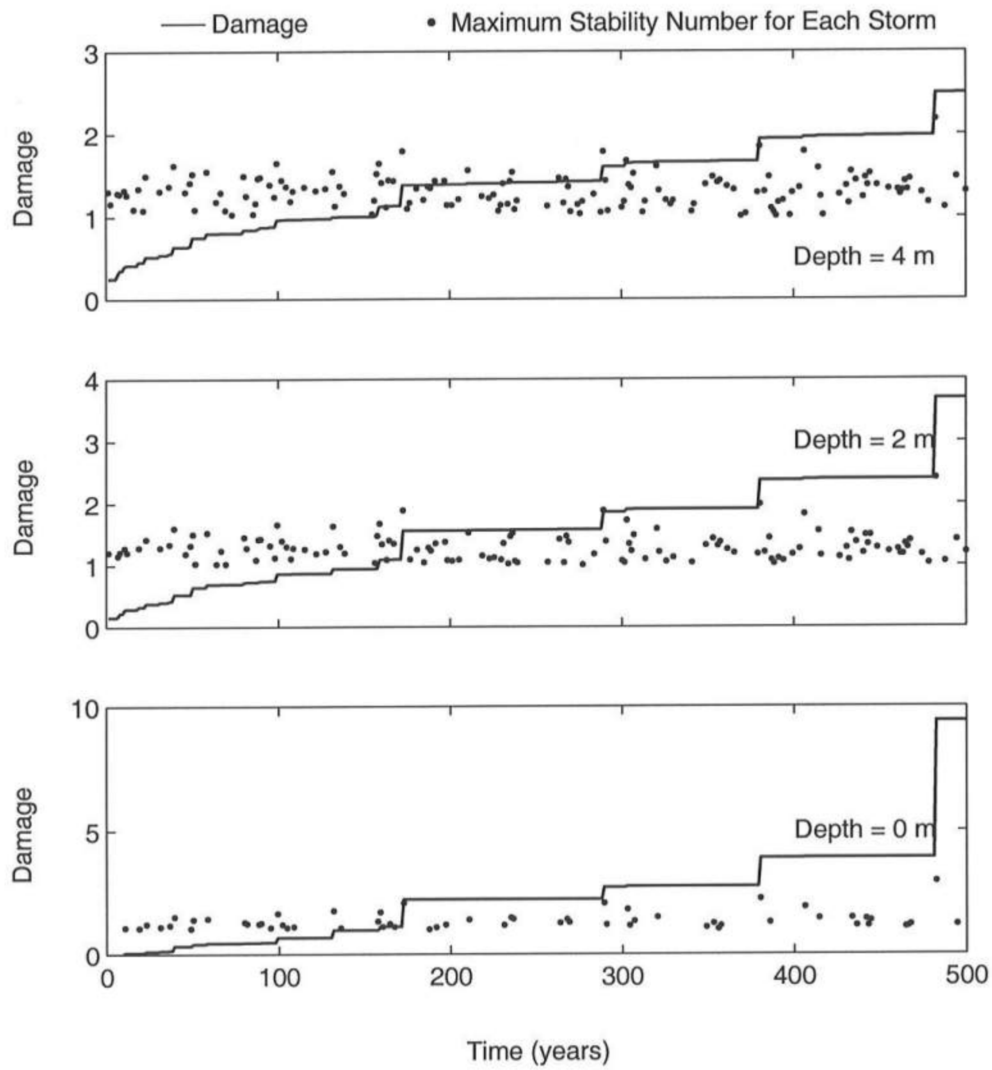

Figure 5.35: Damage Progression of Armor Layer with Low Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/800 Slope at Locations of $d = 4$, 2 and 0 m for $N500 = 9$.

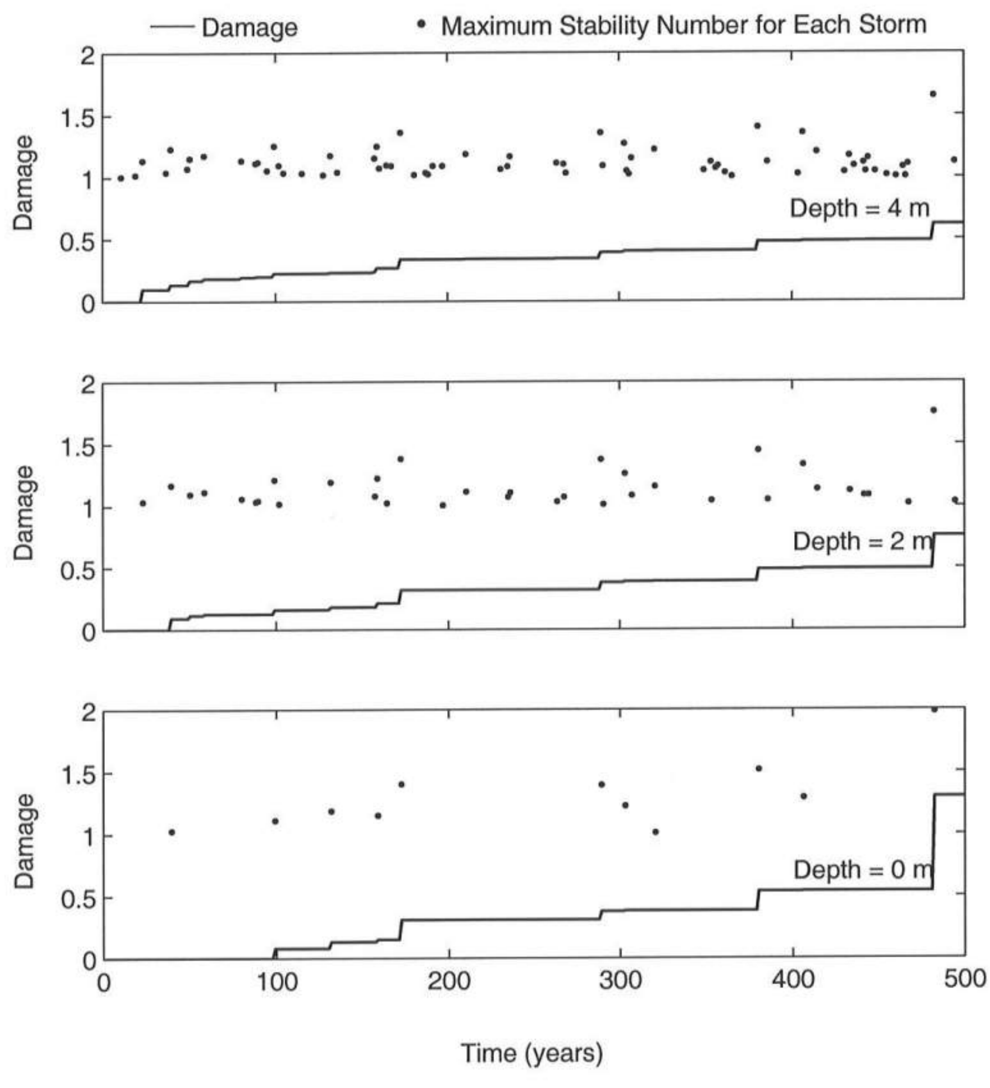

Figure 5.36: Damage Progression of Armor Layer with High Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/800 Slope at Locations of $d = 4$, 2 and 0 m for $N500 = 9$.

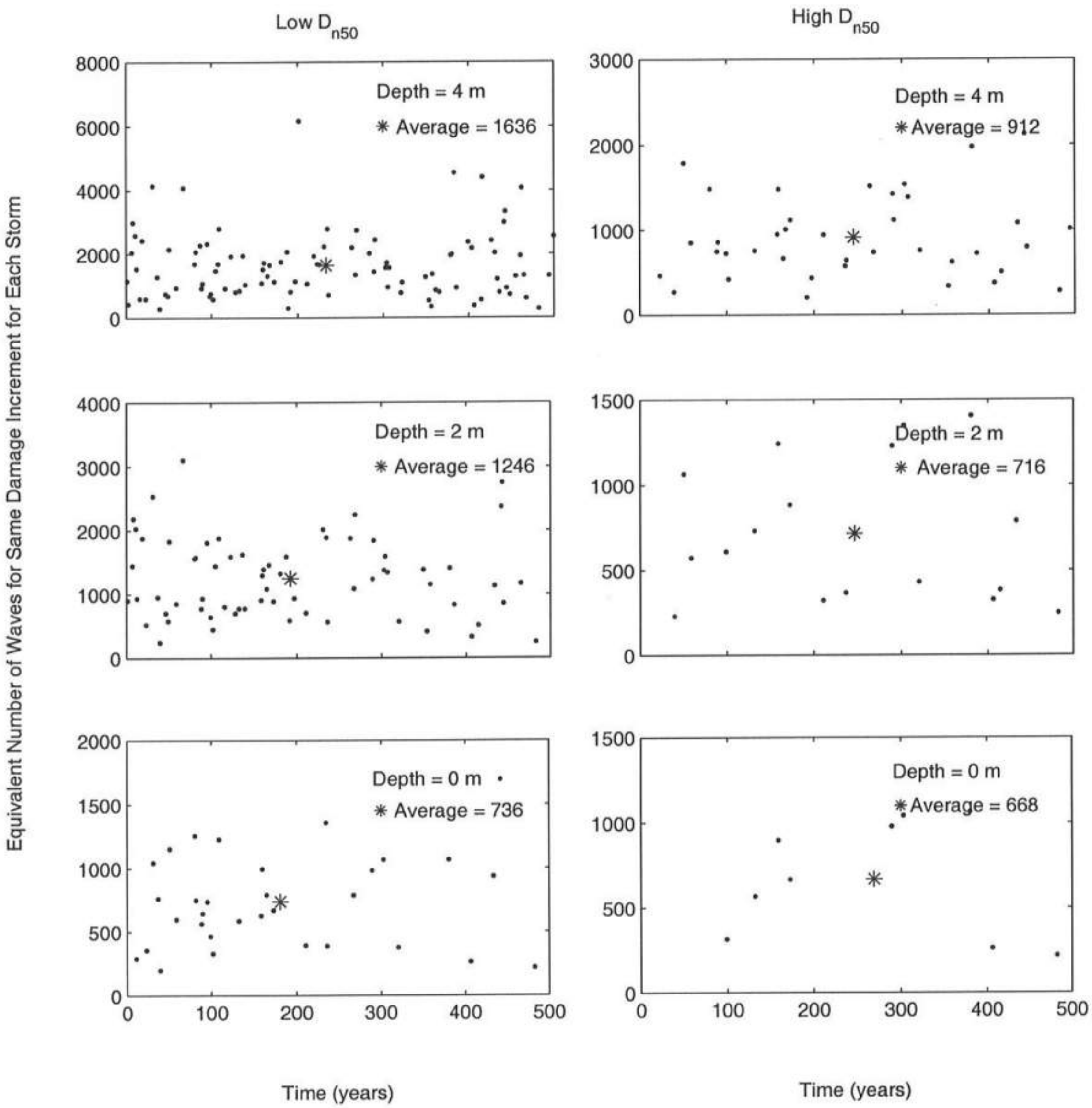

Figure 5.37: Equivalent Number of Waves to Cause the Same Damage Increment for Low and High Diameters D_{n50} on 1/800 Slope at Locations of d = 4, 2 and 0 m for $N500 = 9$.

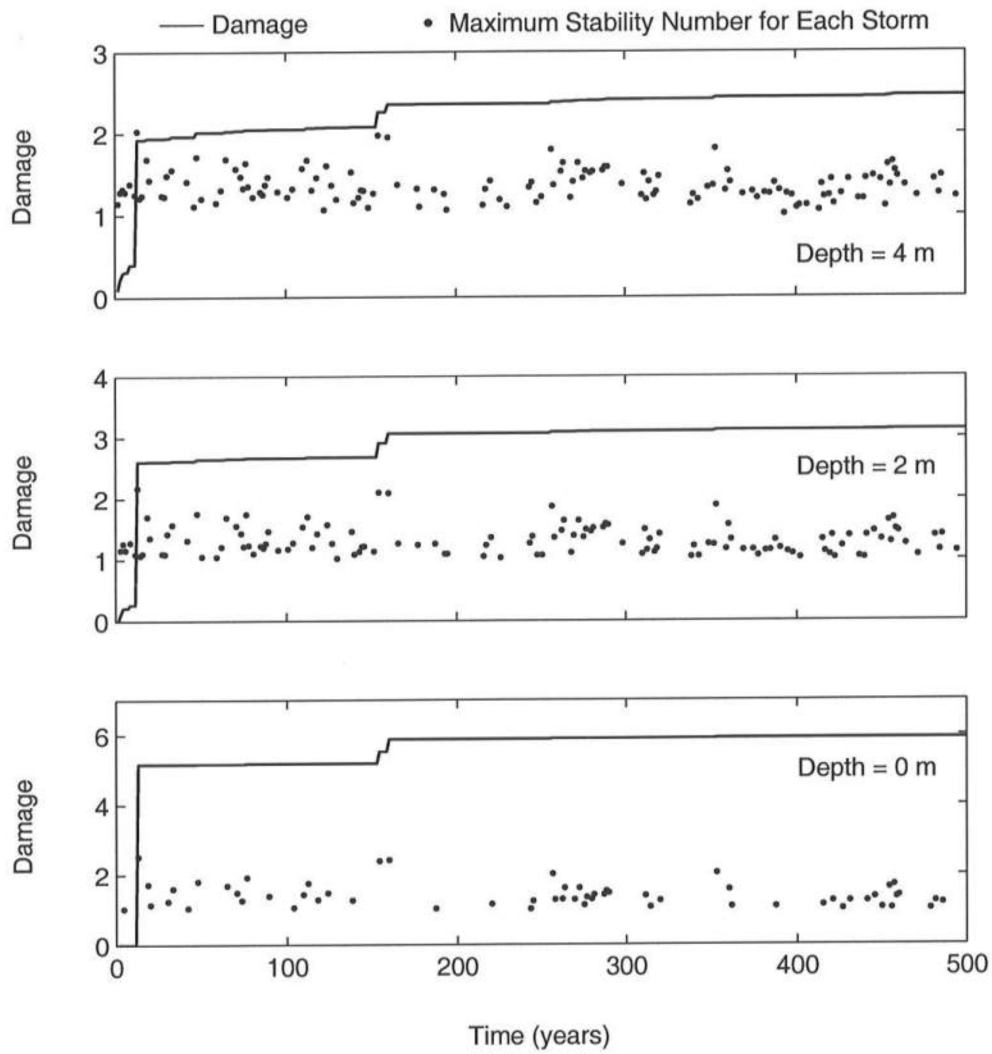

Figure 5.38: Damage Progression of Armor Layer with Low Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/800 Slope at Locations of $d = 4$, 2 and 0 m for $N500 = 10$.

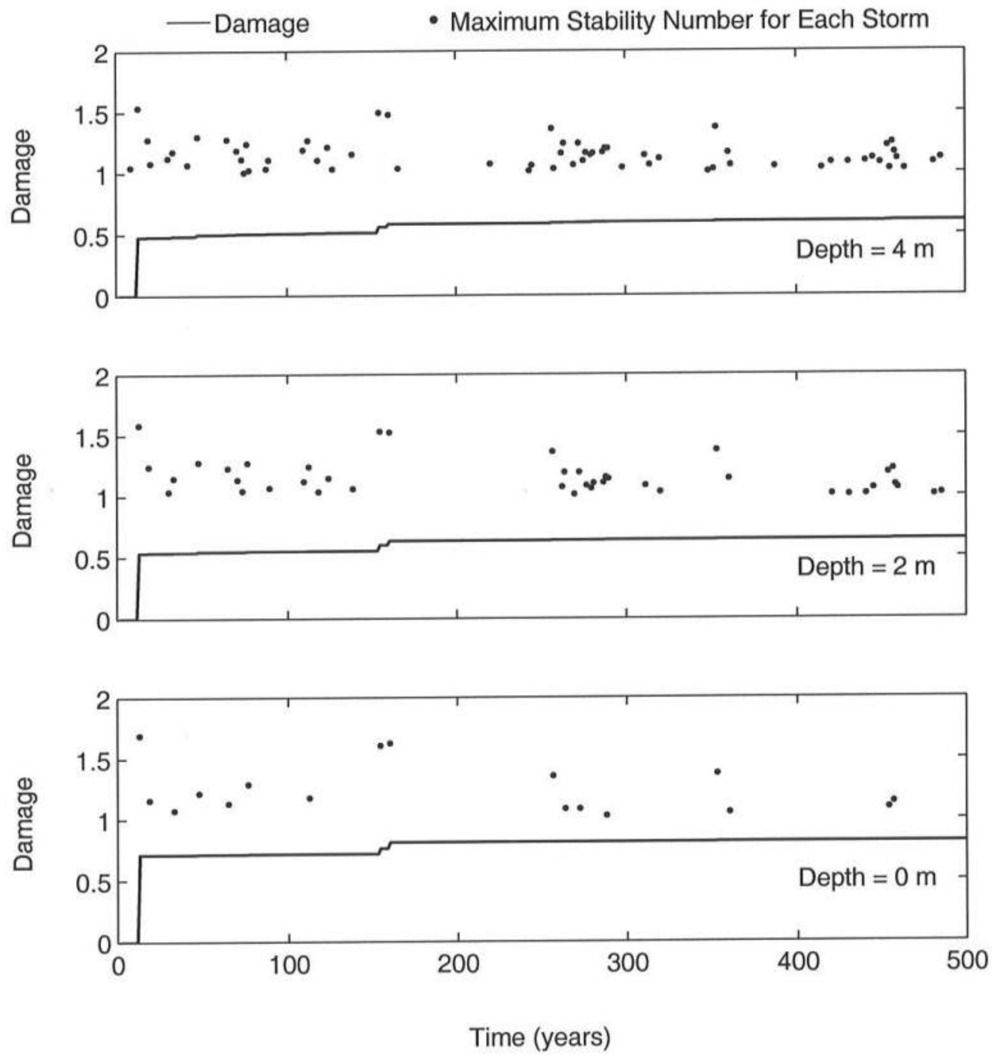

Figure 5.39: Damage Progression of Armor Layer with High Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/800 Slope at Locations of d = 4, 2 and 0 m for $N500$ = 10.

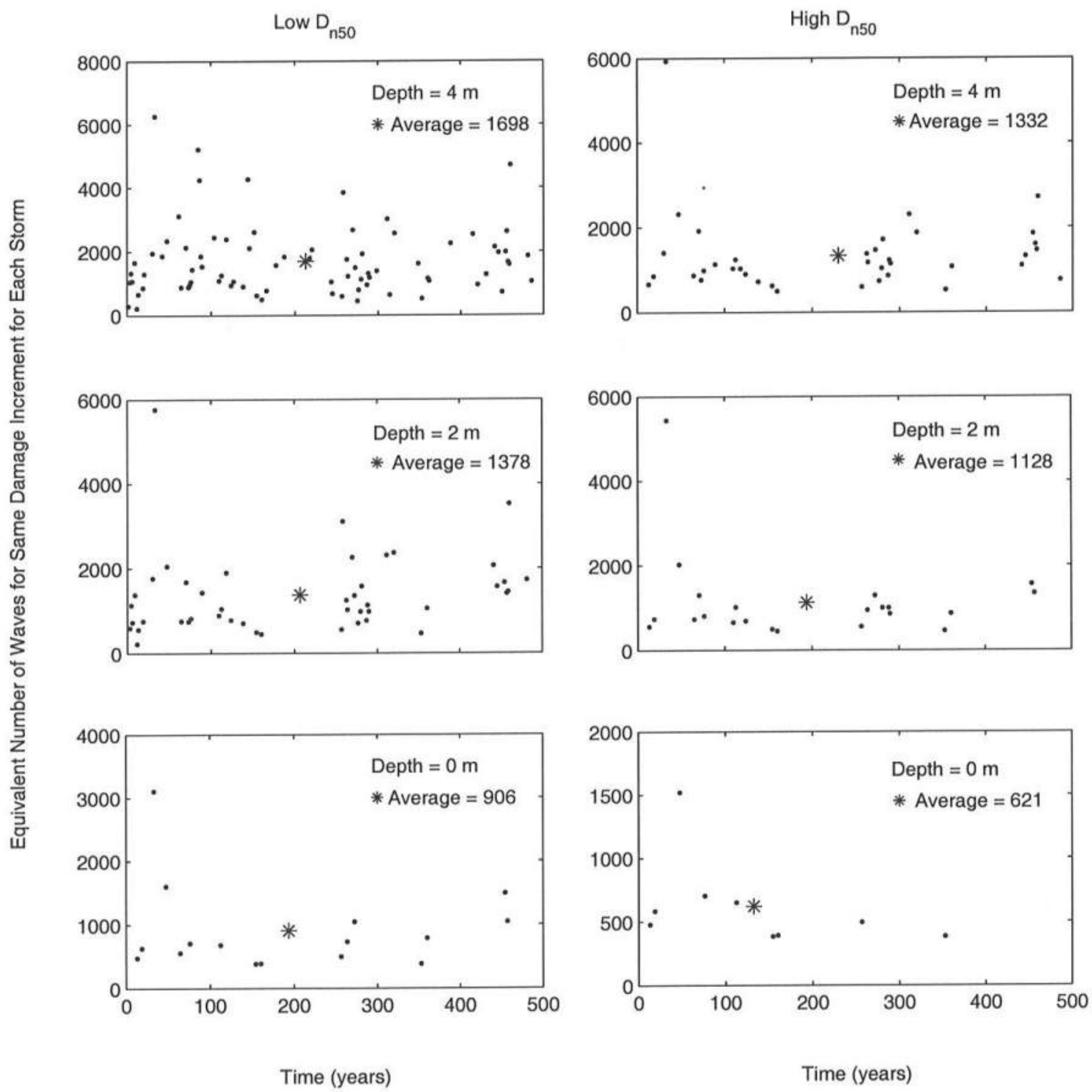

Figure 5.40: Equivalent Number of Waves to Cause the Same Damage Increment for Low and High Diameters D_{n50} on 1/800 Slope at Locations of d = 4, 2 and 0 m for $N500 = 10$.

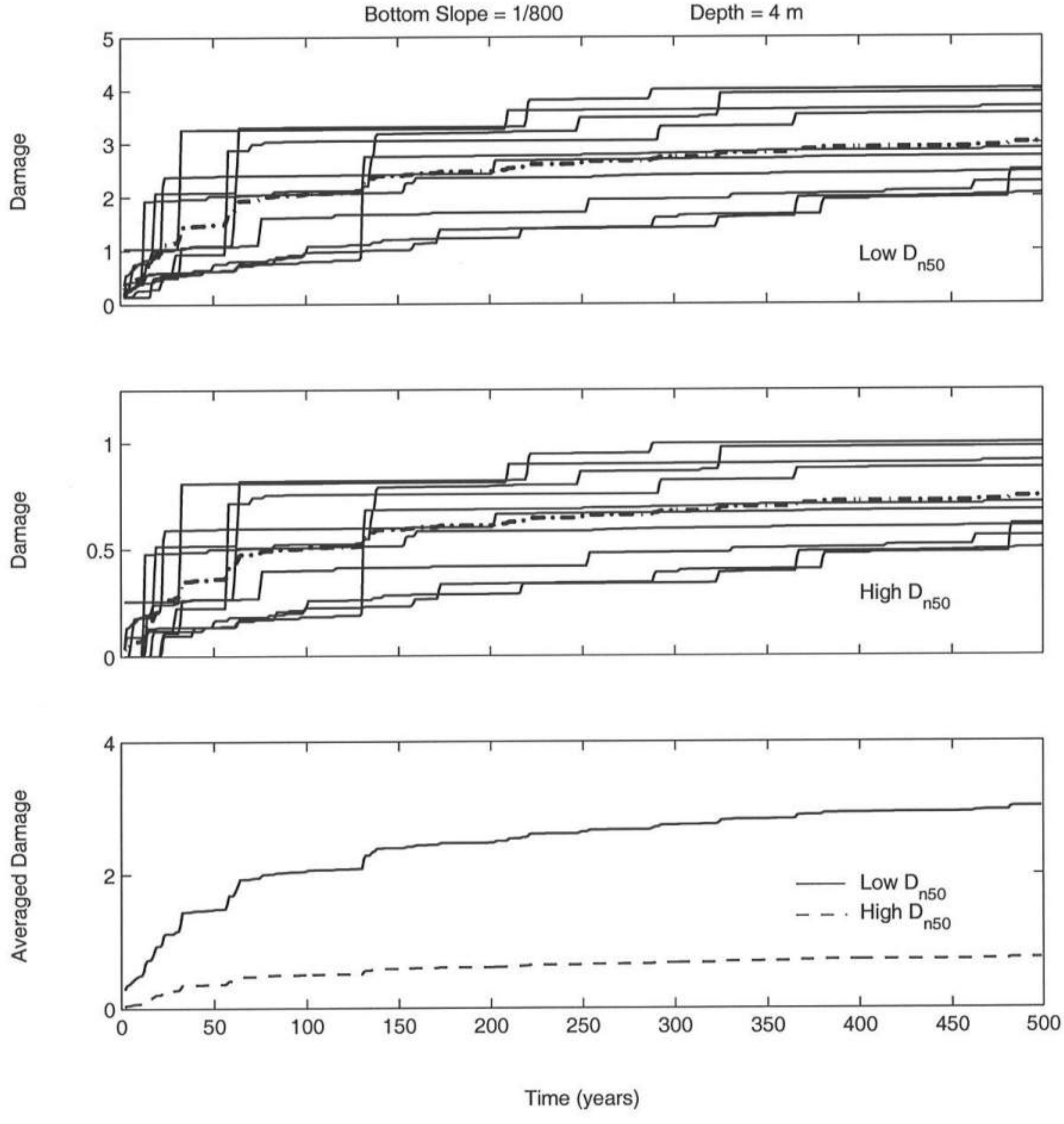

Figure 5.41: Damage Progression of Armor Layer with Low and High Diameters D_{n50} on 1/800 Slope at Location of $d = 4$ m for Each and Average of the Ten 500-yr Simulations: in the Top Two Panels, (—) Each Simulation; (-.-) Average.

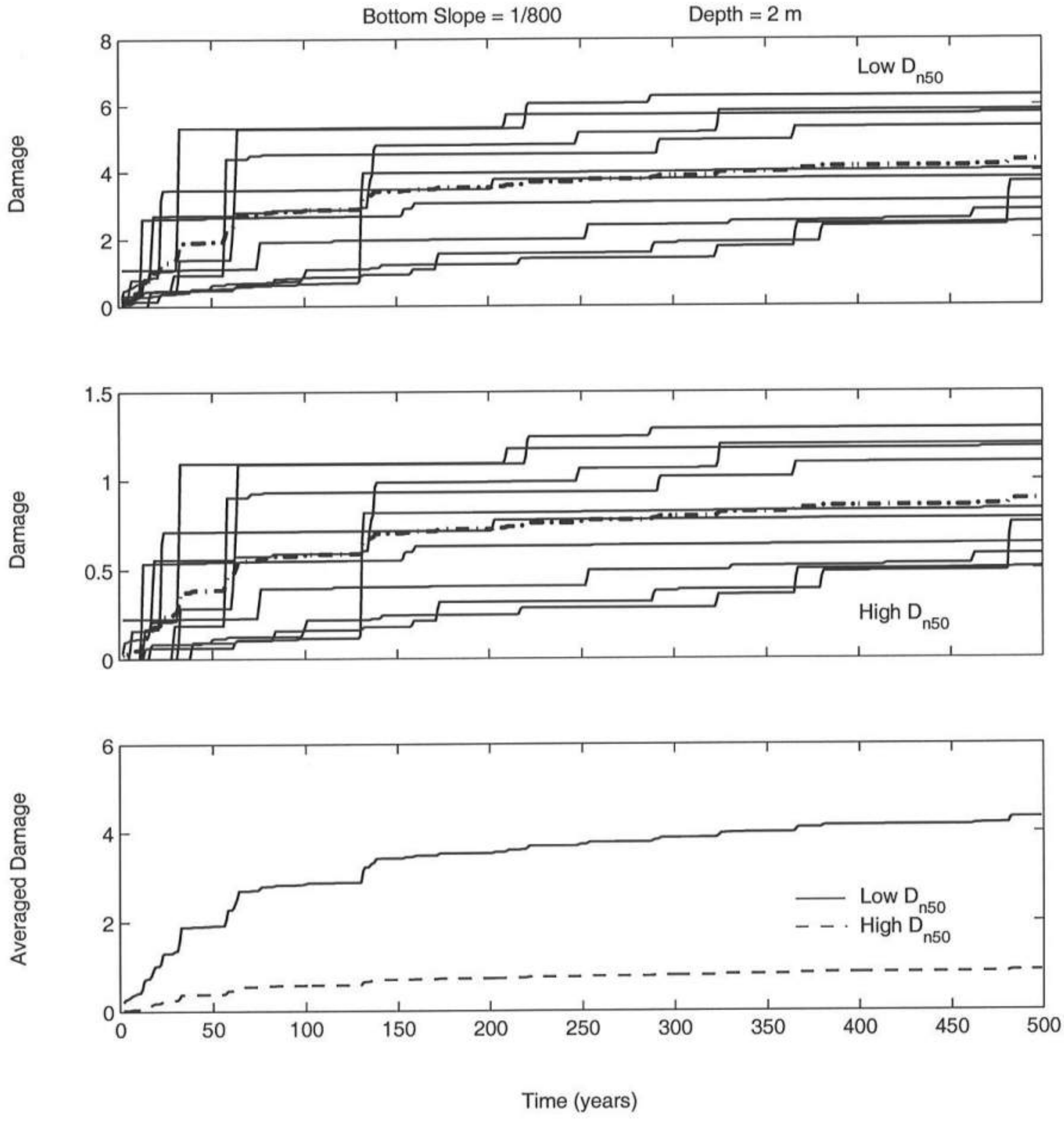

Figure 5.42: Damage Progression of Armor Layer with Low and High Diameters D_{n50} on 1/800 Slope at Location of d = 2 m for Each and Average of the Ten 500-yr Simulations: in the Top Two Panels, (—) Each Simulation; (-.-) Average.

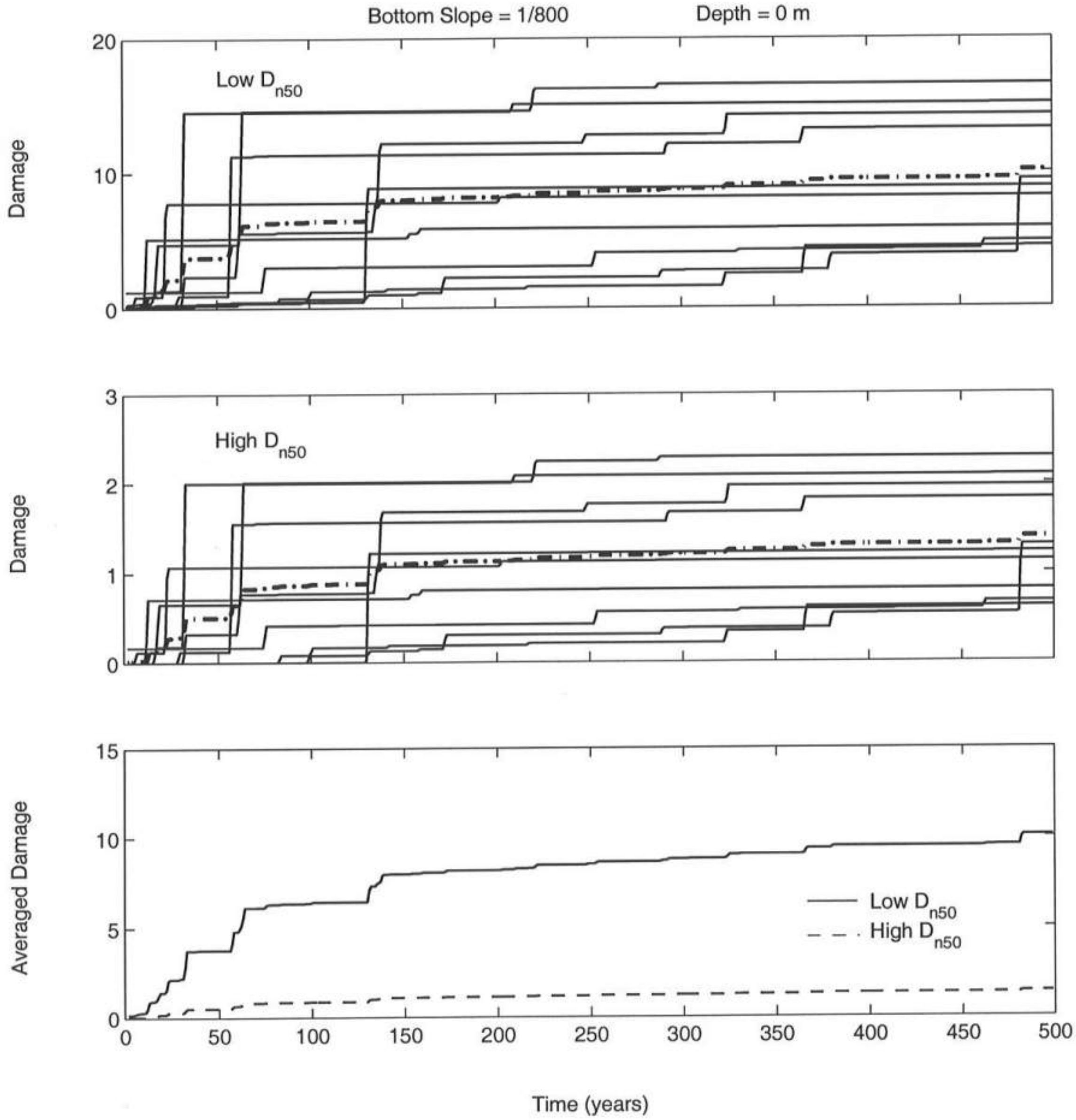

Figure 5.43: Damage Progression of Armor Layer with Low and High Diameters D_{n50} on 1/800 Slope at Location of $d = 0$ m for Each and Average of the Ten 500-yr Simulations: in the Top Two Panels, (—) Each Simulation; (-.-) Average.

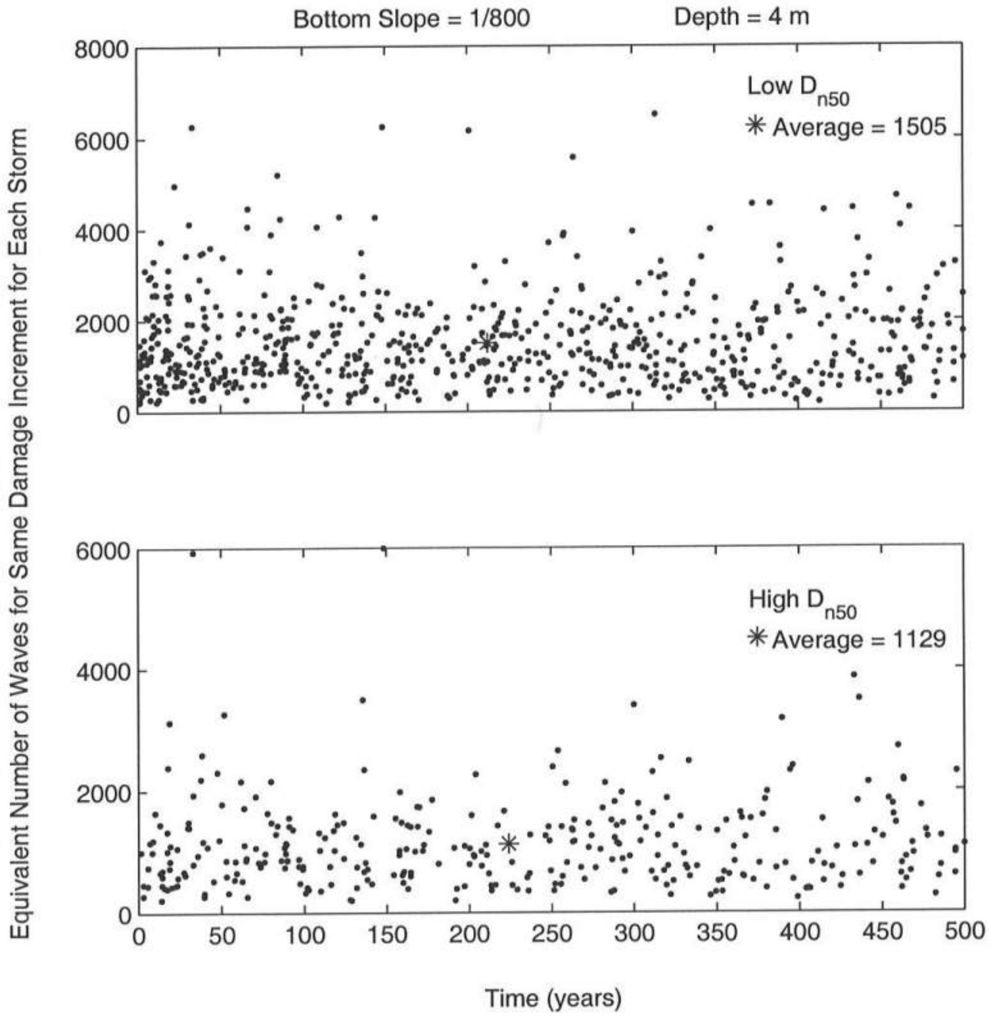

Figure 5.44: Equivalent Number of Waves to Cause the Same Damage Increment for Low and High Diameters D_{n50} on 1/800 Slope at Location of d = 4 m for Ten 500-yr Simulations.

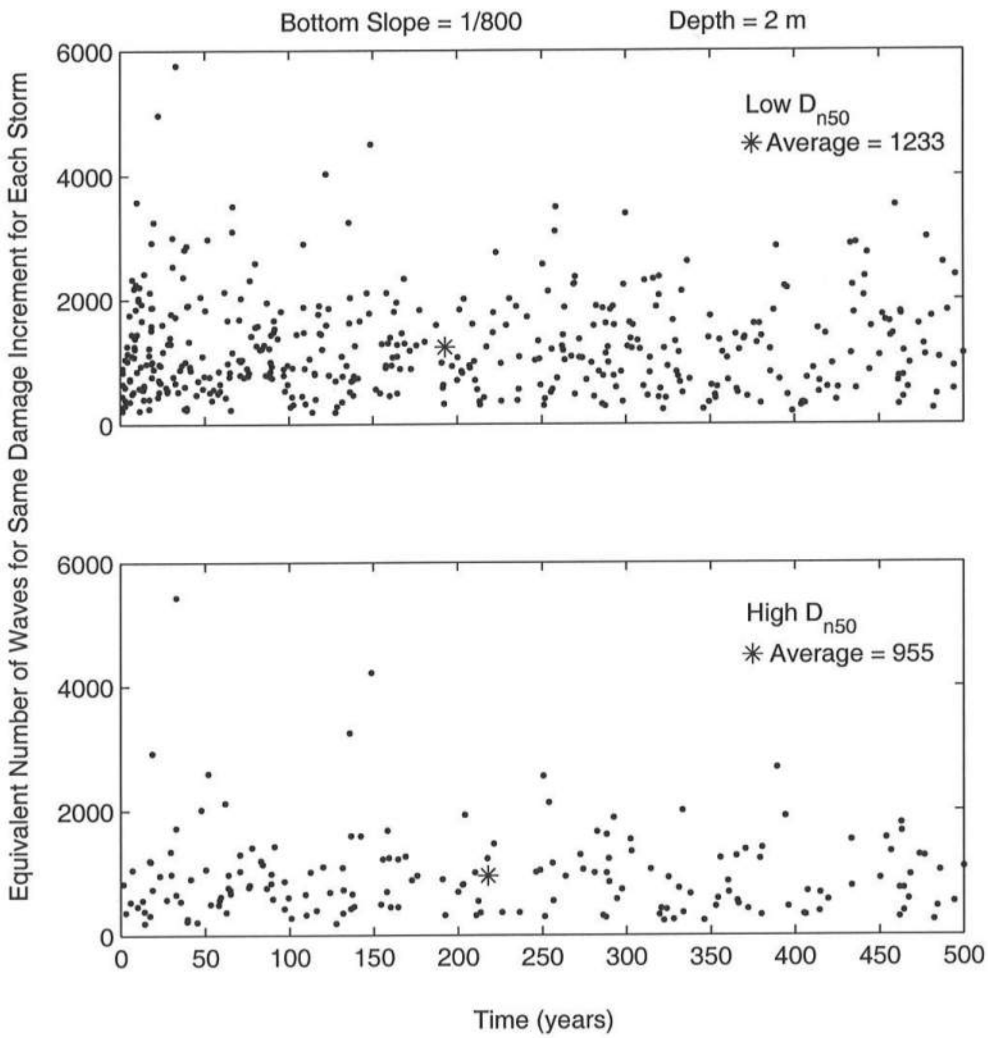

Figure 5.45: Equivalent Number of Waves to Cause the Same Damage Increment for Low and High Diameters D_{n50} on 1/800 Slope at Location of d = 2 m for Ten 500-yr Simulations.

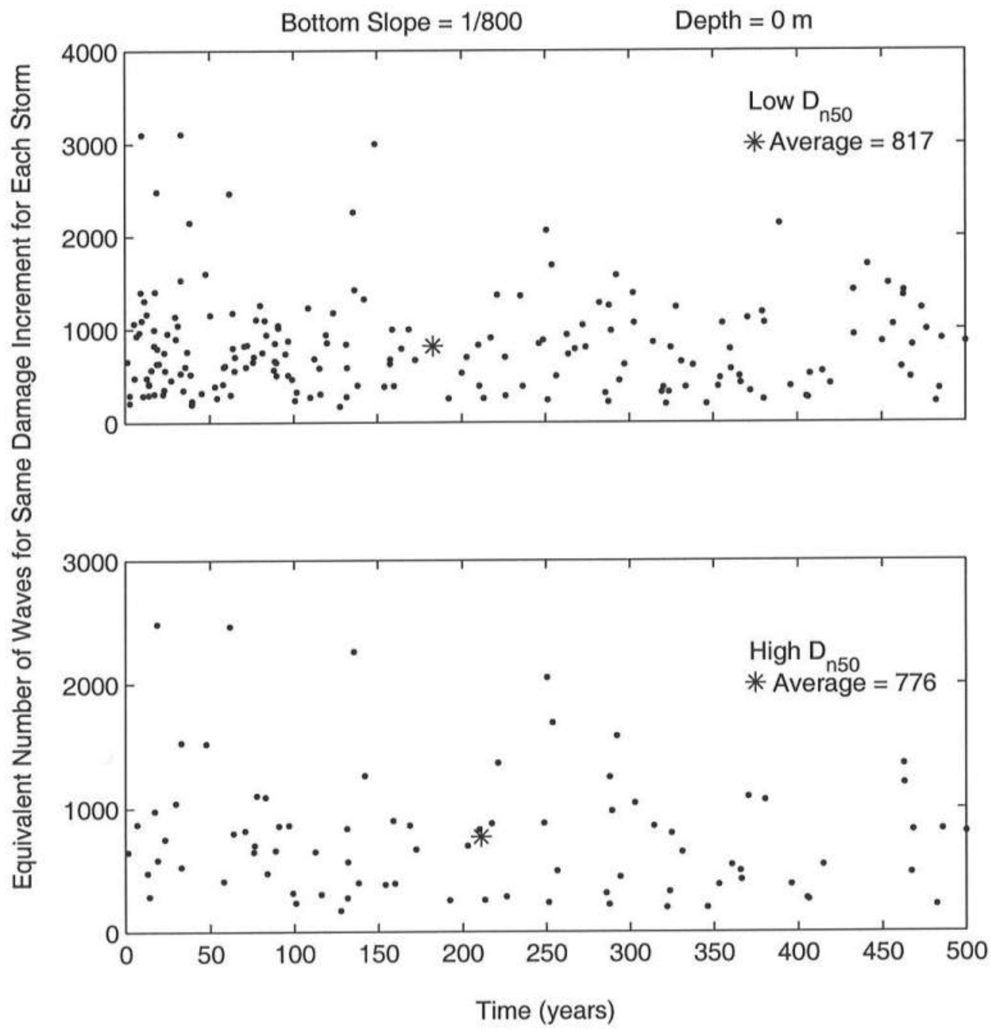

Figure 5.46: Equivalent Number of Waves to Cause the Same Damage Increment for Low and High Diameters D_{n50} on 1/800 Slope at Location of d = 0 m for Ten 500-yr Simulations.

5.7 Individual Figures for Bottom Slope of 1/40

For each of the ten 500-yr simulations (indicated by the integer $N500 =$ 1-10), three figures related to the damage progression are presented first. The summary of the ten 500-yr simulations at the depth $d = 4$, 2 and 0 m is then presented using six figures.

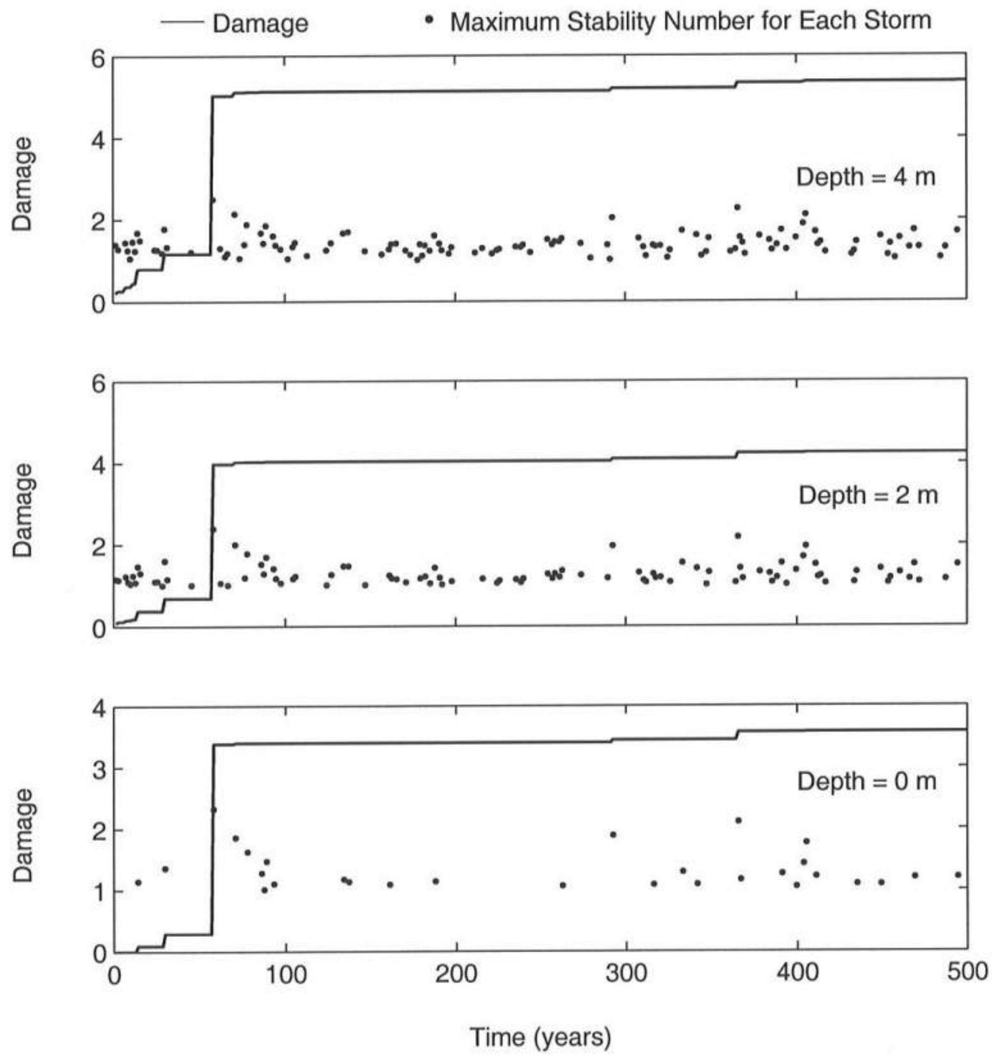

Figure 5.47: Damage Progression of Armor Layer with Low Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/40 Slope at Locations of $d = 4$, 2 and 0 m for $N500 = 1$.

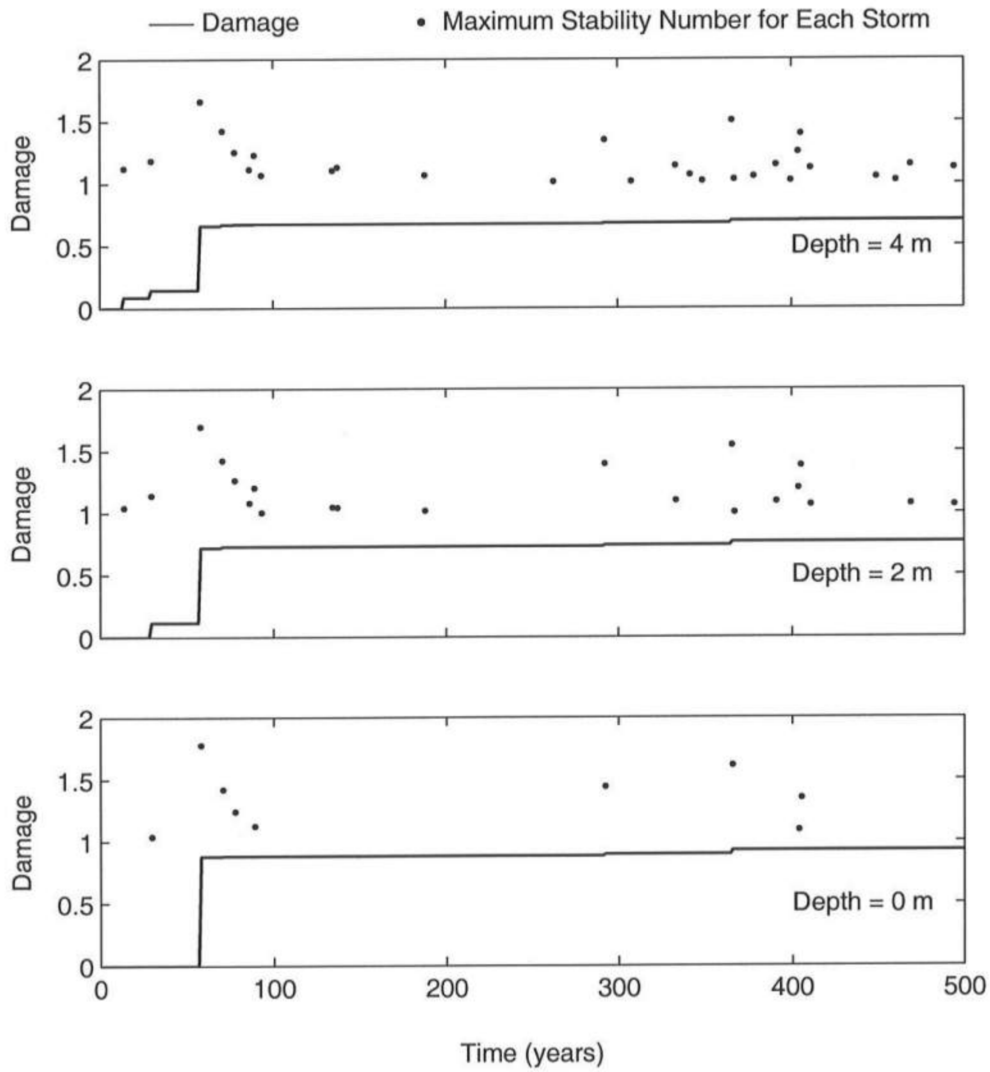

Figure 5.48: Damage Progression of Armor Layer with High Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/40 Slope at Locations of $d = 4$, 2 and 0 m for $N500 = 1$.

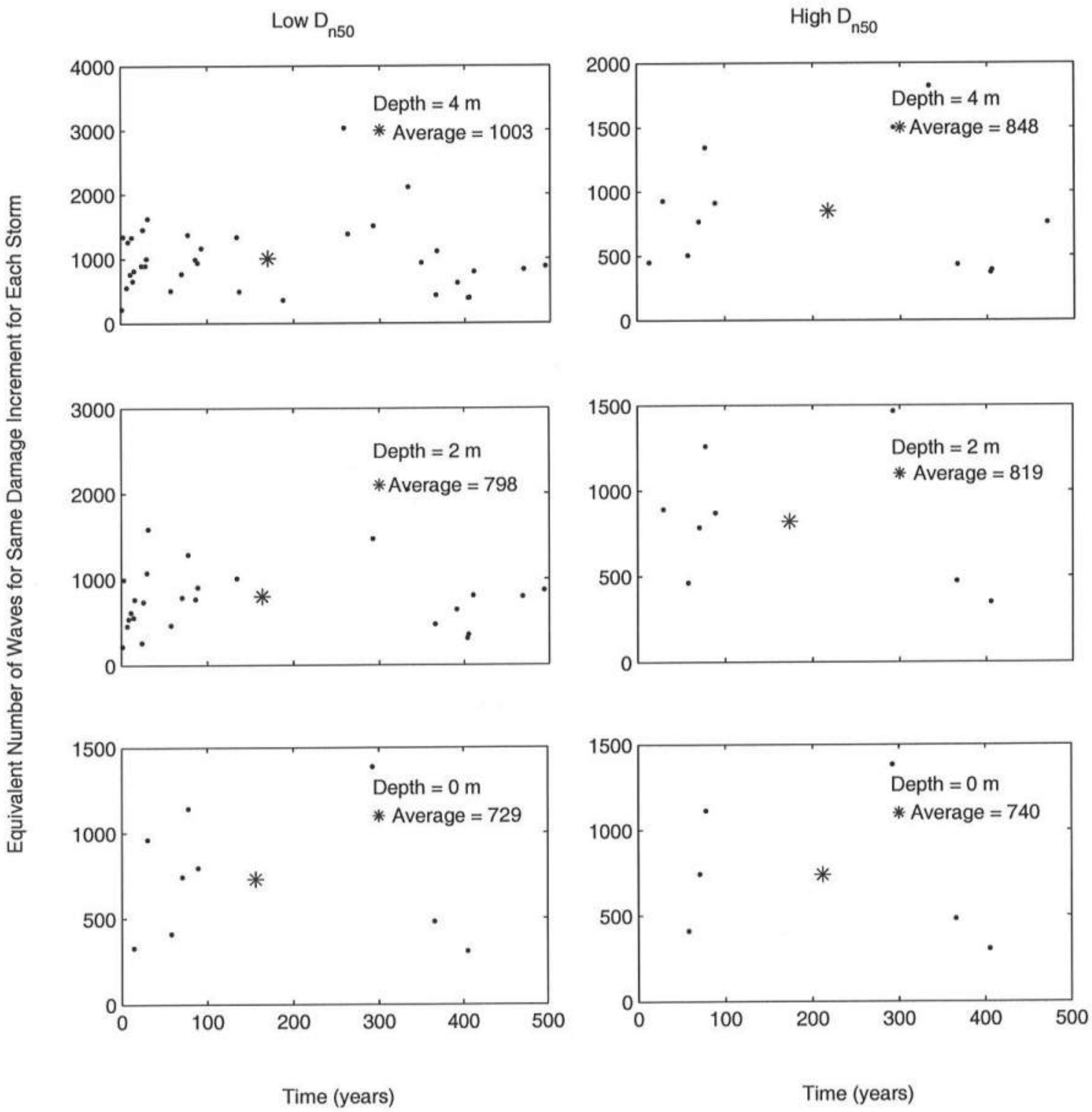

Figure 5.49: Equivalent Number of Waves to Cause the Same Damage Increment for Low and High Diameters D_{n50} on 1/40 Slope at Locations of $d =$ 4, 2 and 0 m for $N500 = 1$.

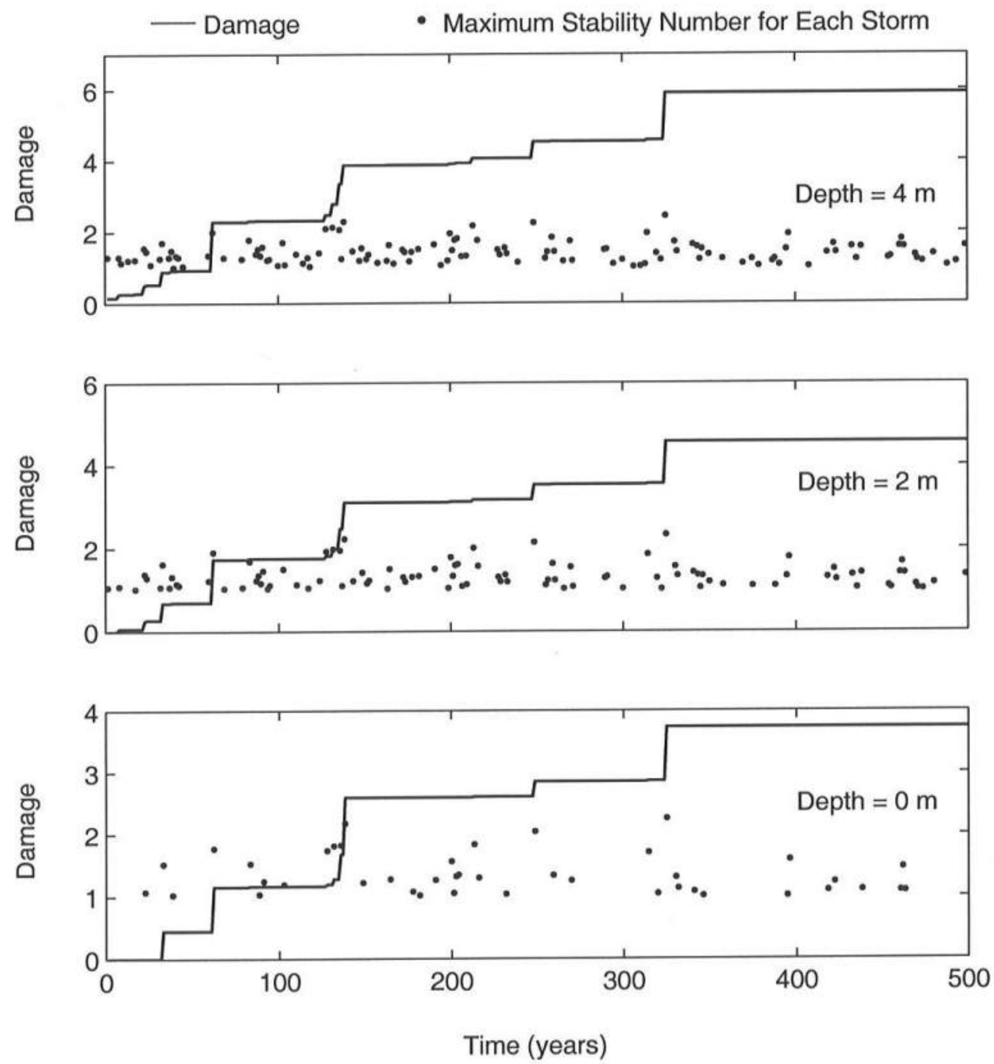

Figure 5.50: Damage Progression of Armor Layer with Low Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/40 Slope at Locations of $d = 4$, 2 and 0 m for $N500 = 2$.

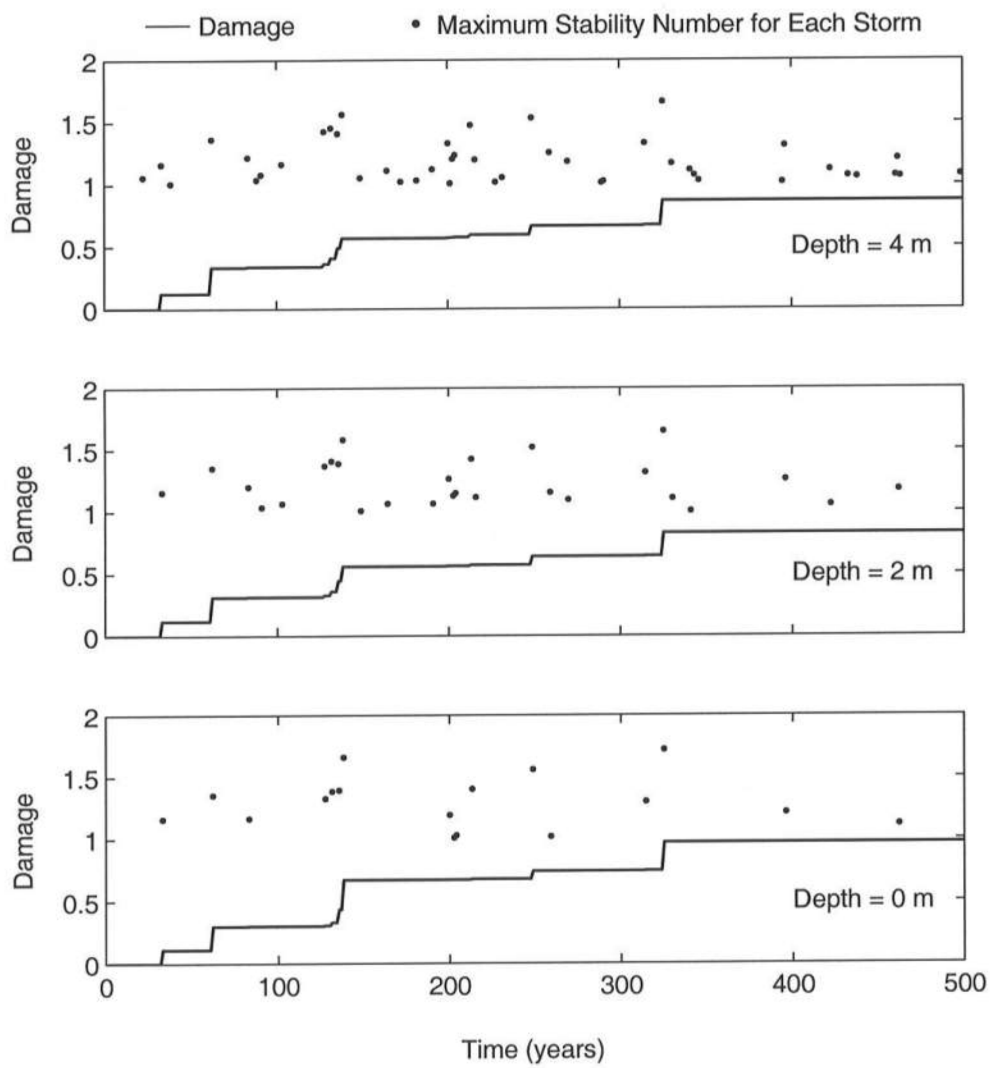

Figure 5.51: Damage Progression of Armor Layer with High Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/40 Slope at Locations of $d = 4$, 2 and 0 m for $N500 = 2$.

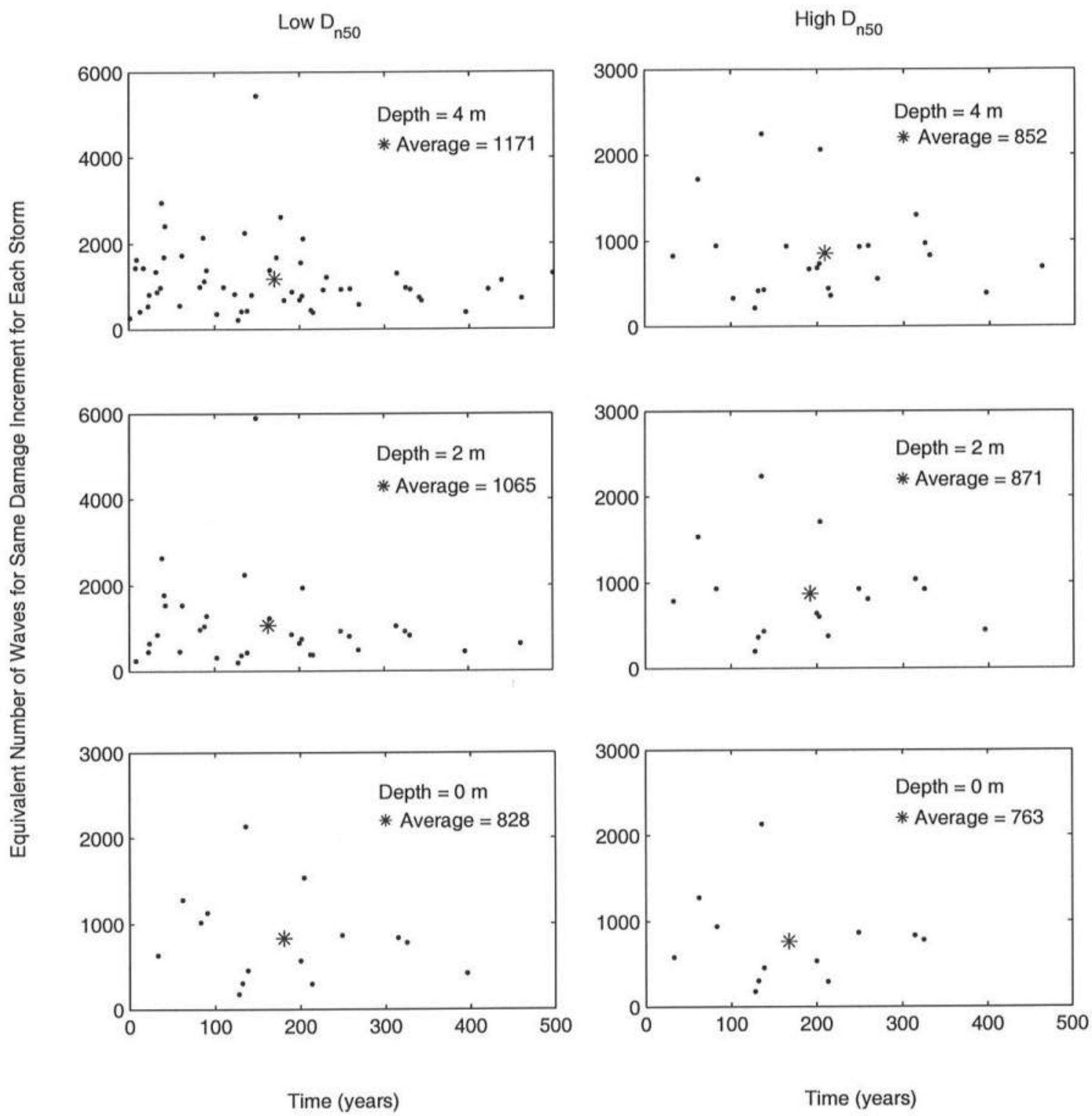

Figure 5.52: Equivalent Number of Waves to Cause the Same Damage Increment for Low and High Diameters D_{n50} on 1/40 Slope at Locations of $d =$ 4, 2 and 0 m for $N500 = 2$.

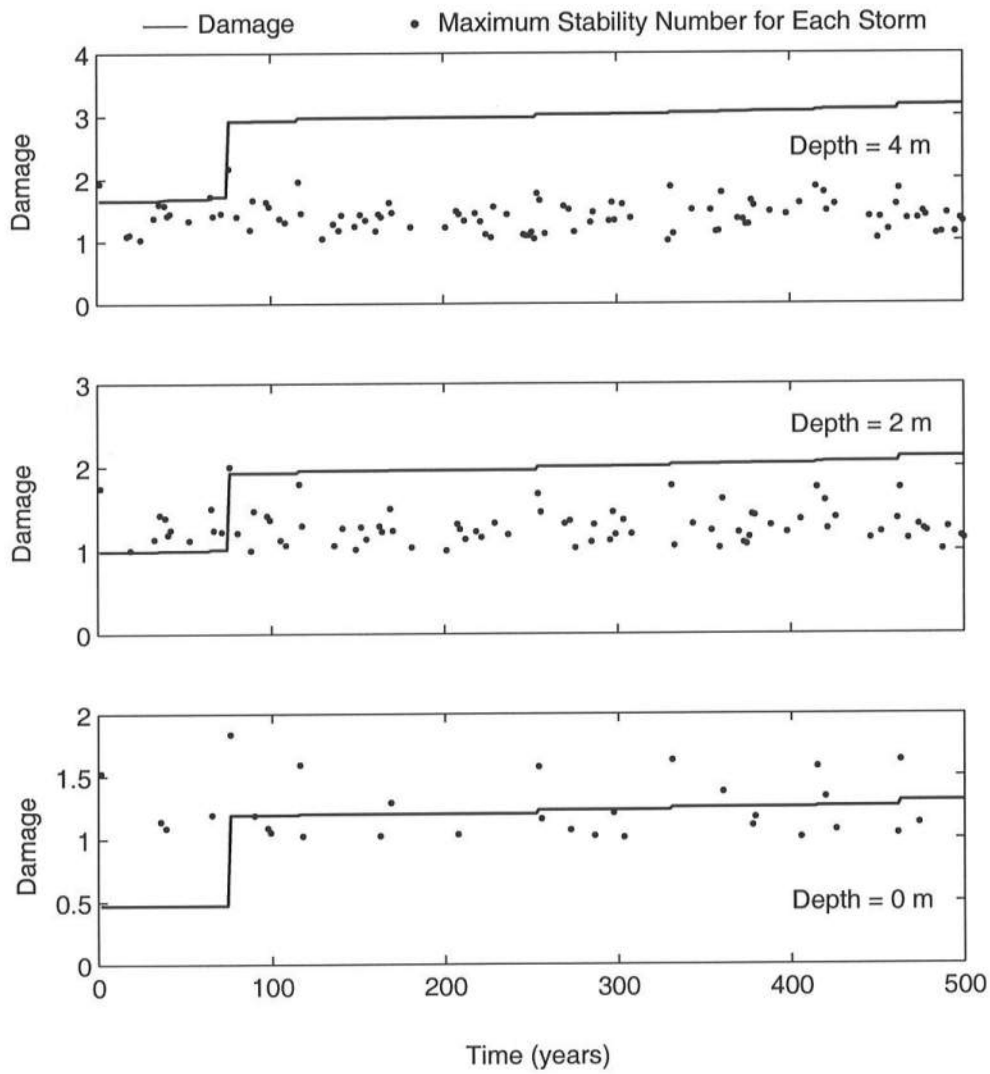

Figure 5.53: Damage Progression of Armor Layer with Low Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on $1/40$ Slope at Locations of $d = 4$, 2 and 0 m for $N500 = 3$.

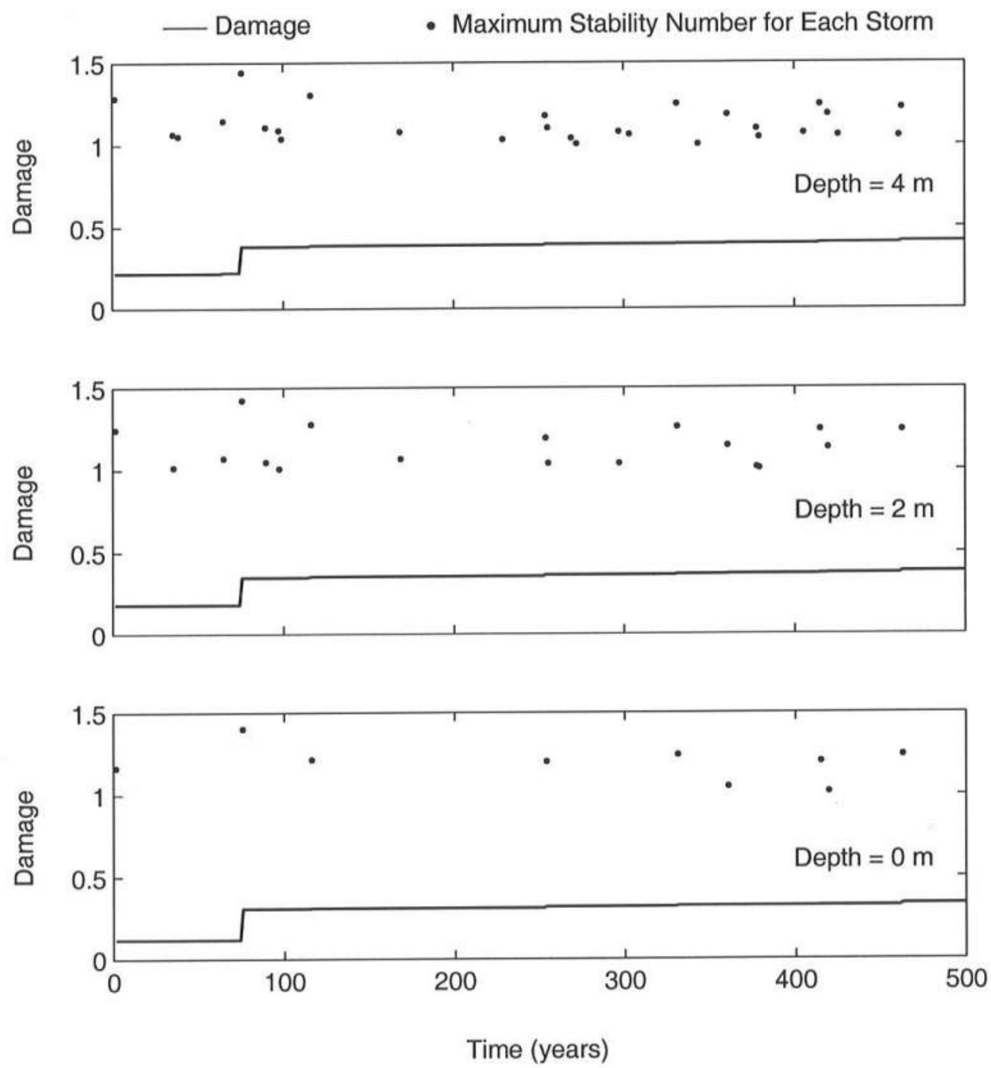

Figure 5.54: Damage Progression of Armor Layer with High Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/40 Slope at Locations of d = 4, 2 and 0 m for $N500 = 3$.

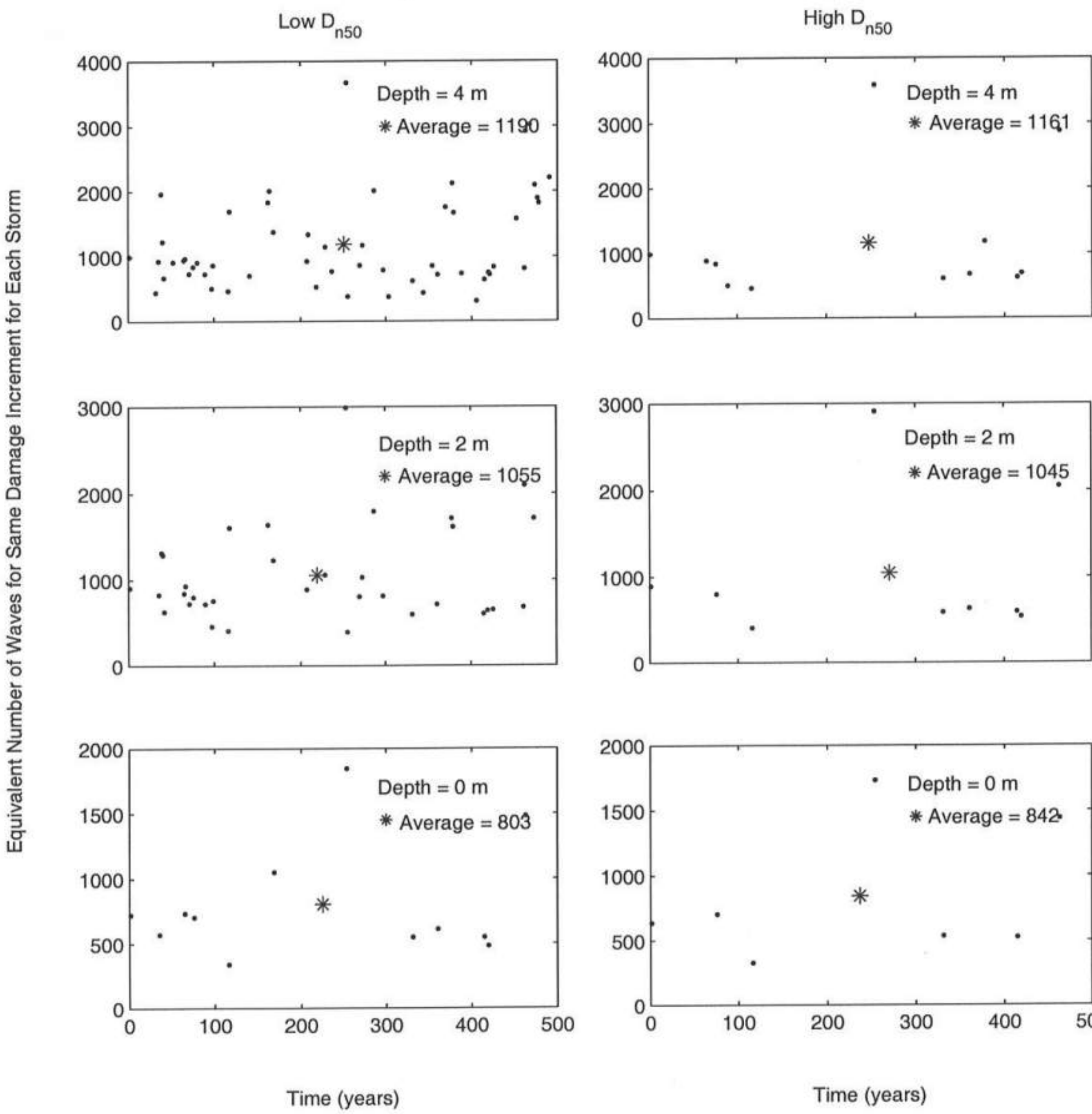

Figure 5.55: Equivalent Number of Waves to Cause the Same Damage Increment for Low and High Diameters D_{n50} on $1/40$ Slope at Locations of $d =$ 4, 2 and 0 m for $N500 = 3$.

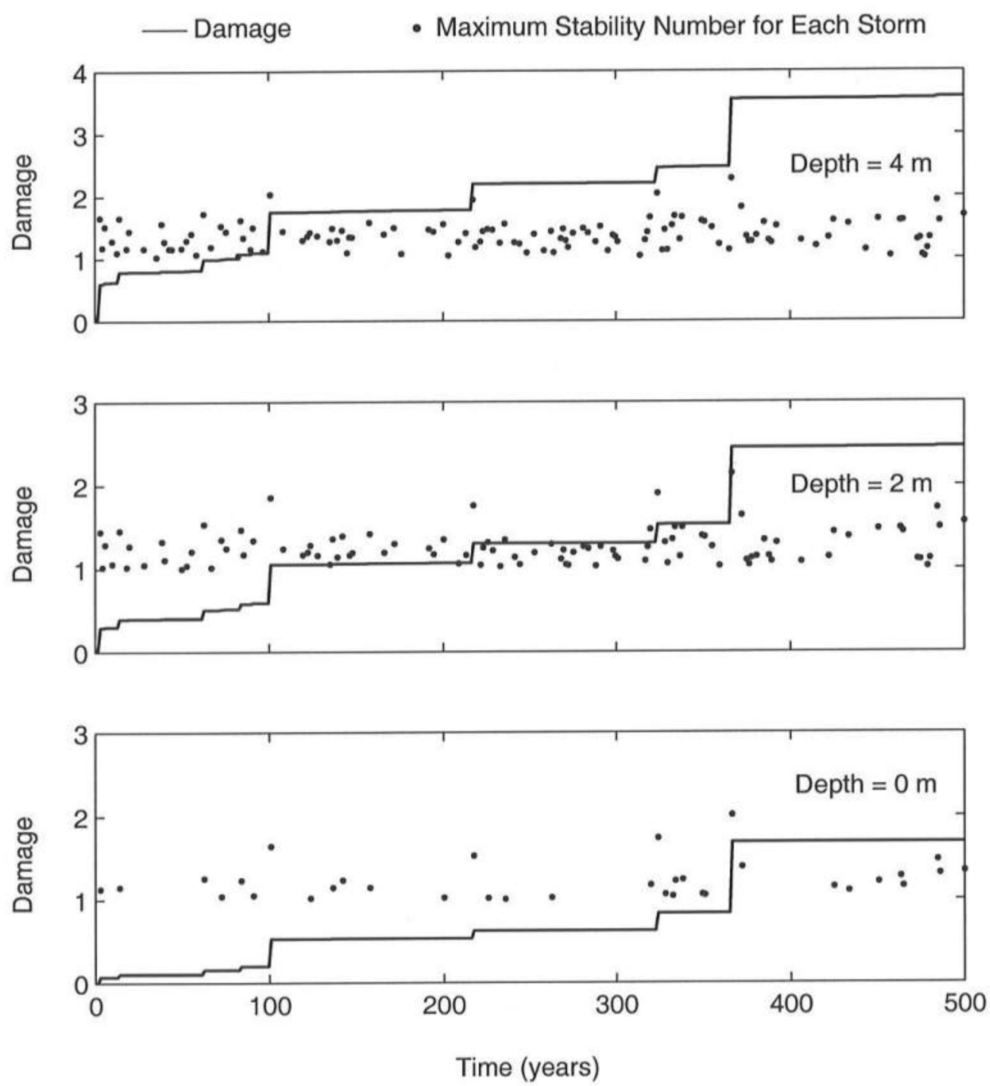

Figure 5.56: Damage Progression of Armor Layer with Low Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/40 Slope at Locations of $d = 4$, 2 and 0 m for $N500 = 4$.

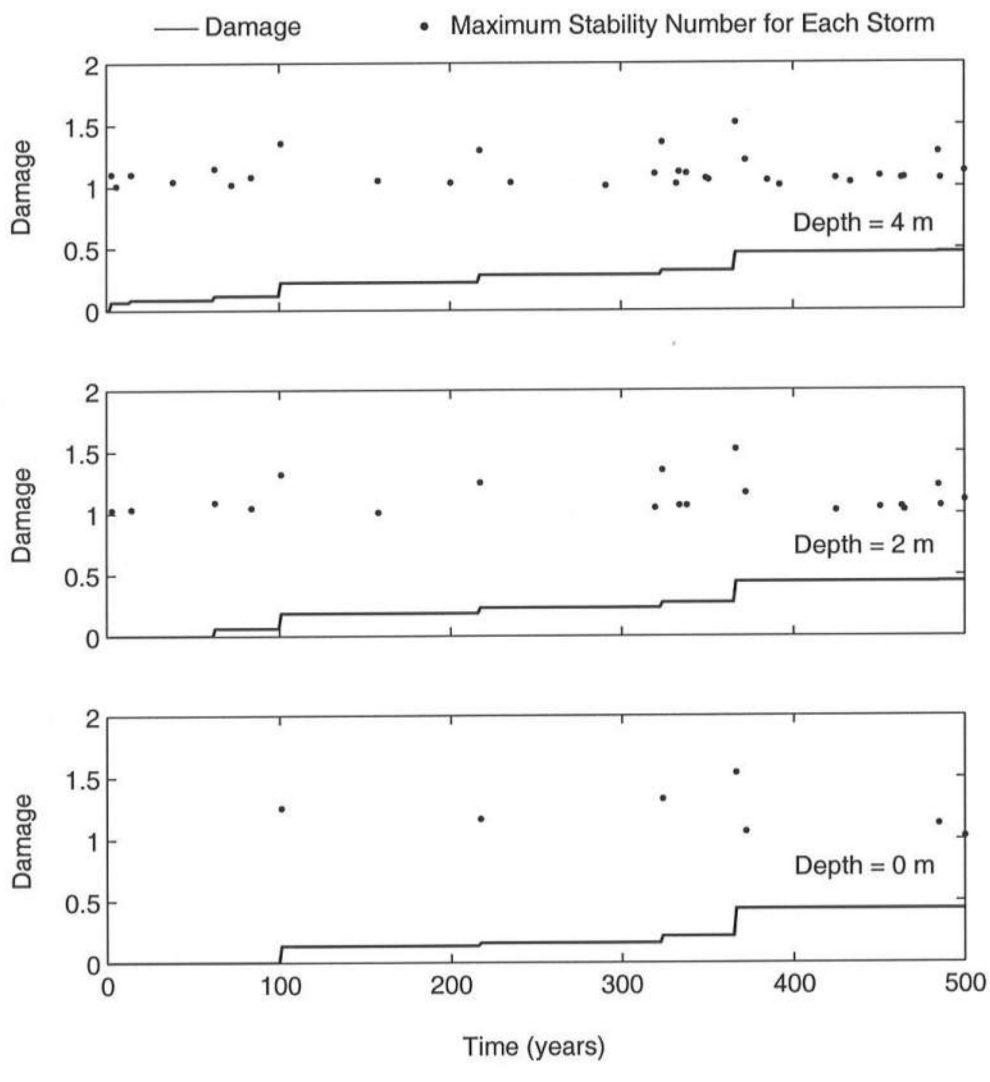

Figure 5.57: Damage Progression of Armor Layer with High Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/40 Slope at Locations of $d = 4$, 2 and 0 m for $N500 = 4$.

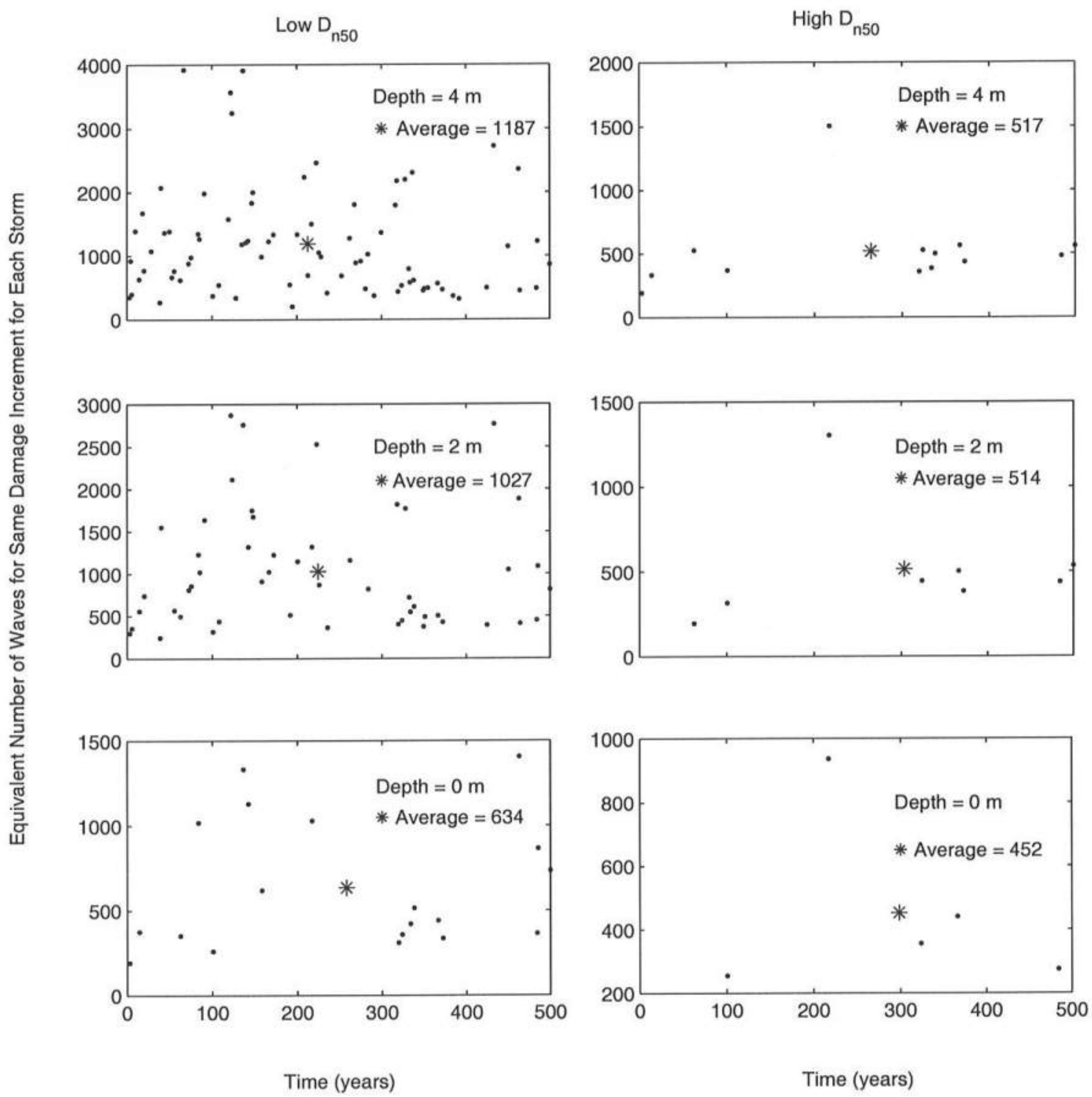

Figure 5.58: Equivalent Number of Waves to Cause the Same Damage Increment for Low and High Diameters D_{n50} on 1/40 Slope at Locations of $d =$ 4, 2 and 0 m for $N500 = 4$.

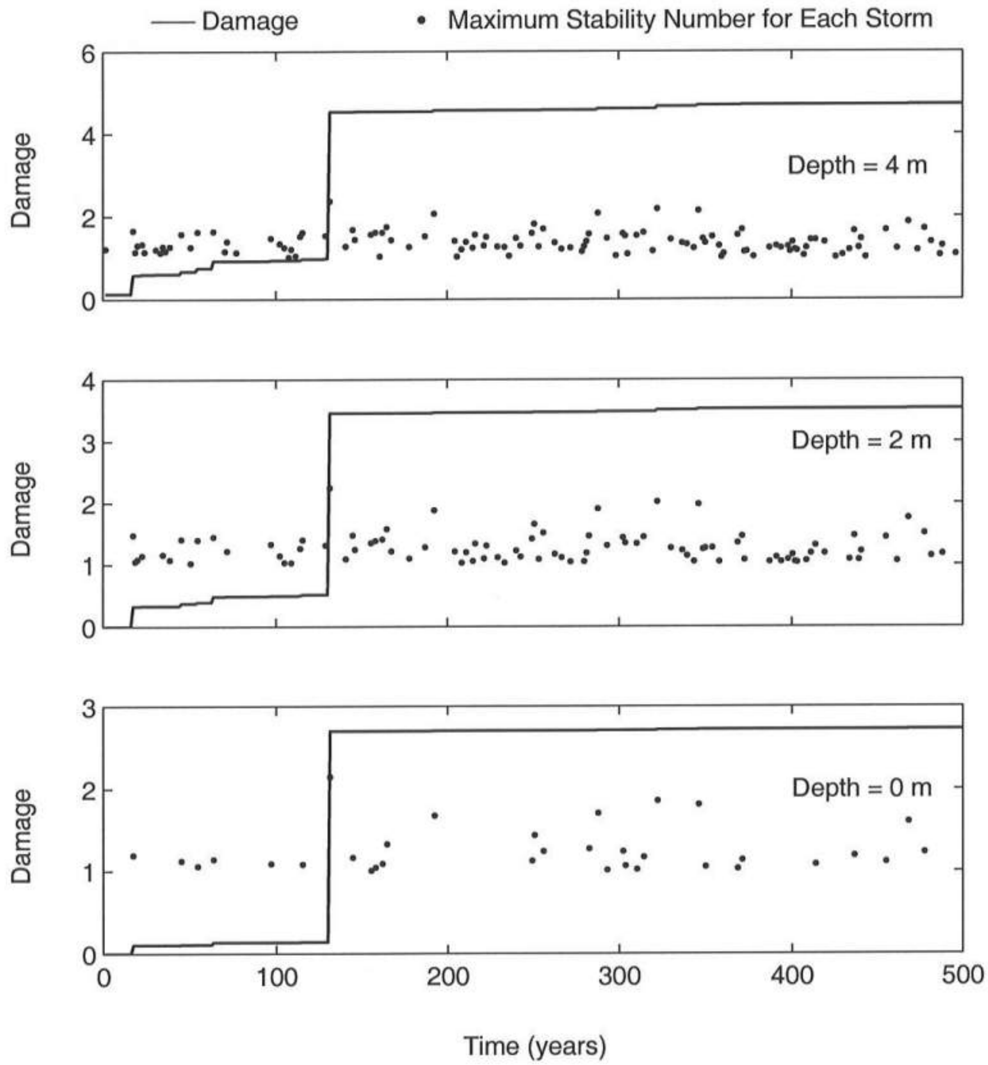

Figure 5.59: Damage Progression of Armor Layer with Low Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/40 Slope at Locations of $d = 4$, 2 and 0 m for $N500 = 5$.

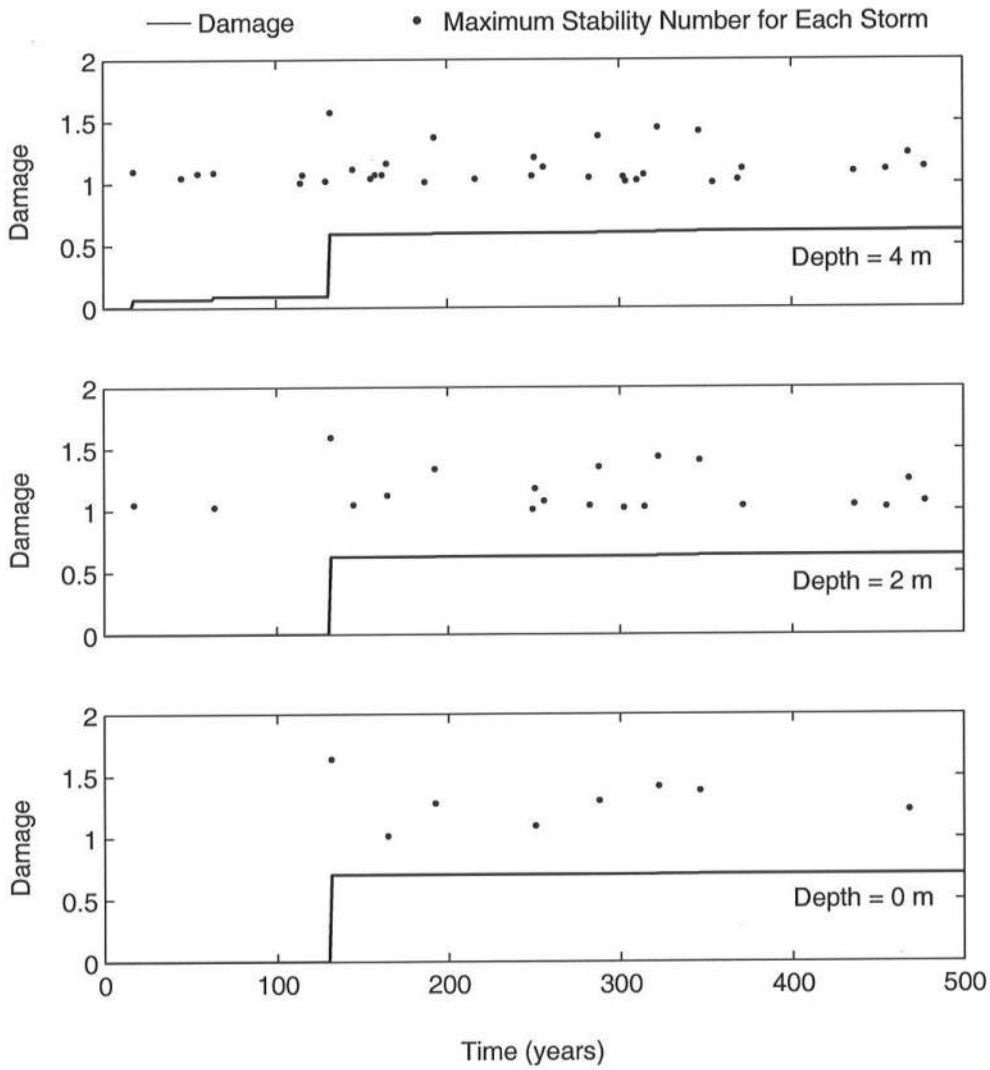

Figure 5.60: Damage Progression of Armor Layer with High Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/40 Slope at Locations of $d = 4$, 2 and 0 m for $N500 = 5$.

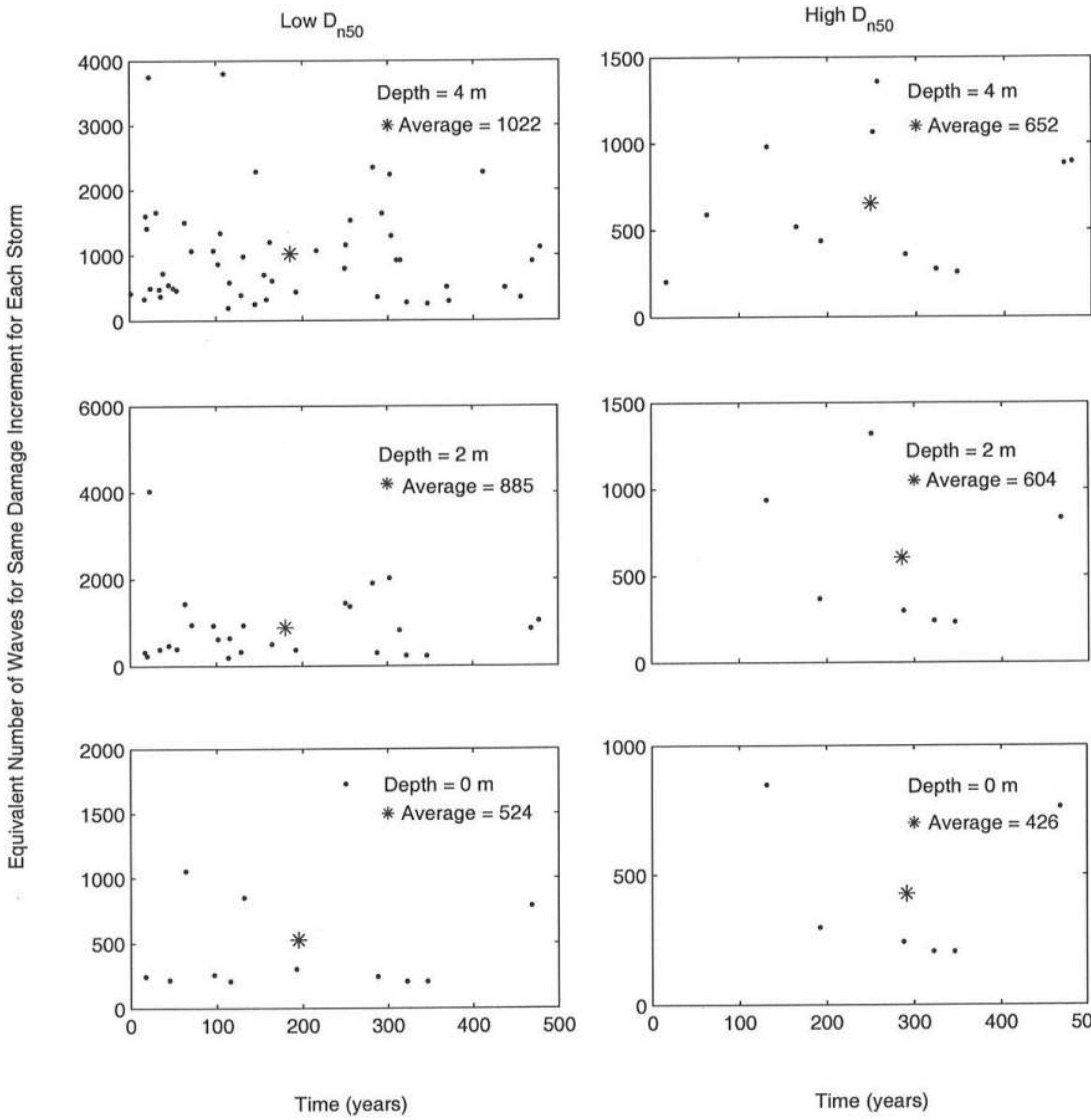

Figure 5.61: Equivalent Number of Waves to Cause the Same Damage Increment for Low and High Diameters D_{n50} on $1/40$ Slope at Locations of $d =$ 4, 2 and 0 m for $N500 = 5$.

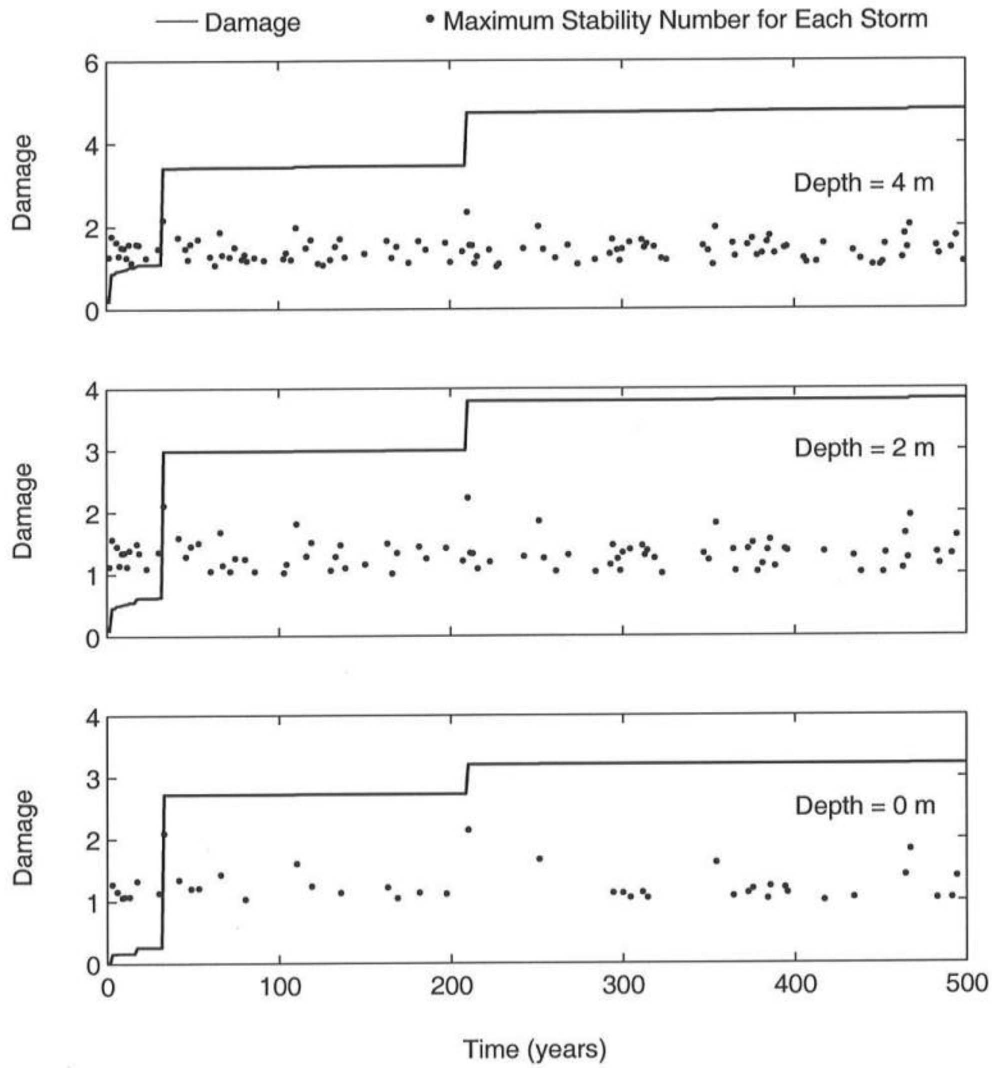

Figure 5.62: Damage Progression of Armor Layer with Low Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/40 Slope at Locations of $d = 4$, 2 and 0 m for $N500 = 6$.

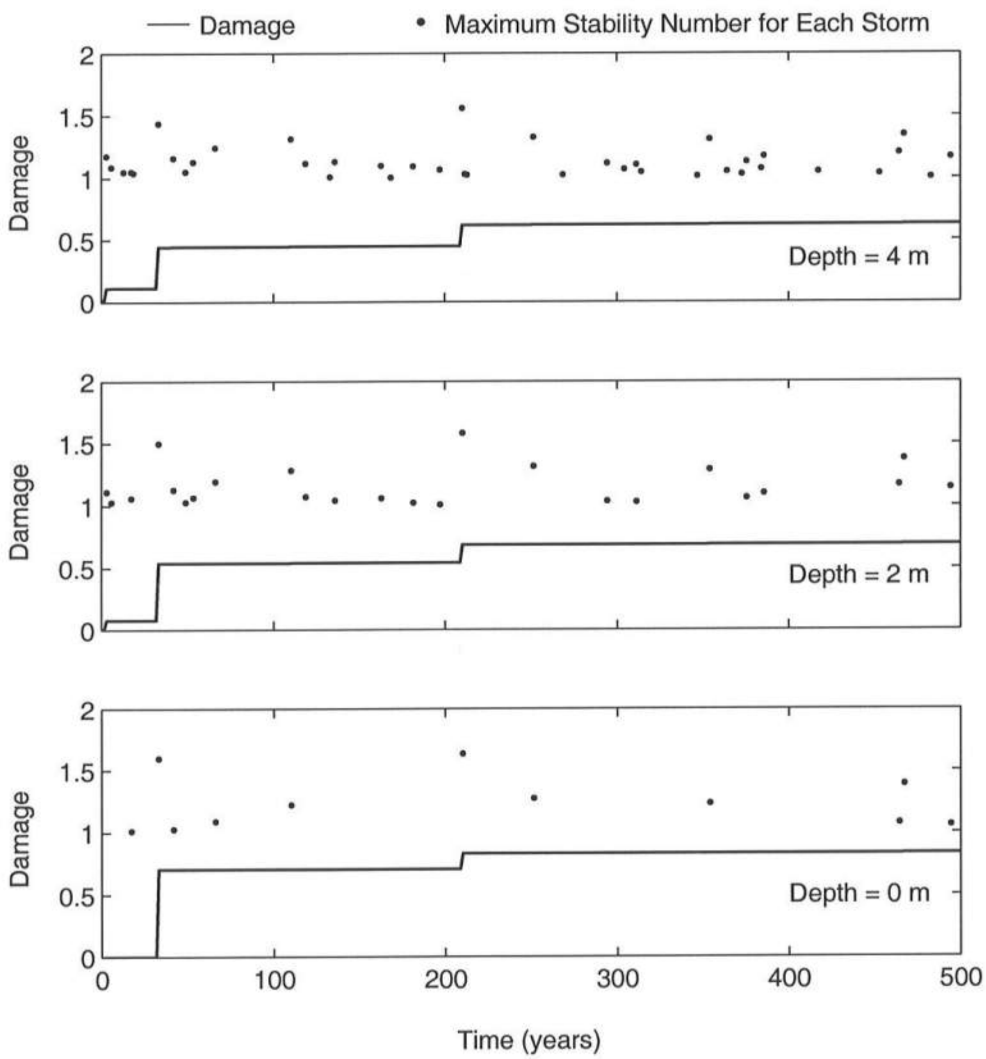

Figure 5.63: Damage Progression of Armor Layer with High Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/40 Slope at Locations of $d = 4$, 2 and 0 m for $N500 = 6$.

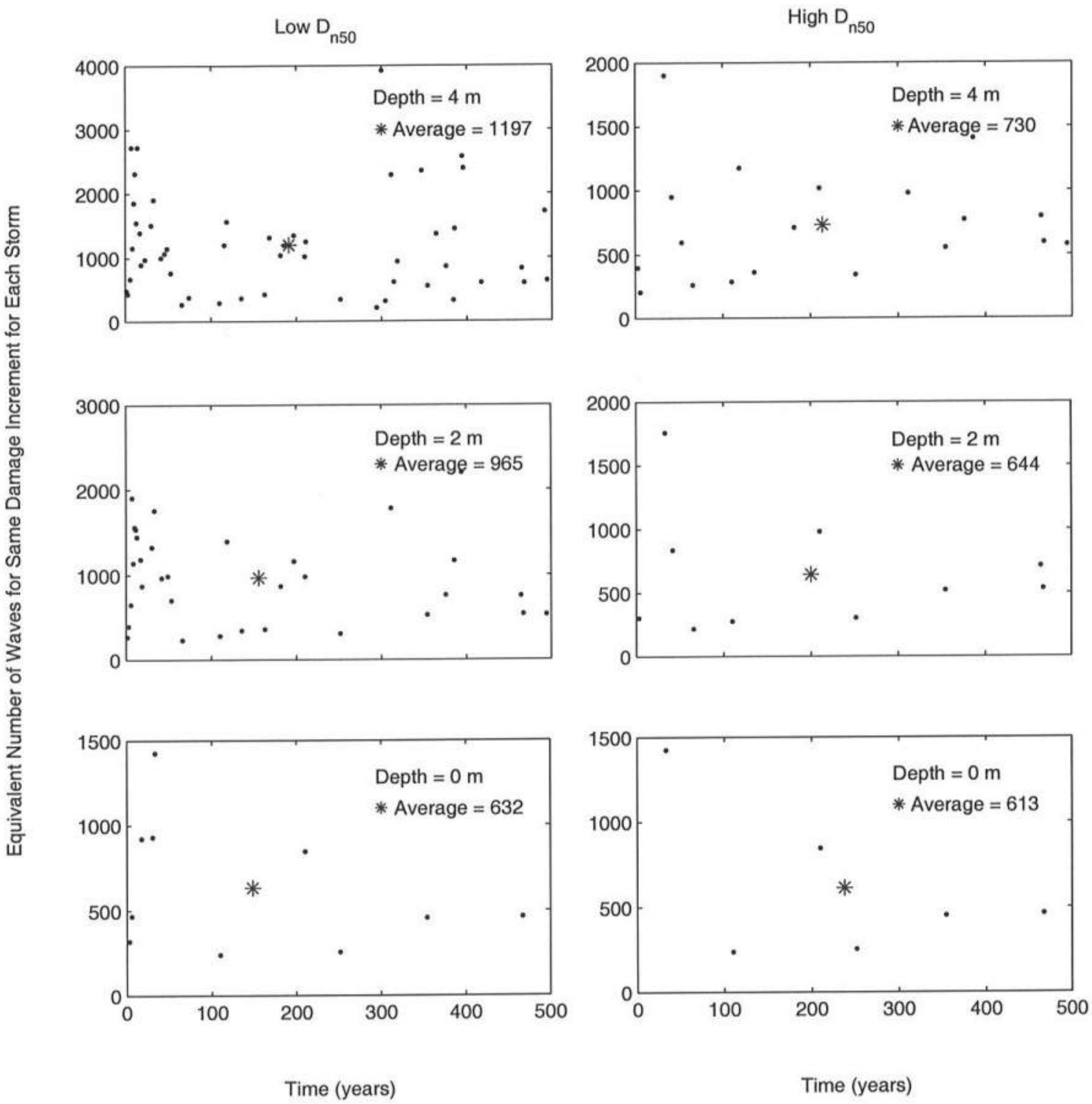

Figure 5.64: Equivalent Number of Waves to Cause the Same Damage Increment for Low and High Diameters D_{n50} on 1/40 Slope at Locations of $d =$ 4, 2 and 0 m for $N500 = 6$.

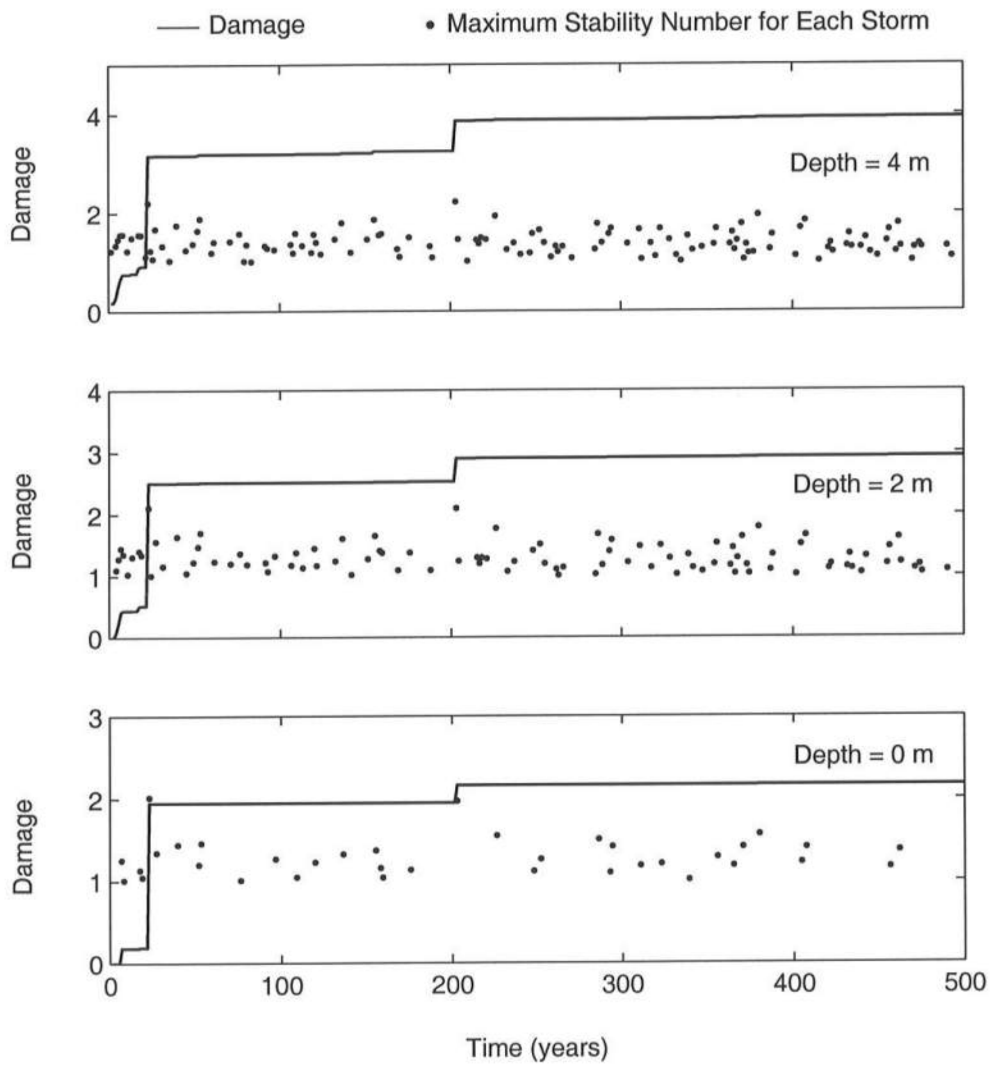

Figure 5.65: Damage Progression of Armor Layer with Low Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/40 Slope at Locations of d = 4, 2 and 0 m for $N500$ = 7.

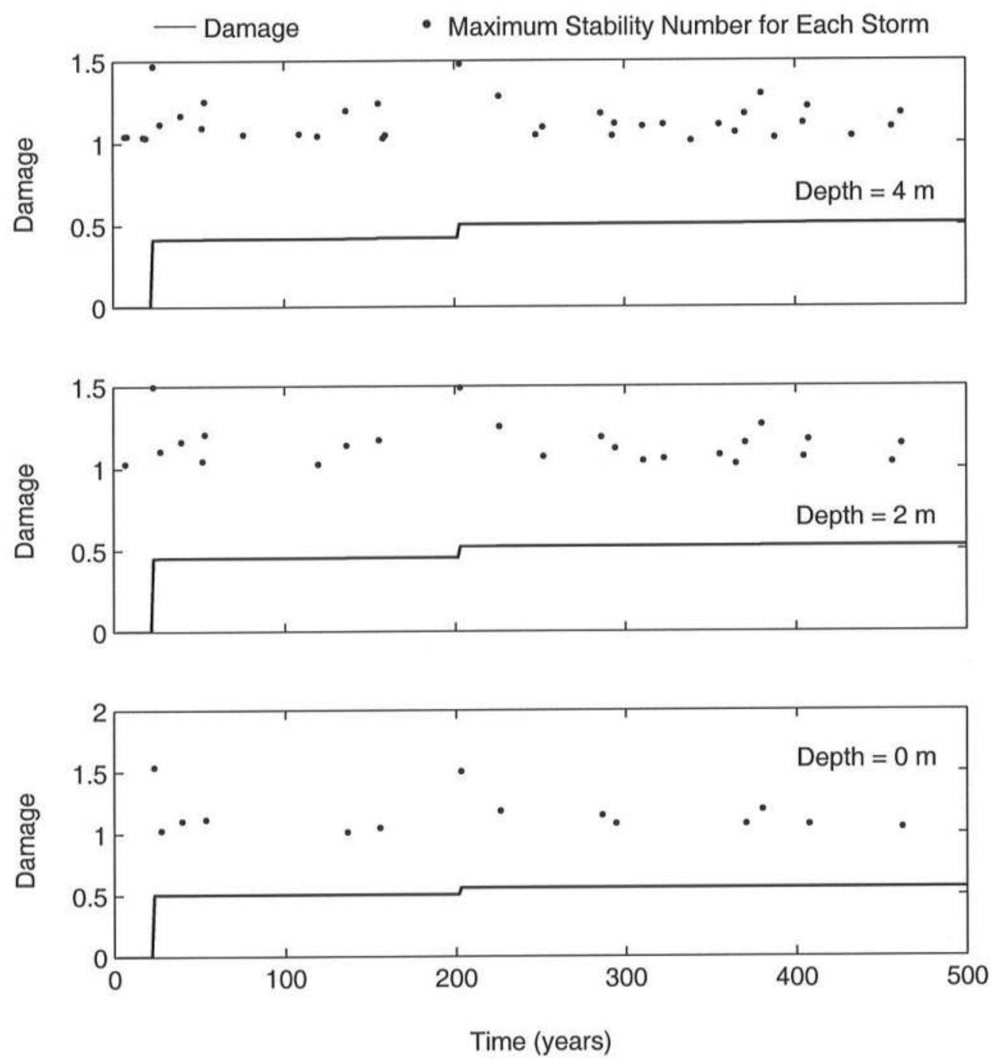

Figure 5.66: Damage Progression of Armor Layer with High Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/40 Slope at Locations of d = 4, 2 and 0 m for $N500$ = 7.

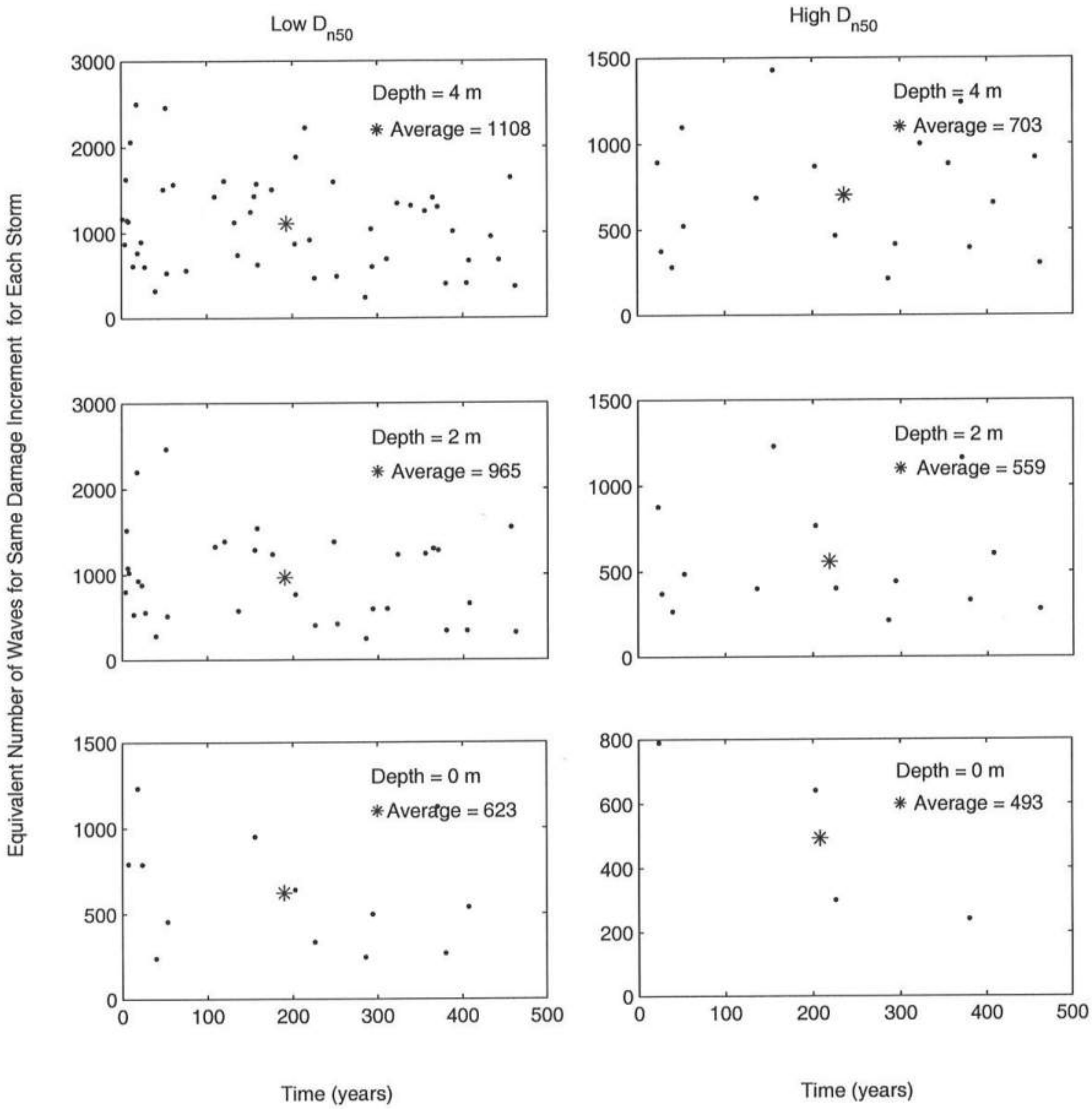

Figure 5.67: Equivalent Number of Waves to Cause the Same Damage Increment for Low and High Diameters D_{n50} on $1/40$ Slope at Locations of $d =$ 4, 2 and 0 m for $N500 = 7$.

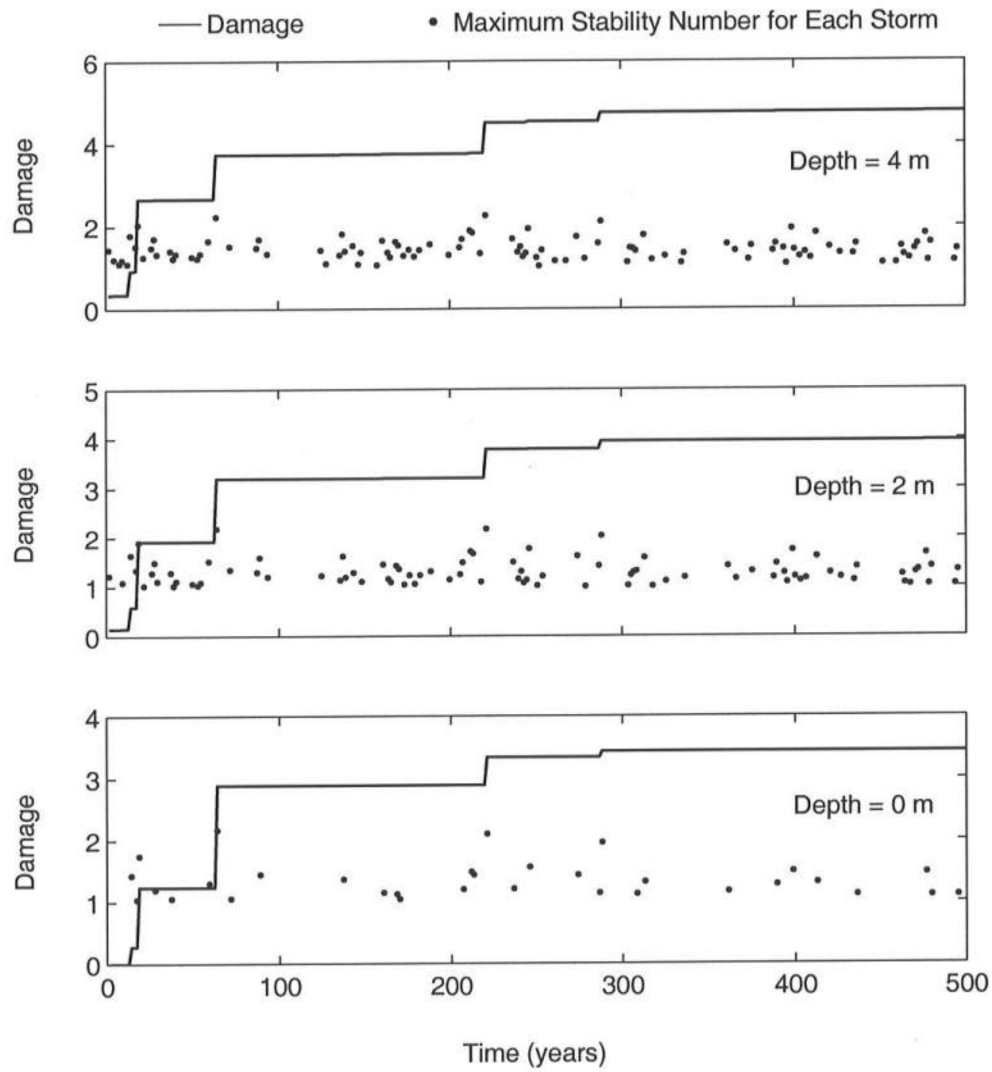

Figure 5.68: Damage Progression of Armor Layer with Low Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/40 Slope at Locations of $d = 4$, 2 and 0 m for $N500 = 8$.

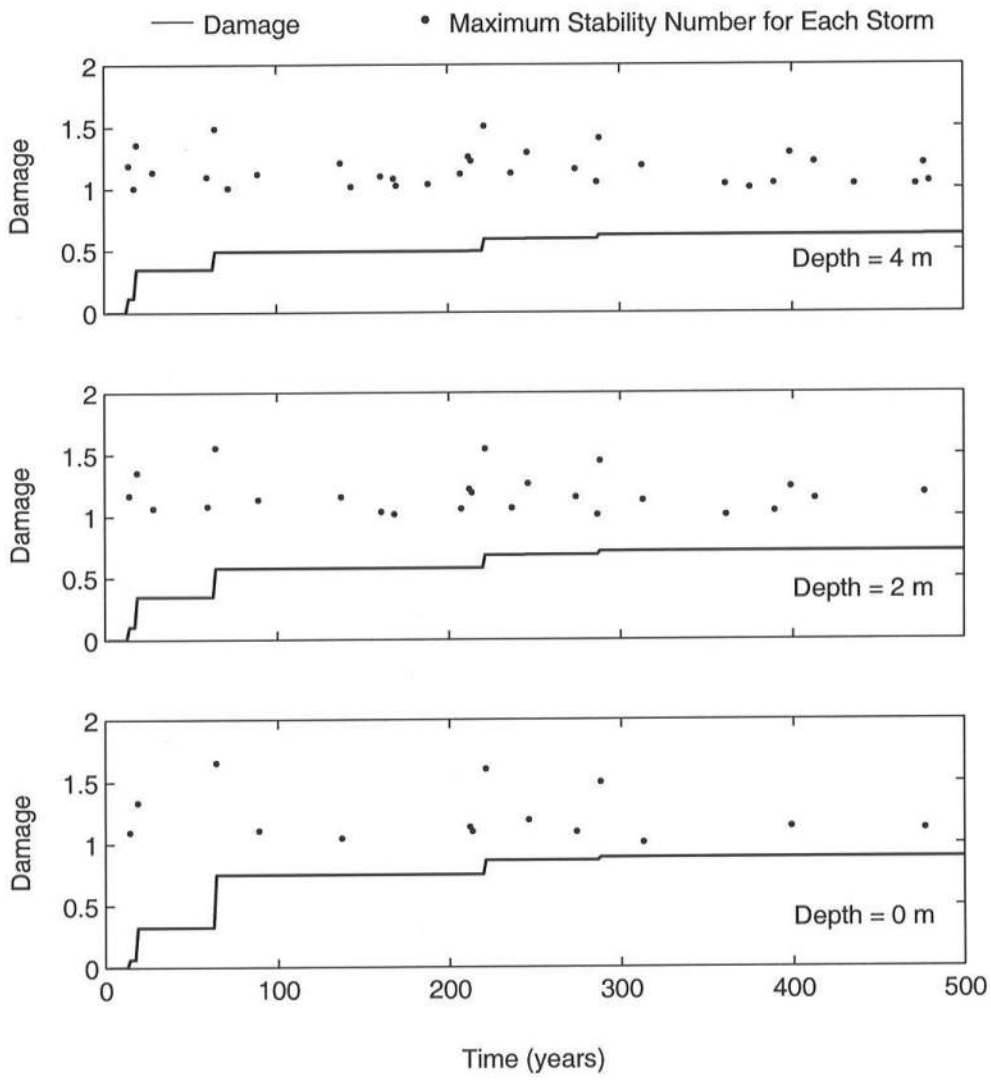

Figure 5.69: Damage Progression of Armor Layer with High Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/40 Slope at Locations of $d = 4$, 2 and 0 m for $N500 = 8$.

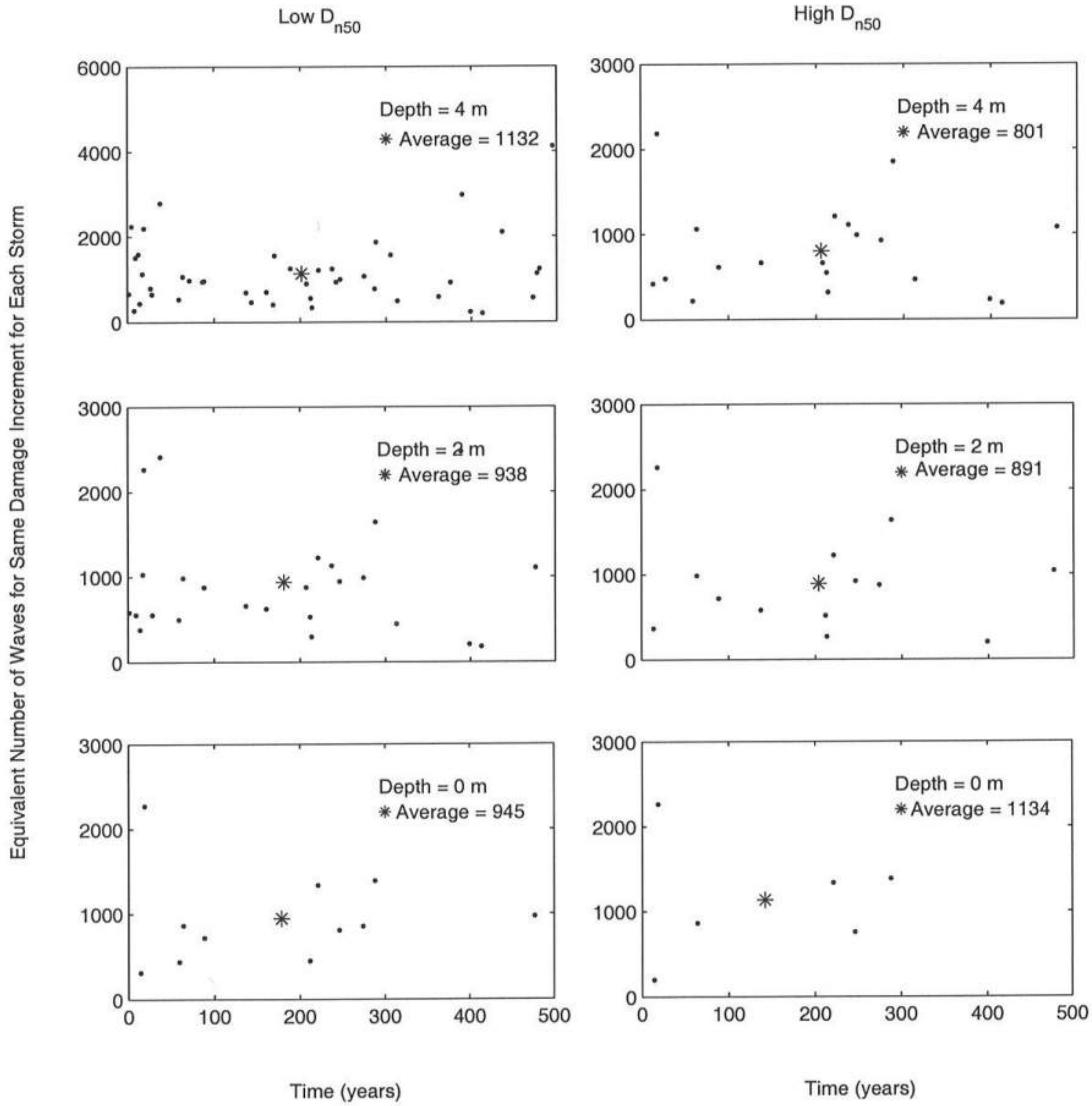

Figure 5.70: Equivalent Number of Waves to Cause the Same Damage Increment for Low and High Diameters D_{n50} on 1/40 Slope at Locations of $d =$ 4, 2 and 0 m for $N500 = 8$.

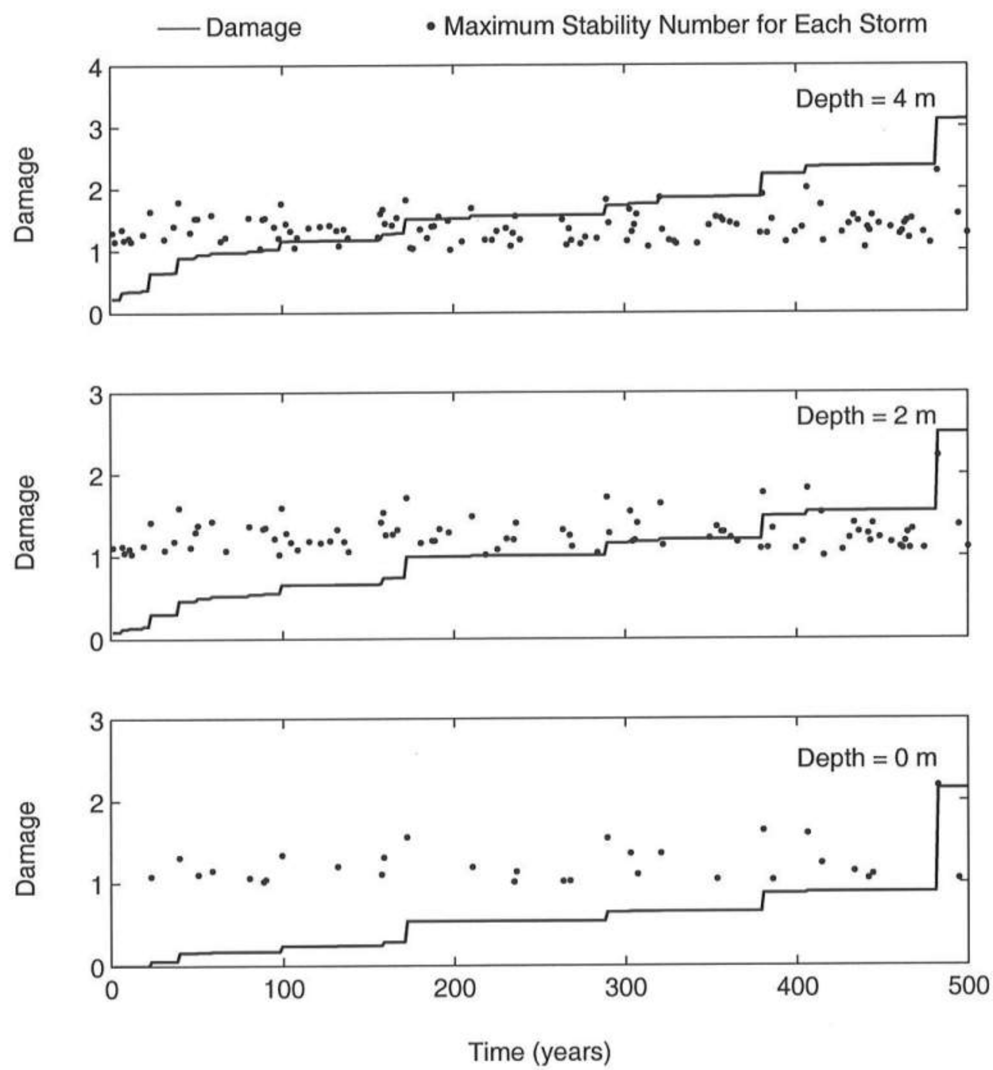

Figure 5.71: Damage Progression of Armor Layer with Low Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/40 Slope at Locations of $d = 4$, 2 and 0 m for $N500 = 9$.

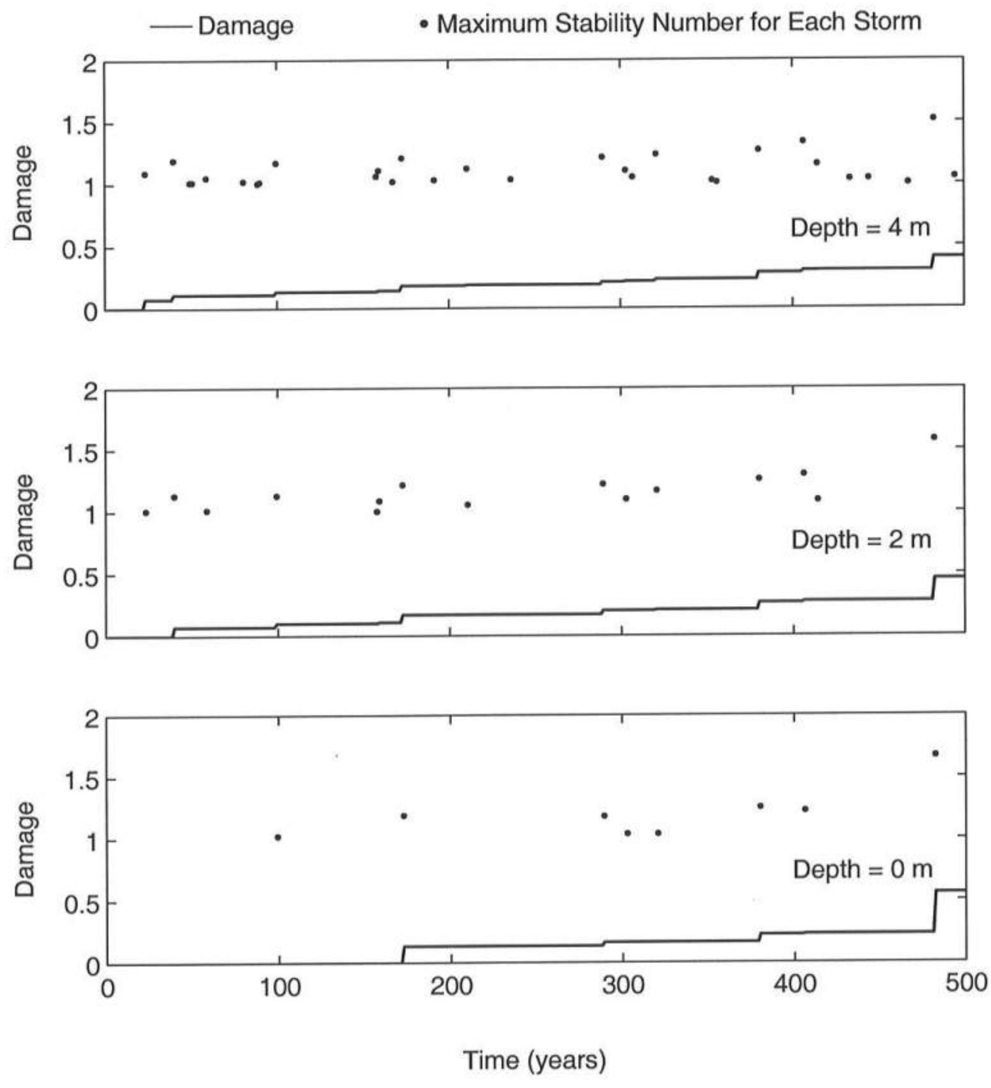

Figure 5.72: Damage Progression of Armor Layer with High Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/40 Slope at Locations of d = 4, 2 and 0 m for $N500 = 9$.

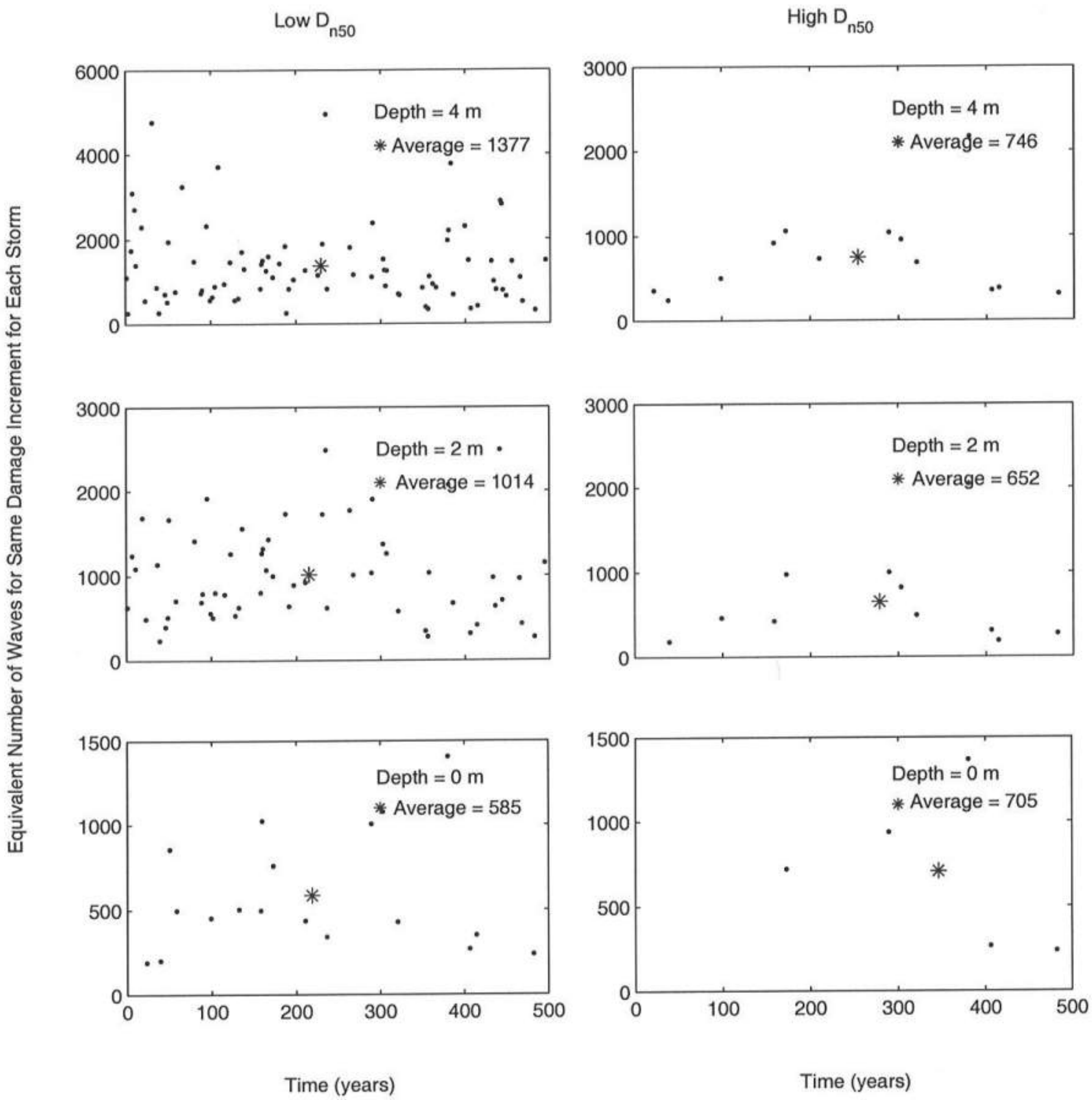

Figure 5.73: Equivalent Number of Waves to Cause the Same Damage Increment for Low and High Diameters D_{n50} on 1/40 Slope at Locations of $d =$ 4, 2 and 0 m for $N500 = 9$.

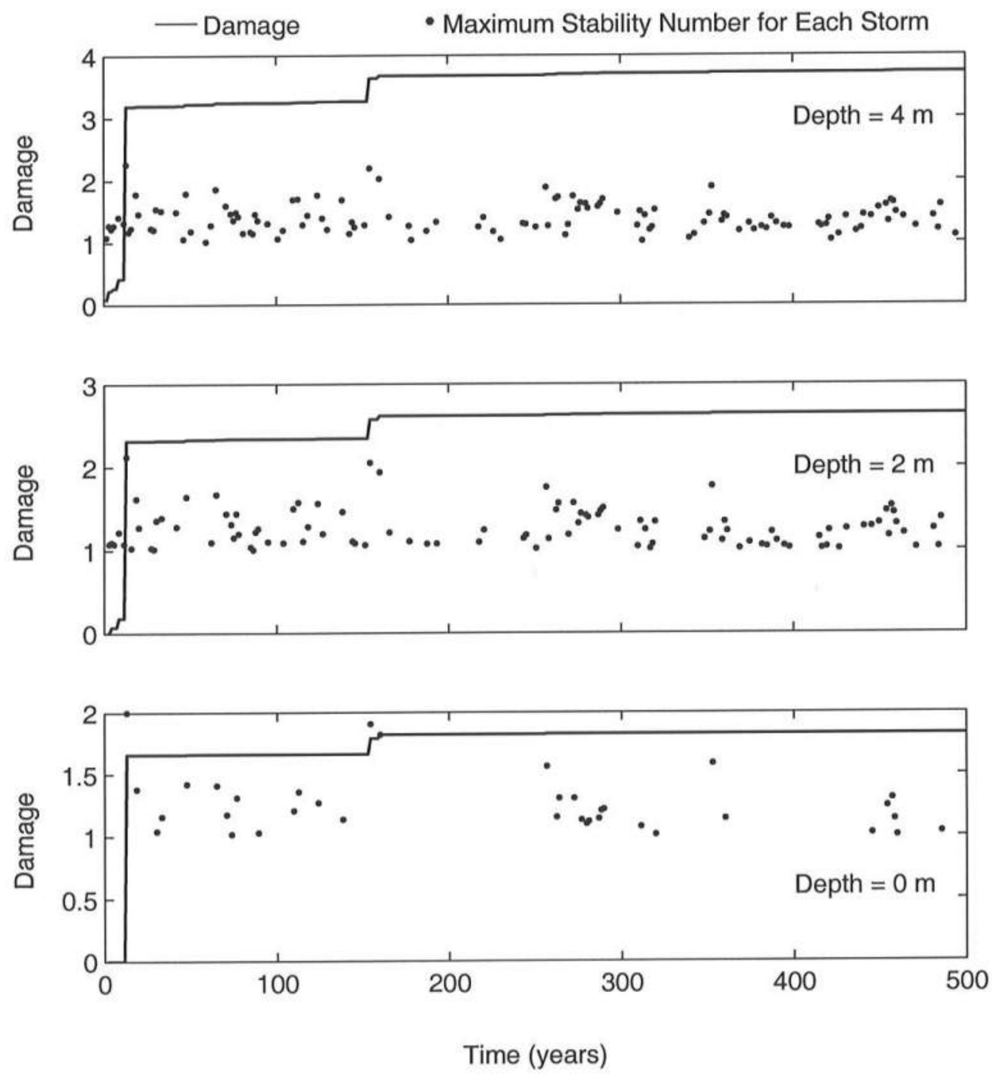

Figure 5.74: Damage Progression of Armor Layer with Low Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/40 Slope at Locations of $d = 4$, 2 and 0 m for $N500 = 10$.

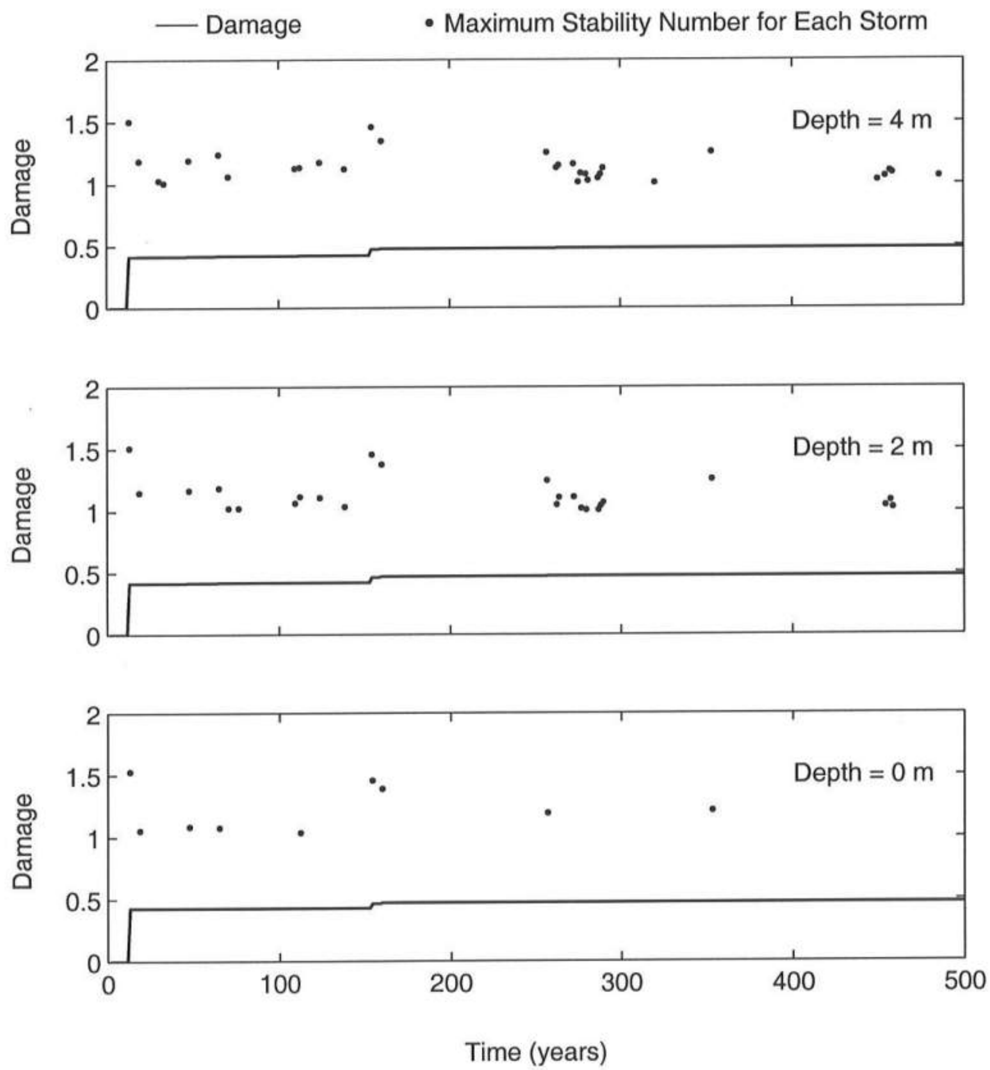

Figure 5.75: Damage Progression of Armor Layer with High Diameter D_{n50} and Maximum Stability Number N_{mo} for Each Storm on 1/40 Slope at Locations of $d = 4$, 2 and 0 m for $N500 = 10$.

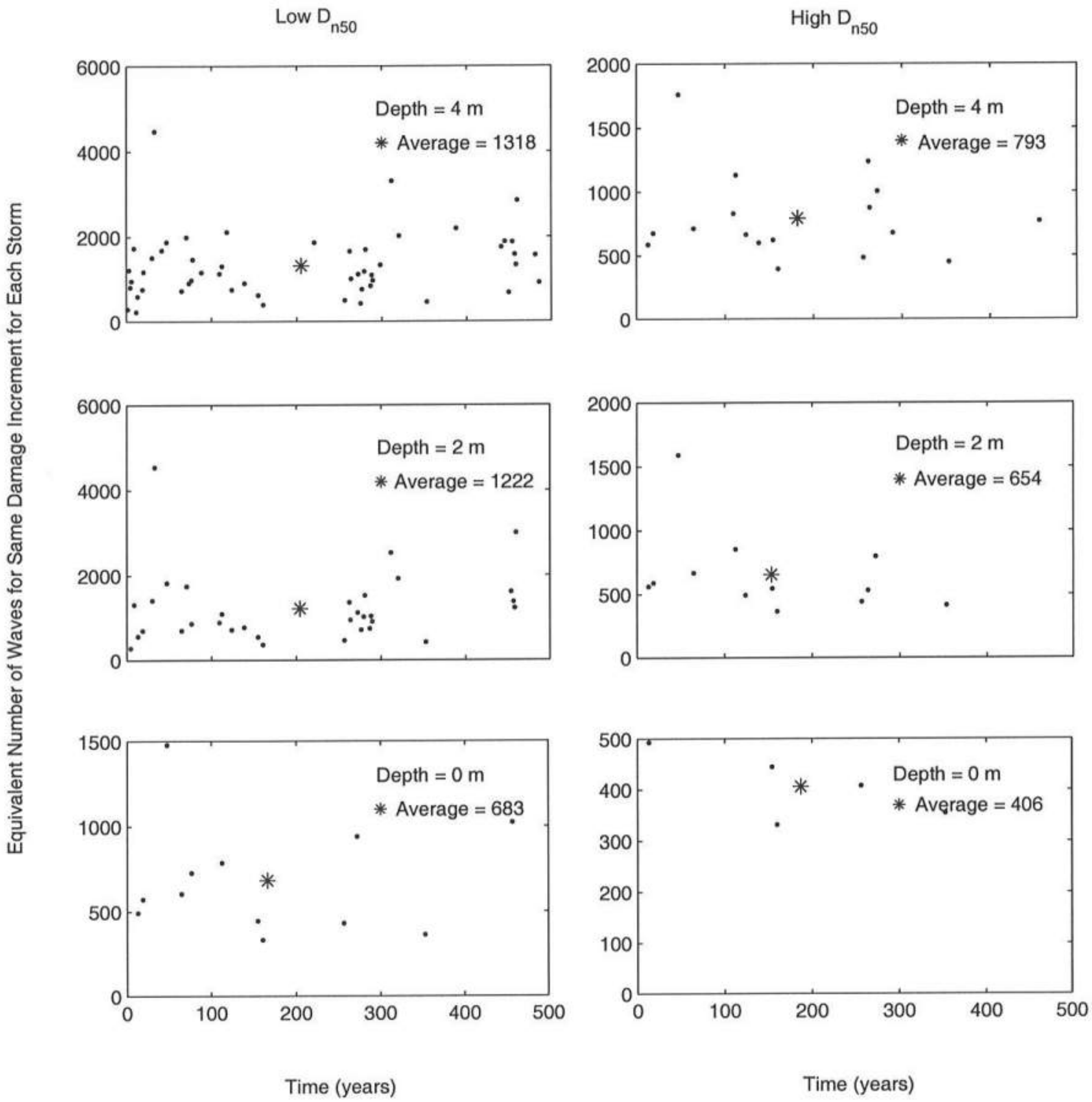

Figure 5.76: Equivalent Number of Waves to Cause the Same Damage Increment for Low and High Diameters D_{n50} on 1/40 Slope at Locations of $d =$ 4, 2 and 0 m for $N500 = 10$.

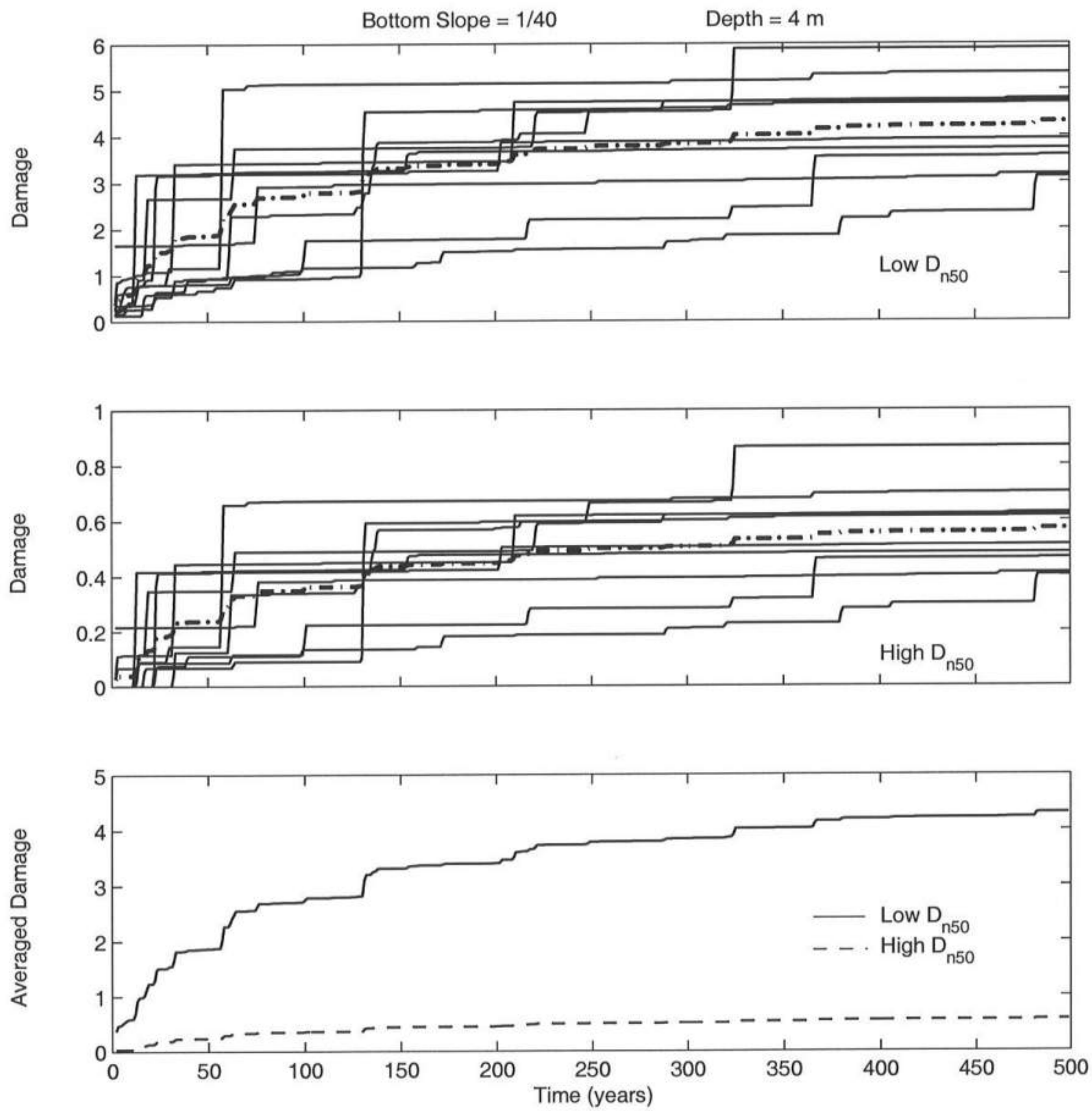

Figure 5.77: Damage Progression of Armor Layer with Low and High Diameters D_{n50} on 1/40 Slope at Location of d = 4 m for Each and Average of the Ten 500-yr Simulations: in the Top Two Panels, (—) Each Simulation; (-.-) Average.

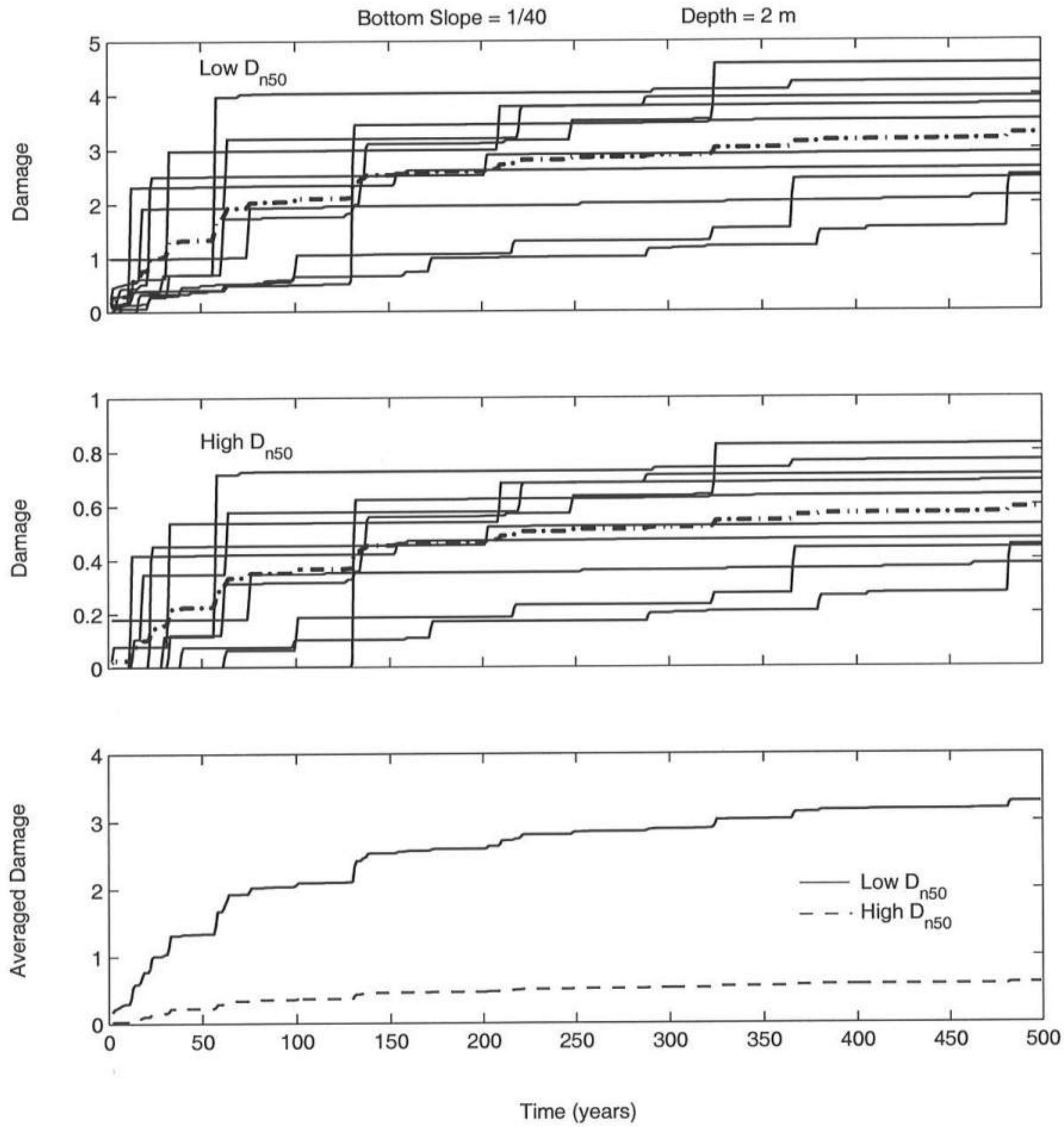

Figure 5.78: Damage Progression of Armor Layer with Low and High Diameters D_{n50} on 1/40 Slope at Location of $d = 2$ m for Each and Average of the Ten 500-yr Simulations: in the Top Two Panels, (—) Each Simulation; (-.-) Average.

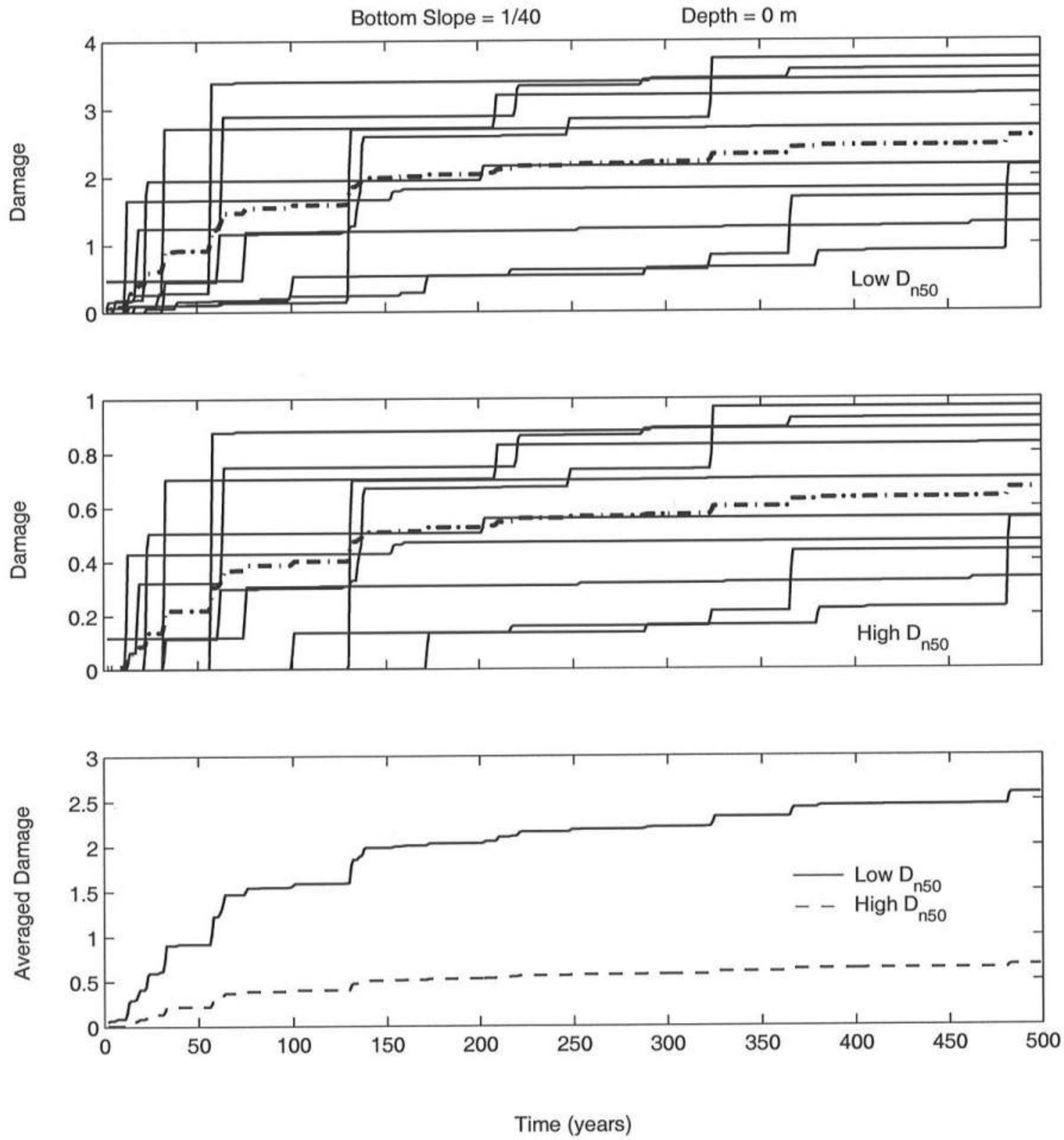

Figure 5.79: Damage Progression of Armor Layer with Low and High Diameters D_{n50} on $1/40$ Slope at Location of $d = 0$ m for Each and Average of the Ten 500-yr Simulations: in the Top Two Panels, (—) Each Simulation; (-.-) Average.

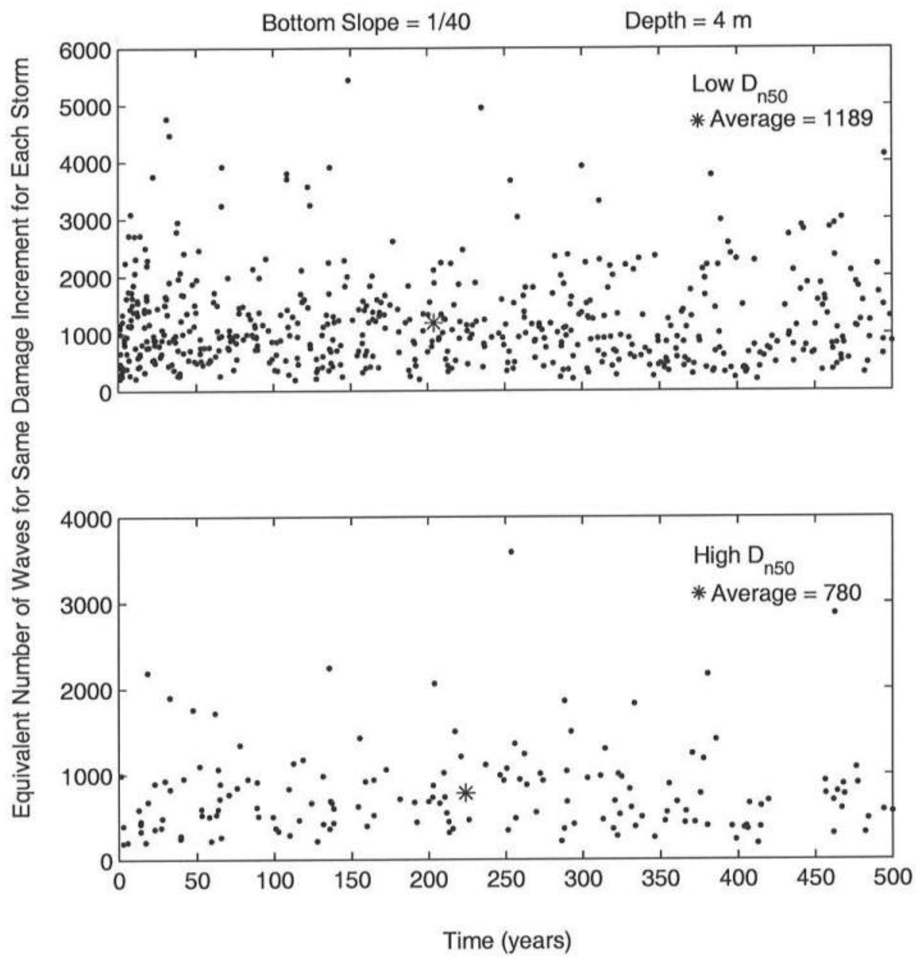

Figure 5.80: Equivalent Number of Waves to Cause the Same Damage Increment for Low and High Diameters D_{n50} on 1/40 Slope at Location of $d =$ 4 m for Ten 500-yr Simulations.

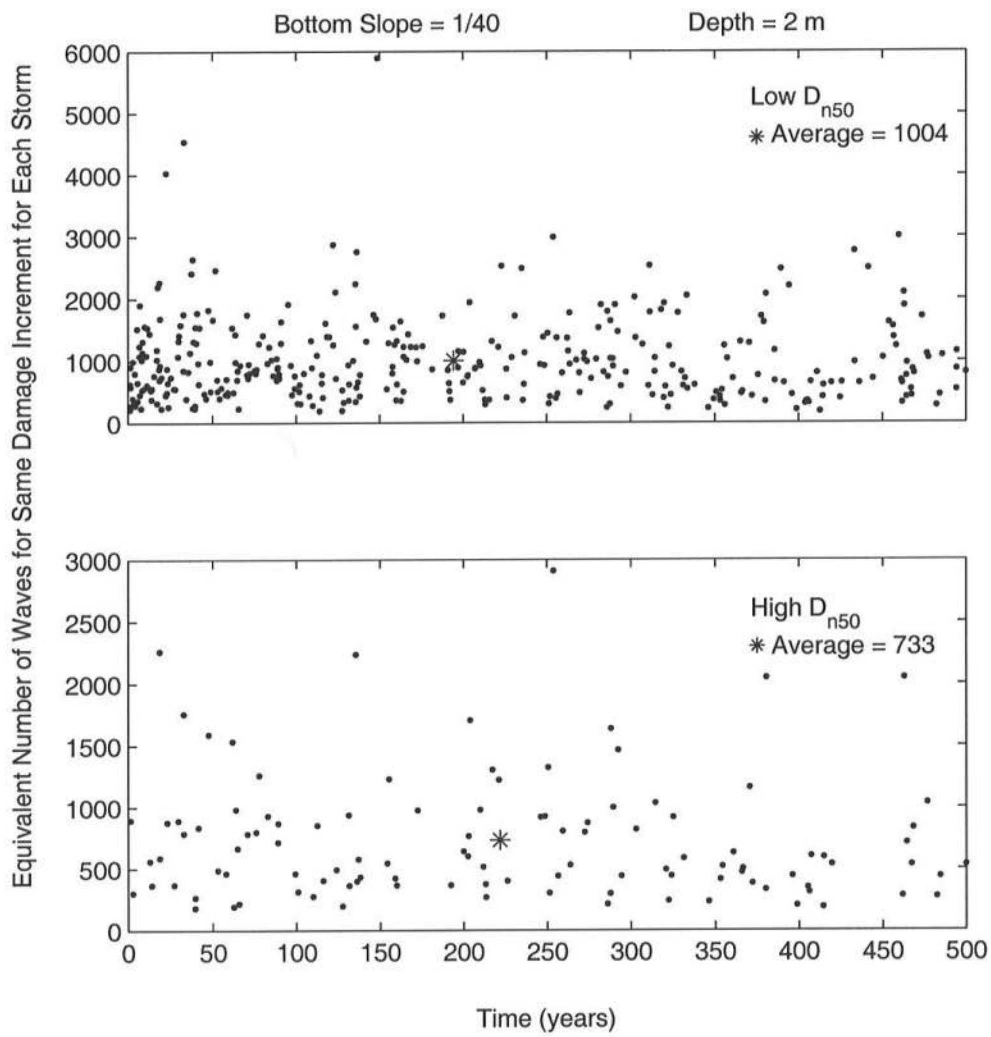

Figure 5.81: Equivalent Number of Waves to Cause the Same Damage Increment for Low and High Diameters D_{n50} on 1/40 Slope at Location of $d = 2$ m for Ten 500-yr Simulations.

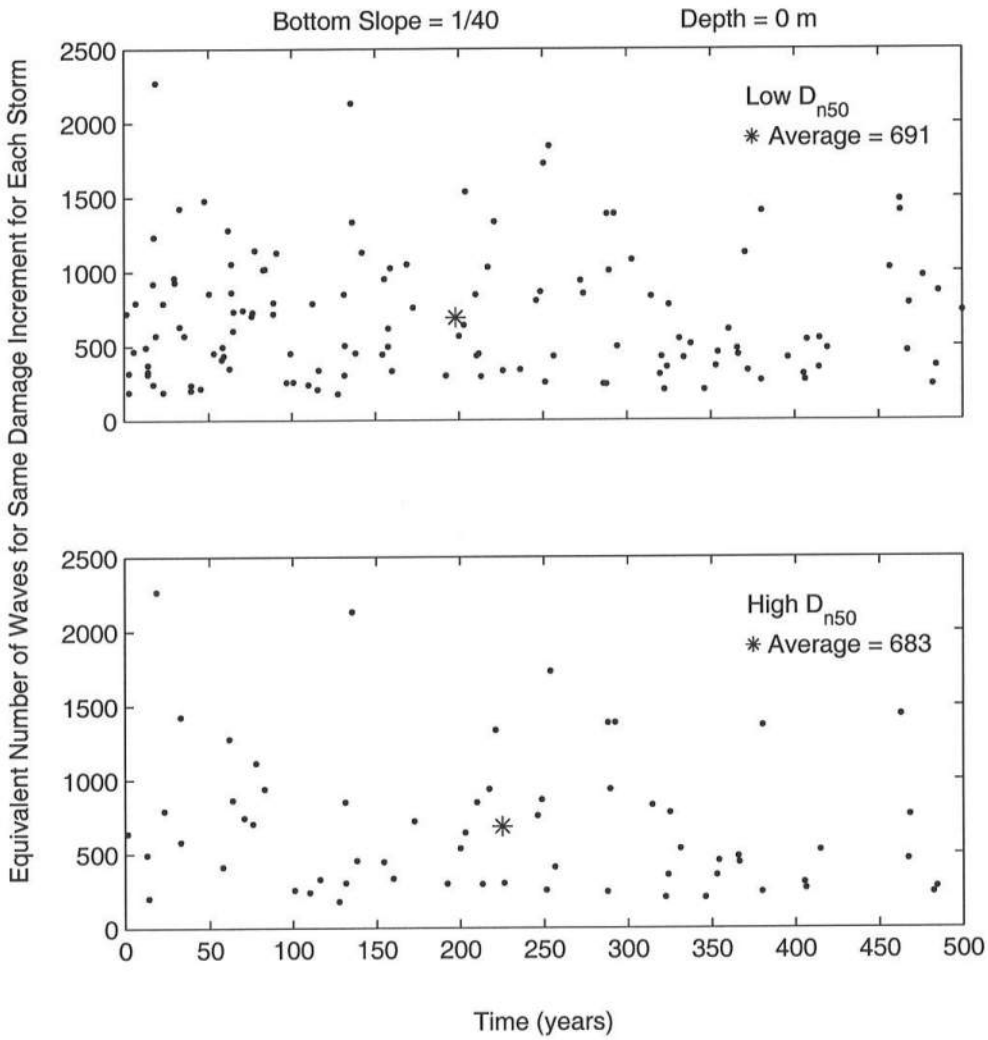

Figure 5.82: Equivalent Number of Waves to Cause the Same Damage Increment for Low and High Diameters D_{n50} on 1/40 Slope at Location of $d =$ 0 m for Ten 500-yr Simulations.

Chapter 6

SUMMARY AND CONCLUSIONS

Numerical models for storm surge, wind waves, irregular wave transformation, and wave overtopping have become more and more complex with little regard to the synthesis of various models required to solve actual coastal engineering problems. Such a synthesis is attempted here using simple models and formulas to simulate the virtual performance of rubble mound structures in shallow water under the combined storm tide and breaking waves of hurricanes for the duration of 500 years.

The crest height of the structure is designed conventionally for minor overtopping during the peak of a 100-yr storm. The variability of the computed overtopping rate for ten 500-yr simulations is one order of magnitude and consistent with the crude estimate of the allowable overtopping rate used for practical applications. The overtopping water volume during the entire duration of a storm is shown to be approximated by multiplying the maximum overtopping rate by an equivalent hurricane duration of about 3 hr.

The nominal diameter of the armor stone against the 100-yr storm is designed using the Hudson and Van der Meer formulas. The progression of damage to the armor layer computed using the formula of Melby and Kobayashi (2000) is caused episodically by several major storms with large wave heights but slows down as the damaged armor layer ages. The variability about the average damage progression for the ten 500-yr simulations is a factor of about two and relatively small because of the aging effect. The damage increment during a storm is shown to correspond to the damage caused by approximately 1,000 waves during the peak of a hurricane storm. The computed damage progression is used to assess whether the nominal armor diameter needs to be increased or can be reduced.

Finally, the virtual performance will need to be verified against the long-term performance monitored for real structures. However, such monitoring is rarely performed.

Relatedly, the virtual performance of rubble mound structures has been computed in this study under the assumption of alongshore uniformity. In the near future, the alongshore variability will need to be included in this virtual performance because damage on rubble mound structures was observed to occur locally. Melby and Kobayashi (1998a) conducted laboratory experiments on the progression and variability of damage on a conventional rubble mound breakwater and showed that the damage variability along the structure was significant even for uniform wave conditions because of the irregularity of placed stone along the structure. Furthermore, the damage variability and the variability of the armor layer thickness were shown to be important for the prediction of the occurrence of the localized failure of the armor layer. The damage variability model of Melby and Kobayashi (1998a) will need to be added to the numerical simulations of the armor layer damage progression developed in this study in order to investigate reasons why a rubble mound structure tends to be damaged locally, rather than uniformly along the structure. These numerical simulations may indicate the degree of importance of a localized thin armor layer (due to poor construction) and a localized large wave height for the localized failure of the conventional rubble mound.

BIBLIOGRAPHY

Battjes, J.A., and Groenendijk, H.A. (2000). "Wave height distributions on shallow foreshores." *Coast. Engrg.*, **40**, 161–182.

Battjes, J.A., and Janssen, J.P.F.M. (1978). "Energy loss and set-up due to breaking of random waves." *Proc. 16th Coast. Engrg. Conf.*, ASCE, 569–587.

Battjes, J.A., and Stive, M.J.F. (1985). "Calibration and verification of a dissipation model for random breaking waves." *J. Geophys. Res.*, **90**(C5), 9159–9167.

Benjamin, J.R., and Cornell, C.A. (1970). *Probability, statistics, and decision for civil engineers.* McGraw-Hill, New York.

Booij, N., Ris, R.C., and Holthuijsen, L.H. (1999). "A third-generation wave model for coastal regions. 1. Model description and validation." *J. Geophys. Res.*, **104**(C4), 7649–7666.

Cardone, V.J., Greenwood, C.V., and Greenwood, J.A.(1992). "Unified program for the specification of hurricane boundary layer winds over surfaces of specified roughness." *Rept. CERC-92-1*, USAE Waterways Experiment Station, Vicksburg, Miss.

Goda, Y. (1985). *Random seas and design of maritime structures.* Univ. of Tokyo Press, Tokyo, Japan.

Hanzawa, M., Sato, H., Takahasi, S., Shimosako, K., Takayama, T., and Tanimoto, K. (1996). "New stability formula for wave-dissipation concrete blocks covering horizontally composite breakwaters." *Proc. 25th Coast. Engrg. Conf.*, ASCE, 1665–1678.

Hudson, R.Y. (1959). "Laboratory investigation of rubble-mound breakwaters." *J. Wtrwy., Port, Coast. and Oc. Engrg.*, ASCE, **85**(3), 93–121.

Johnson, B.D., and Kobayashi, N. (1998). "Nonlinear time-averaged model in surf and swash zones." *Proc. 26th Cost. Engrg. Conf.*, ASCE, 2785–2798.

Johnson, B.D., and Kobayashi, N. (2000). "Free surface statistics and probabilities in surf zones on beaches." *Proc. 27th Coast. Engrg. Conf.*, ASCE, 1022–1035.

Kearney, P.G., and Kobayashi, N. (2000). "Time-averaged probabilistic model for irregular wave runup on coastal structures." *Proc. 27th Coast. Engrg. Conf.*, ASCE, 2004–2017.

Kobayashi, N., and Johnson, B.D. (1998). "Computer program CSHORE for predicting cross-shore transformation of irregular breaking waves." *Res. Rep. No. CACR-98-04*, Ctr. for Appl. Coast. Res., Univ. of Delaware, Newark, Del.

Kobayashi, N., Pozueta, B. and Melby, J.A. (2002). "Performance of coastal structures against sequences of hurricanes." *J. Wtrwy., Port, Coast. and Oc. Engrg.*, ASCE, (Submitted).

Kobayashi, N., and Raichle, A.W. (1994). "Irregular wave overtopping of revetments in surf zones." *J. Wtrwy., Port, Coast. and Oc. Engrg.*, ASCE, **120**(1), 56–73.

Kobayashi, N., and Wurjanto, A. (1992). "Irregular wave setup and run-up on beaches." *J. Wtrwy., Port, Coast. and Oc. Engrg.*, ASCE, **118**(4), 368–386.

Kriebel, D.L. (1982). *"Beach and dune response to hurricanes."* Master's thesis, Dept. of Civil Engrg., Univ. of Delaware, Newark, Del.

Kriebel, D.L., and Dean, R.G. (1984). "Beach and dune response to severe storms." *Proc. 19th Coast. Engrg. Conf.*, ASCE, 1584–1599.

Melby, J.A., and Kobayashi, N. (1998a). "Progression and variability of damage on rubble mound breakwaters." *J. Wtrwy., Port, Coast. and Oc. Engrg.*, ASCE, **124**(6), 286–294.

Melby, J.A., and Kobayashi, N. (1998b). "Damage progression on breakwaters." *Proc., 26th Coast. Engrg. Conf.*, ASCE, 1884–1897.

Melby, J.A., and Kobayashi, N. (1999). "Damage progression and variability on breakwater trunks." *Proc., Coastal Structures' 99.* Balkema, Rotterdam, 309–315.

Melby, J.A. and Kobayashi, N. (2000). "Damage development on stone-armored rubble mounds." *Proc. 27th Coast. Engrg. Conf.*, ASCE, 1022–1035.

Pozueta, B., Kobayashi, N. and Melby, J.A. (2002). "Monte Carlo simulation of cumulative damage on rubble mound breakwaters." *Proc. 28th Coast. Engrg. Conf.*, ASCE, (In press).

Raubenheimer, B., Guza, R.T., and Elgar, S. (1996). "Wave transformation across the inner surf zone." *J. Geophys. Res.*, **101**(C10), 25,589–25,597.

Raubenheimer, B., Guza, R.T., and Elgar, S. (2001). "Field observations of wave-driven setdown and setup." *J. Geophys. Res.*, **106**(C3), 4629–4638.

Ris, R.C., Holthuijsen, L.H., and Booij, N. (1999). "A third-generation model for coastal regions. 2. Verification." *J. Geophys. Res.*, **104**(C4), 7667–7681.

Sarpkaya, T., and Isaacson, M. (1981). *Mechanics of wave forces on offshore structures.* Van Nostrand Reinhold, New York, NY.

Scheffner, N.W., Borgman, L.E., and Mark, D.J. (1996). "Empirical simulation technique based on storm surge frequency analyses." *J. Wtrwy., Port, Coast. and Oc. Engrg.*, ASCE, **122**(2), 93–101.

Shore Protection Manual. (1977). Coast. Engrg. Res. Ctr., U.S. Government Printing Office, Washington, D.C.

Shore Protection Manual. (1984). Coast. Engrg. Res. Ctr., U.S. Government Printing Office, Washington, D.C.

Smith, W.G., Kobayashi, N., and Kaku, S. (1992). "Profile changes of rock slopes by irregular waves." *Proc. 23rd Coast. Engrg. Conf.*, ASCE, 1559–1572.

Van der Meer, J.W. (1988a). "Deterministic and probabilistic design of breakwater armor layers." *J. Wtrwy., Port, Coast. and Oc. Engrg.*, ASCE, **114**(1), 66–80.

Van der Meer, J.W. (1988b). "Rock slopes and gravel beaches under wave attack." *Comm. No. 396*, Delft Hydr. Res., Emmeloord, The Netherlands.

Van der Meer, J.W., and Janssen, J.P.F.M. (1995). "Wave run-up and wave overtopping at dikes." *Wave forces on inclined and vertical wall structures*, ASCE, 1–27.

Westerink, J.J., Luettich, A.M., Baptista, A.M., Scheffner, N.W., and Farrar, P. (1992). "Tide and storm surge predictions using finite element model." *J. Hydr. Engrg.*, ASCE, **118**(10), 1373–1390.

Appendix A

COMPUTER PROGRAM CYCLONE

```
C
C
C    #######   ##    ##    #######   ##          ######    ###      ##   #######
C   ##          ##  ##    ##         ##         ##    ##   ####     ##   ##
C   ##           ####     ##         ##         ##    ##   ## ##    ##   ##
C   ##            ##      ##         ##         ##    ##   ##  ##   ##   ######
C   ##            ##      ##         ##         ##    ##   ##   ## ##   ##
C   ##            ##      ##         ##         ##    ##   ##    ####   ##
C    #######      ##       #######   ########    ######    ##     ###   #######
C
C
C
C*****************************************************************************
C*                                                                           *
C*                    HURRICANE WIND, STORM TIDE AND WAVES                   *
C*                                                                           *
C*****************************************************************************
C*                                                                           *
C*                   NOBUHISA KOBAYASHI AND BEATRIZ POZUETA                  *
C*                                                                           *
C*                    CENTER FOR APPLIED COASTAL RESEARCH                    *
C*                                                                           *
C*                                                                           *
C*              University of Delaware, Newark, Delaware 19716               *
C*                                                                           *
C*                               October 2001                                *
C*                                                                           *
C*****************************************************************************
C*                                                                           *
C*        MONTE CARLO SIMULATION OF TIME SERIES OF STORM TIDE                *
C*        (SURGE + ASTRONOMICAL TIDE) AND SIGNIFICANT WAVE HEIGHT            *
C*        AND PERIOD DUE TO HURRICANES AND TROPICAL STORMS GENERATED         *
C*        NUMERICALLY ON THE BASIS OF THE POISSON PROCESS AND SITE-SPECIFIC  *
C*        STORM DATA FOR THE DURATION OF 500 YEARS                           *
C*                                                                           *
C*****************************************************************************
C*00******************  MAIN PROGRAM  ****************************************
C
C        MAIN PROGRAM OPENS INPUT AND OUTPUT FILES,GENERATES STORMS
C        AND STORE COMPUTED TIME SERIES OF STORM TIDE AND SIGNIFICANT
```

```
C         WAVE HEIGHT AND PERIOD AT THE MOST LANDWARD NODE
C
C*****************************************************************************


      PROGRAM CYCLONE

      COMMON/TIMESE/NTIMEL,STTIDE(200),SWAVEH(200),SWAVET(200)
      DIMENSION DPTH(500),NYNS(6)
      DIMENSION NUMBST(500),SMAXST(500),SMAXWH(500),SMAXWT(500)
      REAL LAMDA
      CHARACTER*10 FINPUT,FSAVEA(10),FSAVEB(10)

      DATA FSAVEA/
     1'CYCLONEA01','CYCLONEA02','CYCLONEA03','CYCLONEA04',
     2'CYCLONEA05','CYCLONEA06','CYCLONEA07','CYCLONEA08',
     3'CYCLONEA09','CYCLONEA10'/
      DATA FSAVEB/
     1'CYCLONEB01','CYCLONEB02','CYCLONEB03','CYCLONEB04',
     2'CYCLONEB05','CYCLONEB06','CYCLONEB07','CYCLONEB08',
     3'CYCLONEB09','CYCLONEB10'/


C-----INPUT AND READ INPUT FILE FOR WATER DEPTHS------------------------------

      WRITE(*,*)
     1 'SPECIFY INTEGER N500 FOR N500-th SIMULATION OF 500 YEARS'

      READ(*,*) N500

      IF(N500.LT.1.OR.N500.GT.10) WRITE(*,*)
     1 'N500 MUST BE IN THE RANGE 1-10'

      WRITE (*,*) 'NAME OF INPUT FILE FOR WATER DEPTHS'
      READ(*,5000) FINPUT

 5000 FORMAT(A10)

C-----OPEN INPUT FILE AND READ NOFF=NUMBER OF NODES AND DPTH(N)=WATER DEPTH--
C     (IN METERS) WITH N=1 FOR THE MOST LANDWARD NODE AND N=NOFF FOR THE MOST
C     SEAWARD NODE IN VERY DEEP WATER IN THE LATITUDE LAMDA (IN DEGREES, + IN
```

```
C     NORTHERN HEMISPHERE)

      OPEN(UNIT=2,FILE=FINPUT,STATUS='OLD',ACCESS='SEQUENTIAL')
      READ(2,5001) NOFF,LAMDA

      DO 999 N=1,NOFF
         READ(2,5002) DPTH(N)
         WRITE(*,*) DPTH(N)
         DPTH(N)=3.281*DPTH(N)

 999  CONTINUE

 5001 FORMAT(I3,F5.1)
 5002 FORMAT(F8.2)

C-----------------------------------------------------------------------
C
C     NOTE THAT WATER DEPTHS IN FEET ARE USED IN THE SUBROUTINES DEVELOPED
C     ORIGINALLY BY DAVE KRIEBEL, USING 1977SPM
C     THE FOLLOWING COMPUTATIONAL PARAMETERS ARE SPECIFIED AS STANDARD VALUES
C     (CAN BE CHANGED)
C     PN=STANDARD SEA-LEVEL ATMOSPHERIC PRESSURE IN INCHES OF MERCURY
C     DT=TIME STEP SIZE IN HOURS FOR TIME MARCHING COMPUTATION
C     DX=DY=HORIZONTAL NODAL SPACING IN NAUTICAL MILES FOR DEPTH(N)
C     IMAX=JMAX=INTEGERS USED TO LIMIT THE DOMAIN OF THE STORM CENTER NODE
C               (IO,JO) SUCH THAT IO AND JO IN THE RANGE OF (-IMAX) TO IMAX
C               AND (-JMAX) TO JMAX
C
C-----------------------------------------------------------------------

      PN=29.92
      DT=0.5
      DX=5.0
      DY=5.0
      IMAX=NOFF
      JMAX=NOFF


C-----OPEN FOUR OUTPUT FILES--------------------------------------------

      OPEN(UNIT=3,FILE=FSAVEA(N500),STATUS='NEW',ACCESS='SEQUENTIAL')
```

```
      OPEN(UNIT=4,FILE=FSAVEB(N500),STATUS='NEW',ACCESS='SEQUENTIAL')
      OPEN(UNIT=5,FILE='OUTCYCLONE',STATUS='UNKNOWN',ACCESS=
     1      'SEQUENTIAL')
      OPEN(UNIT=7,FILE='CYCLONEDATA',STATUS='OLD',ACCESS=
     1      'APPEND')

C-----SPECIFY NSEED FOR RANDOM NUMBER GENERATOR FOR N500-th 500-YEAR SIMULATION

      NSEED=1000+100*N500

      IF(N500.EQ.10) NSEED=NSEED+100

C*****DO LOOP FOR 500-YEAR SIMULATION*******************************************

      NSTORM=0
      DO 1000 NYEAR=1,500



C-----RANDOMLY SELECT NUMBST(NYEAR)=NUMBER OF STORMS FOR NYEAR-th YEAR---------
C     USING SUBR.01 NUMBER


         CALL NUMBER(NSEED,NOSTOM)
         NUMBST(NYEAR)=NOSTOM

C-----IF NOSTOM IS NOT A POSITIVE NUMBER, NO STORM OCCURS FOR NYEAR-th YEAR----

         IF(NOSTOM.LT.1) GO TO 1000

C-----INTEGER NSTORM IS USED TO IDENTIFY EACH STORM----------------------------

         DO 200 M=1,NOSTOM
            NSTORM=NSTORM+1


C-----CALL SUBR.02 STORM1 TO COMPUTE THE TIME SERIES OF STORM TIDE AND--------
C     SIGNIFICANT WAVE HEIGHT AND PERIOD

            CALL STORM1(NSEED,SS,WH1,WT1,PN,DT,DX,DY,NOFF,
     1                  DPTH,IMAX,JMAX,LAMDA)
```

```
C-----STORE THE COMPUTED TIME SERIES OF STORM TIDE, STTIDE(N),SIGNIFICANT------
C     WAVE HEIGHT, SWAVEH(N), AND SIGNIFICANT WAVE PERIOD, SWAVET(N), WITH
C     N=1,2,...,NTIMEL, AT THE MOST LANDWARD NODE WHERE THESE VARIABLES
C     ARE TRANSFERRED THROUGH COMMON/TIMESE

 301        FORMAT(2I3)
 302        FORMAT(4F7.2)

            WRITE(4,301) NSTORM,NTIMEL

            DO 300 N=1,NTIMEL
               TIME=(N-1)*DT

C-----CONVERT STTIDE(FT) AND SWAVEH(FT) BACK TO STTIDE(M) AND SWAVEH(M)--------
C     WHERE TIME(HR) AND SWAVET(SEC)

               STTIDE(N)=0.3048*STTIDE(N)
               SWAVEH(N)=0.3048*SWAVEH(N)
               WRITE(4,302) TIME,STTIDE(N),SWAVEH(N),SWAVET(N)

 300        CONTINUE
C
C-----STORE THE MAXIMUM STORM TIDE, SS(M), SIGNIFICANT WAVE HEIGHT, WH1(M)-----
C     AND SIGNIFICANT WAVE PERIOD, WT1(SEC), FOR NSTORM-th STORM IN THE
C     DIMENSIONS OF SMAXST, SMAXWH AND SMAXWT, RESPECTIVELY

            SMAXST(NSTORM)=0.3048*SS
            SMAXWH(NSTORM)=0.3048*WH1
            SMAXWT(NSTORM)=WT1

C-----END OF NSTORM-th STORM---------------------------------------------------

 200     CONTINUE

 1000 CONTINUE


C*****END OF 500-YEAR SIMULATION**********************************************

      CLOSE(UNIT=4)
```

```
C-----IF THE SIMULATION IS SUCCESFUL, NYEAR=500 AND NSTORM=NUMBER OF STORMS----
C     DURING THE 500 YEARS

      NYEAR=500

C-----STORE THE SUMMARY OF THE SIMULATED RESULTS------------------------------

 401  FORMAT(2I4,F8.2)
 402  FORMAT(25I2)
 403  FORMAT(I3,3F7.2)

      WRITE(3,401) NYEAR,NSTORM,DPTH(1)*0.3048
      WRITE(3,402) (NUMBST(N),N=1,NYEAR)

      DO 400 N=1,NSTORM
         WRITE(3,403) N,SMAXST(N),SMAXWH(N),SMAXWT(N)

 400  CONTINUE

      CLOSE(UNIT=3)


C-----FIND NYNS(M)=NUMBER OF YEARS WITH (M-1) STORMS

      DO 500 M=1,6
         NYNS(M)=0
         M1=M-1
         DO 510 N=1,NYEAR
            IF(NUMBST(N).EQ.M1) NYNS(M)=NYNS(M)+1

 510     CONTINUE
 500  CONTINUE

C-----RANK SMAXST(N),SMAXWH(N) AND SMAXWT(N) INTO DESCENDING NUMERICAL---------
C     ORDER USING SUBR.19 SHELL

      CALL SHELL(NSTORM,SMAXST)
      CALL SHELL(NSTORM,SMAXWH)
      CALL SHELL(NSTORM,SMAXWT)
```

```
C-----STORE THE ESSENTIAL OUTPUT FOR N500-th SIMULATION IN FILE 'OUTCYCLONE'---
C     FOR SUBSEQUENT PRINTING AND PLOTTING

      WRITE(5,600) NYEAR,N500,NSTORM

 600  FORMAT(/I3,'-YEAR SIMULATION NUMBER=',I2/'NUMBER OF STORMS=',I3/)

      DO 700 M=1,6
         M1=M-1
         WRITE(5,610) M1,NYNS(M)

 700  CONTINUE
 610  FORMAT(/'NUMBER OF YEARS WITH ',I2,' STORMS=',I3)
      WRITE(5,620) DPTH(1)*0.3048
 620  FORMAT(/'WATER DEPTH BELOW MEAN SEA LEVEL=',F8.2,' (M)'/
     1       ' RANK ',' RE.YEAR ',' STORM TIDE (M) ',
     2       ' W. HEIGHT (M) ',' W. PERIOD (S)'/)

      DO 710 N=1,NSTORM
         REYEAR=FLOAT(NYEAR)/FLOAT(N)

C     STORE (500/5)=100 YEAR MAXIMUM STORM TIDE, SIGNIFICANT WAVE HEIGHT,
C     AND SIGNIFICANT WAVE PERIOD SEPARATELY

         IF(N.EQ.5) THEN
            WRITE(7,777) N500,NSTORM,REYEAR,SMAXST(N),SMAXWH(N),
     1                   SMAXWT(N)
         ENDIF

         WRITE(5,630) N,REYEAR,SMAXST(N),SMAXWH(N),SMAXWT(N)

 710  CONTINUE
 777  FORMAT(I3,I5,F8.2,3F7.2)
 630  FORMAT(2X,I3,5X,F8.2,8X,F7.2,8X,F7.2,8X,F7.2)

      STOP
      END

C--00------------------- END OF MAIN PROGRAM --------------------------------
```

```
C*****************************************************************************
C
C     LIST OF 19 SUBROUTINES ARRANGED IN NUMERICAL ORDER
C
C*****************************************************************************
C
C##01#########################  SUBROUTINE NUMBER  ############################

      SUBROUTINE NUMBER(NSEED,NOSTOM)

C-----------------------------------------------------------------------------
C     THIS SUBROUTINE GENERATES NUMBER OF STORMS PER YEAR BASED ON A
C     POISSON DISTRIBUTION WITH MEAN=0.7 PER YEAR
C-----------------------------------------------------------------------------

      DIMENSION PROB(7)

C-----INPUT CUMULATIVE PROBABILITY FOR NOSTOM---------------------------------

      DATA(PROB(M),M=1,7)/0.0,0.4966,0.8442,0.9659,0.9943,0.9993,1.0/

C-----PICK RANDOM NUMBER CORRESPONDING TO CUMULATIVE PROBABILITY--------------

      RNUM=RAN(NSEED)

      DO 1 M=2,7
         IF(RNUM.GE.PROB(M-1).AND.RNUM.LT.PROB(M)) NOSTOM=M-2
 1    CONTINUE

      RETURN
      END

C--01--------------------  END OF SUBROUTINE NUMBER  -------------------------

C##02#########################  SUBROUTINE STORM1  ############################

      SUBROUTINE STORM1(NSEED,SS,WH1,WT1,PN,DT,DX,DY,NOFF,
     1                  DPTH,IMAX,JMAX,LAMDA)

C-----------------------------------------------------------------------------
C     THIS SUBROUTINE RANDOMLY SELECTS THE STORM PARAMETERS, THEN COMPUTES
```

```
C     THE STORM SURGE USING THE BATHYSTROPHIC STORM SURGE AT EACH TIME STEP.
C     THIS SUBROUTINE ALSO CALLS THE WAVE HEIGHT AND PERIOD SUBROUTINES.
C
C     WRITEN BY DAVID KRIEBEL IN 1982
C     MODIFIED BY N. KOBAYASHI AND B. POZUETA IN 2001
C-----------------------------------------------------------------------

      DIMENSION TAUX(500),TAUY(500),PS(500),DPTH(500)
      DIMENSION ETA(500)
      REAL LAMDA,LAMDA1
      COMMON/TIMESE/NTIMEL,STTIDE(200),SWAVEH(200),SWAVET(200)


C-----NUMERICAL HURRICANE GENERATION------------------------------------

C     RANDOM GENERATION OF HURRICANE TRANSLATION DIRECTION, ZETA(DEGREES),
C     COUNTERCLOCKWISE FROM OFFSHORE DIRECTION USING SUBR.08 DIR.
C     ALSO, DEFINE STORM AS ALONGSHORE USING SUBR.09 ALONG OR LANDFALLING
C     USING SUBR.10 LANDF WHERE THESE SUBROUTINES ARE CALLED IN SUBR.08 DIR.

        CALL DIR(NSEED,ZETA,XC,YC,IALONG)

C     RANDOM GENERATION OF RADIUS TO MAXIMUM WIND, RMAX(NAUTICAL MILES),
C     USING SUBR.11 RAD.

        CALL RAD(NSEED,RMAX)

C     RANDOM GENRATION OF CENTRAL PRESSURE, PO(INCHES OF MERCURY), USING
C     SUBR.12 CENTP.

        CALL CENTP(NSEED,PO)

C     RANDOM GENERATION OF FORWARD SPEED, VF(KNOTS), USING SUBR.13 FORSP.

        CALL FORSP(NSEED,VF)

C     ASSIGN INITIAL STORM CENTER NODE,(IO,JO), USING SUBR.14 STMCEN.

        CALL STMCEN(IALONG,ZETA,XC,YC,DX,DY,IO,JO)

C     ASSIGN INITIAL RISE, RISEIN(FT), FROM STORM DIRECTION USING SUBR.16 RISE.
```

```
      CALL RISE(ZETA,RISEIN)


C     ASSIGN ASTRONOMICAL TIDE AMPLITUDE, AMP(FT) AND RANDOM PHASE, TPHASE
C     (RADIANS) BASED ON RANDOM NUMBER SELECTION USING SUBR.15 ASTTID.

      CALL ASTTID(NSEED,AMP,TAST,TPHASE)

C     CONSTANTS WITH UNIT CONVERSIONS WHERE
C     F=CORIOLIS PARAMETER, RO=AIR DENSITY, AND RHOW=SEA WATER DENSITY

     ZETA=ZETA*3.14159/180.
     LAMDA1=LAMDA*3.14159/180.
     F=0.525*SIN(LAMDA1)
     RO=0.00194*0.01414*(6076.**2.)/(3600.**2.)
     RHOW=1.99*(6076.**2.)/(3600.**2.)

C-----SET INITIAL CONDITIONS--------------------------------------------------

      XO=(IO-1)*DX
      YO=(JO-1)*DY
      T=0.0
      VLAND=1.0
      COUNT=0.0
      KTIME=1
      WSELEV=0.0

C     TENTATIVE VALUES FOR FINDING MAXIMUM VALUES

      SS=0.0
      WH1=0.0
      WT1=0.0

C     WHERE T=TIME, VLAND=EMPIRICAL WIND REDUCTION FACTOR WHEN THE STORM
C     CENTER MOVES OVER LAND, COUNT=COUNTER USED TO COUNT THE DURATION OVER
C     LAND, KTIME=1 TO INDICATE THE INITIAL CONDITION, AND WSELEV=STORM TIDE.

C     DETERMINE MAXIMUM GRADIENT WIND SPEED, UMAX(KNOTS), USING SUBR.17 WAVE1.
```

```
      CALL WAVE1(PO,PN,RMAX,VF,RO,F,UMAX)

C     HORIZONTAL COORDINATE OF MOST LANDWARD NODE

      YPROF=0.0
      XPROF=0.0

C-----START INDIVIDUAL STORM CALCULATIONS-------------------------------------

      DO 3 LTIME=1,101


        T=FLOAT(LTIME-1)*DT

        IF(RISEIN.EQ.0.0) GO TO 105
        ZETAO=ZETA*180./3.14159
        IF((ZETAO.GE.150.).AND.(ZETAO.LE.210.)) GO TO 100
        IF((ZETAO.LT.150.). OR.(ZETAO.GT.210.)) GO TO 101
        GO TO 105
 100    CONTINUE
        IF(IO.LT.(-5)) RISEIN=RISEIN-0.5
        GO TO 105

 101    CONTINUE
        IF(IO.LT.(-3)) RISEIN=RISEIN-0.5
 105    CONTINUE

        ASTRO=AMP*COS((2.*3.14159*T/TAST)+TPHASE)

C     COMPUTE HURRICANE PRESSURE AND WIND FIELD FOR CROSS-SHORE LINE USING
C     SUBR.03 HURR

      CALL HURR(NOFF,DX,DY,XO,YO,IO,JO,ZETA,RMAX,PO,PN,
     1         RO,F,VF,VLAND,RHOW,PS,TAUX,TAUY,VR,THETAO,UMAX,VMAX)

C     COMPUTE HURRICANE STORM SURGE BY INTEGRATING WIND STRESS ALONG
C     CROSS-SHORE LINE USING SUBR.07 SURGE

      CALL SURGE(NOFF,PN,PS,TAUX,TAUY,DPTH,DT,F,DX,T,
     1          ETA,KTIME)
```

```
C     TOTAL STORM TIDE WSELEV(FT) INCLUDING TIDE AND INITIAL RISE AT MOST
C     LANDWARD NODE

      WSELEV=-ETA(NOFF)+ASTRO+RISEIN

C     COMPUTE WAVE HEIGHT, WH(FT) AND PERIOD, WT(S) FOR GIVEN WIND CONDITIONS
C     AT MOST LANDWARD NODE USING SUBR.18 WAVE2.

      CALL WAVE2(RMAX,PN,PO,VF,VR,VMAX,RISEIN,THETAO,WH,WT)

C     WHICH ARE STORED FOR OUTPUT IN MAIN PROGRAM

      STTIDE(LTIME)=WSELEV
      SWAVEH(LTIME)=WH
      SWAVET(LTIME)=WT


C-----ADVANCE STORM CENTER-----------------------------------------------------

      XO=XO+(DT*VF*COS(ZETA))
      YO=YO+(DT*VF*SIN(ZETA))
      IO=INT(1.0+(XO/DX))
      JO=INT(1.0+(YO/DY))

C     STORM IS OUT OF DOMAIN AND STOP COMPUTATION

      IF((IO.LT.(-10)).OR.(JO.LT.(-JMAX))) GO TO 301
      IF((IO.GT.IMAX).OR.(JO.GT.JMAX)) GO TO 301

C     ESTIMATE FRICTIONAL REDUCTION FACTOR VLAND OF STORM INTERACTING WITH LAND


      IF(T.EQ.0.5) IOIN=IO
      IF(IO.LT.1) COUNT=COUNT+DT
      VLAND=1.0
      IF(COUNT.GE.1.0) VLAND=1.0-(COUNT*0.03)
      IF(COUNT.GE.6.0) VLAND=0.82
      IF((IOIN.LT.1).AND.(IO.LT.1)) VLAND=0.82


C     DETERMINE MAXIMUM STORM TIDE, WAVE HEIGHT AND PERIOD
```

```
      IF(WSELEV.GT.SS) SS=WSELEV
      IF(WH.GT.WH1) WH1=WH
      IF(WT.GT.WT1) WT1=WT

C    IF WATER SURFACE BECOMES TOO NEGATIVE, (-3 FT HERE) STOP COMPUTATION

      IF(WSELEV.LT.-3.0) GO TO 301
      IF(LTIME.EQ.101) GO TO 301


3     CONTINUE

301   CONTINUE

C     END OF THIS STORM
C     THE NUMBER OF TIME LEVELS FOR THIS STORM
      NTIMEL=LTIME

      RETURN
      END

C--02-------------------- END OF SUBROUTINE STORM1 -------------------------

C##03######################### SUBROUTINE HURR ##############################

      SUBROUTINE HURR(NOFF,DX,DY,XO,YO,IO,JO,ZETA,RMAX,
     1                PO,PN,RO,F,VF,VLAND,RHOW,PS,TAUX,TAUY,VR,
     2                THETAO,UMAX,VMAX)

C----------------------------------------------------------------------------
C     THIS SUBROUTINE GENERATES THE HURRICANE PRESSURE AND WIND FIELD FOR
C     POINTS ON THE CROSS-SHORE LINE FROM THE MOST LANDWARD NODE NEAR THE
C     SHORELINE TO THE EDGE OF THE CONTINENTAL SHELF MOSTLY ON THE BASIS OF
C     1977 SHORE PROTECTION MANUAL.
C
C     WRITTEN BY DAVID KRIEBEL IN 1982
C     MODIFIED BY N. KOBAYASHI AND B.POZUETA IN 2001
C----------------------------------------------------------------------------

      DIMENSION PS(500),TAUX(500),TAUY(500)
```

```
      DO 3 I=1,NOFF

          ICOUNT=I
          X=(I-1)*DX
          Y=0.0
          XX=X-XO
          YY=Y-YO
          R=SQRT((ABS(XX))**2.+(ABS(YY))**2.)

C     CALL SUBR.04 ANGLE FOR WIND ANGLES THETA AND ALPHA, SUBR.05 PRESS TO
C     COMPUTE ATMOSPHERIC PRESSURE,P, AND SUBR.06 VEL TO COMPUTE WIND SPEEDS
C     AND SHEAR STRESSES,TAUWX AND TAUWY.

          CALL ANGLE(ICOUNT,IO,JO,YY,XX,THETA,ZETA,ALPHA)
          CALL PRESS(RMAX,PO,PN,R,P)
          CALL VEL(P,PO,RMAX,R,RO,F,VF,ALPHA,VLAND,THETA,
     1            ICOUNT,V,RHOW,TAUWX,TAUWY,THETA1,UMAX,VMAX)

          PS(I)=P
          TAUX(I)=TAUWX
          TAUY(I)=TAUWY
          IF(I.EQ.1) VR=V
          IF(I.EQ.1) THETAO=THETA1

3     CONTINUE

C     NOTE THAT WIND SPEED, VR, AND WIND ANGLE, THETAO AT THE MOST LANDWARD
C     NODE I=1 ARE USED TO PREDICT THE SINIFICANT WAVES IN SUBR.18 WAVE2.

      RETURN
      END

C--03-------------------- END OF SUBROUTINE HURR ---------------------------

C##04######################### SUBROUTINE ANGLE #############################

      SUBROUTINE ANGLE(ICOUNT,IO,JO,YY,XX,THETA,ZETA,ALPHA)
```

```
C-----------------------------------------------------------------------
C     THIS SUBROUTINE COMPUTES THE ANGLE THETA COUNTER CLOCKWISE FROM THE
C     CROSS-SHORE AXIS (+ OFFSHORE) TO THE POINT (ICOUNT=I,J=1) RELATIVE TO
C     THE STORM CENTER (IO,JO), AND THE ANGLE ALPHA COUNTERCLOCKWISE FROM
C     THE FORWARD STORM VELOCITY VECTOR FOR THE SAME POINT.
C-----------------------------------------------------------------------

      PI=3.14159
      IF(XX.EQ.0.0) THEN
         IF(YY.EQ.0.0) THETA=0.0
         IF(YY.GT.0.0) THETA=0.5*PI
         IF(YY.LT.0.0) THETA=1.5*PI
      ELSE
         THETA=ATAN(YY/XX)
         IF(XX.LT.0.0) THETA=THETA+PI
         IF(XX.GT.0.0.AND.YY.LT.0.0) THETA=THETA+2.0*PI
      ENDIF

      ALPHA=(2.0*PI-ZETA)+THETA

      RETURN
      END

C--04-------------------- END OF SUBROUTINE ANGEL -------------------------

C##05######################### SUBROUTINE PRESS ##############################

      SUBROUTINE PRESS(RMAX,PO,PN,R,P)

C-----------------------------------------------------------------------
C     THIS SUBROUTINE COMPUTES THE PRESSURE, P, AT EACH GRID POINT AT THE
C     DISTANCE R FROM THE STORM CENTER
C-----------------------------------------------------------------------

      IF(R.GT.5.0) THEN
         P=PO+(PN-PO)*EXP(-RMAX/R)
      ELSE
         P=PO
      ENDIF
```

```
      RETURN
      END

C--05-------------------- END OF SUBROUTINE PRESS --------------------------

C##06######################### SUBROUTINE VEL #################################

      SUBROUTINE VEL(P,PO,RMAX,R,RO,F,VF,ALPHA,VLAND,THETA,
     1             ICOUNT,V,RHOW,TAUWX,TAUWY,THETA1,UMAX,VMAX)

C-----------------------------------------------------------------------------
C     THIS SUBROUTINE USES THE HURRICANE MODEL DEVELOPED BY REID AND WILSON
C     TO COMPUTE THE WIND VELOCITY COMPONENTS AT EACH GRID POINT ALONG THE
C     CROSS-SHORE LINE

C     SUBROUTINE ALSO COMPUTES WIND SHEAR STRESS ON WATER AND ADJUSTS FOR
C     SLOWING OF WIND SPEEDS DUE TO STORM CENTER OR WINDS INTERACTING WITH LAND
C-----------------------------------------------------------------------------

C     CALCULATE GRADIENT WIND SPEED, V(KNOTS)

      IF(R.GT.5.0) THEN
         UC=SQRT((P-PO)*RMAX/(R*RO))
         UG=(UC**2.)/(R*F)
         GAMMA=0.5*((UC/UG)+(VF*SIN(ALPHA)/UC))
         V=UC*(SQRT(GAMMA**2.+1.)-GAMMA)
      ELSE
         V=0.0

      ENDIF

C     ADJUST FOR SURFACE FRICTION (ASSUMED A REDUCTION FACTOR OF 0.885)AND
C     STORM INTERACTION WITH LAND USING EMPIRICAL VLAND

      V=V*0.885*VLAND

C     DETERMINE INFLOW ANGLE, AINFLO(DEGREES), FOR NEAR-SURFACE WIND RELATIVE
C     TO GRADIENT WIND SPEED

      IF(RMAX.GE.15.) GO TO 4
      IF(RMAX.LT.15.) GO TO 5
```

```
1     CONTINUE

C     DETERMINE WIND VELOCITY COMPONENTS IN X (CROSS-SHORE, + OFFSHORE) AND
C     Y (ALONG-SHORE) DIRECTIONS

      THETA1=THETA+(90.+AINFLO)*3.14159/180.
      WX=V*COS(THETA1)
      WY=V*SIN(THETA1)

C     DETERMINE FRICTION AT SHORELINE

      IF(WX.GT.0.0) GO TO 6
      IF(WX.LE.0.0) GO TO 7
2     CONTINUE

C     DETERMINE WIND STRESSES, IN X AND Y DIRECTIONS USING WSC=SURFACE
C     FRICTION COEFFICIENT BASED ON RHOW=WATER DENSITY

      WSC=1.21E-06
      IF(V.LT.14.0) GO TO 3
      WSC=WSC+2.75E-06*(1.0-14.0/V)**2.
3     CONTINUE
      TAUWX=RHOW*WSC*V*WX
      TAUWY=RHOW*WSC*V*WY
      GO TO 9

C     NEAR-SURFACE WIND ANGLE ADJUSTMENTS

4     CONTINUE
      AINFLO=20.
      IF(R.GT.(3.*RMAX)) AINFLO=25.
      IF(R.LT.(1.5*RMAX)) AINFLO=15.
      GO TO 1

5     CONTINUE
      AINFLO=15.
      IF(R.GT.(3.*RMAX)) AINFLO=10.
      IF(R.LT.(1.5*RMAX)) AINFLO=10.
      GO TO 1
```

```
C     FOR OFFSHORE WIND, REDUCE WIND SPEED AT NODE I=1 AND 2

6     CONTINUE
      IF(ICOUNT.LE.1) VSHORE=0.70
      IF(ICOUNT.EQ.2) VSHORE=0.90
      IF(ICOUNT.GE.3) VSHORE=1.0
      GO TO 8

C     FOR ONSHORE WIND, REDUCE WIND SPEED AT NODE I=1

7     CONTINUE
      VSHORE=1.0
      IF(ICOUNT.LE.1) VSHORE=0.89
      GO TO 8

8     CONTINUE

C     REDUCED WIND SPEEDS

      V=V*VSHORE
      WX=WX*VSHORE
      WY=WY*VSHORE
      GO TO 2


C     ADJUST MAXIMUM WIND SPEED, VMAX(KNOTS), FOR FRICTION EFFECTS
C     NOTE THAT VMAX IS USED TO PREDICT THE MAXIMUM SIGNIFICANT WAVE HEIGHT
C     IN SUBR.18 WAVE2.

9     CONTINUE
      VMAX=UMAX*0.885*VLAND


      RETURN
      END

C--06-------------------- END OF SUBROUTINE VEL ----------------------------

C##07######################### SUBROUTINE SURGE #############################

      SUBROUTINE SURGE(NOFF,PN,PS,TAUX,TAUY,DPTH,DT,F,DX,T,
```

```
     1                ETA,KTIME)
C-----------------------------------------------------------------------
C     THIS SUBROUTINE COMPUTES THE STORM SURGE USING A SIMPLE ONE-DIMENSIONAL
C     (CROSS-SHORE X-DIRECTION) EQUATION OF MOTION

C     THE STORM SURGE, ETA(FT), IS COMPUTED LANDWARD FROM THE EDGE OF
C     CONTINENTAL SHELF (IN NEGATIVE X-DIRECTION) AND NEGATIVE ETA IS POSITIVE
C     (UPWARD) STORM SURGE. THE ASTRONOMICAL TIDE AND INITIAL WATER LEVEL RISE
C     ARE NOT INCLUDED HERE.
C-----------------------------------------------------------------------

      DIMENSION ETAV(500),ETAV1(500),ETAX(500),ETAX1(500)
      DIMENSION ETAP(500),ETAP1(500),V1(500),TAUAVG(500)
      DIMENSION ETA(500),DPTH(500),PS(500),TAUX(500),TAUY(500)

C     SPECIFIC WEIGHT OF SEAWATER IN ENGLISH UNITS WITH 0.01414 FOR THE UNITS
C     CONVERSION FOR INCHES IN MERCURY

      GAMMA=1.99*32.2
      GAMMA1=1.99*32.2*0.01414


C     INITIALIZE VALUES FOR FIRST PASS THROUGH USING KTIME=1

      IF(KTIME.EQ.1) GO TO 2

C     ASSUMED BOTTOM FRICTION COEFFICIENT

      BOTFF=0.001

C     BEGIN STORM SURGE CALCULATIONS --- INTEGRATE FROM EDGE OF SHELF TO
C     SHORELINE IN NEGATIVE X-DIRECTION


      DO 1 N=1,NOFF-1
        NN=NOFF-N+1


C     OFFSHORE BOUNDARY CONDITIONS FOR AIR PRESSURE AND STORM SURGE
```

```
      ETAP(1)=-(PN-PS(NOFF))/GAMMA1

C     DETERMINE CHANGE IN WATER SURFACE DUE TO PRESSURE CHANGE

      ETAP(N+1)=ETAP(N)+(PS(NN-1)-PS(NN))/GAMMA1

C     DETERMINE AVERAGE SURFACE SHEAR STRESS BETWEEN GRID POINTS

      TAUXAV=(TAUX(NN)+TAUX(NN-1))/2.0
      TAUYAV=(TAUY(NN)+TAUY(NN-1))/2.0

C     DETERMINE TOTAL DEPTH

      DPTHAV=(DPTH(NN)+DPTH(NN-1))/2.0
      DEPTH=DPTHAV-0.5*(ETAP(N)+ETAP(N+1))-
     1 0.5*(ETAV1(N)+ETAV1(N+1))-0.5*(ETAX1(N)+ETAX1(N+1))
      DEPTHV=DPTHAV-0.25*(ETAP(N)+ETAP(N+1)+
     1ETAP1(N)+ETAP1(N+1))-0.5*(ETAV1(N)+ETAV1(N+1))-
     20.5*(ETAX(N)+ETAX1(N+1))

C     FIND ALONGSHORE BATHYSTROPHIC FLUX, V (DEPTH*ALONGSHORE CURRENT VELOCITY)

      VNUM=V1(N)+(3600.*DT/1.99)*(0.5*(TAUYAV+TAUAVG(N)))
      VDENOM=1.0+(BOTFF*3600.*DT*ABS(V1(N))/(DEPTHV**2.))
      V=VNUM/VDENOM

      VMAX=SQRT(ABS(TAUYAV)*(DEPTH**2.)/(BOTFF*1.99))
      VV=ABS(V)
      IF(VV.GT.VMAX) V=(V/VV)*VMAX


C     DETERMINE CHANGE IN WATER SURFACE DUE TO CORIOLIS EFFECT

      ETAV(N+1)=ETAV(N)+((F*DX*6076.)/(32.2*3600.))*(V/DEPTH)

C     DETERMINE CHANGE IN WATER SURFACE DUE TO ONSHORE WIND STRESS

      ETAX(N+1)=ETAX(N)+(TAUXAV*DX*6076.)/(GAMMA*DEPTH)

C     DETERMINE TOTAL CHANGE IN WATER SURFACE AT EACH GRID EXCEPT FOR N=1
```

```
      ETA(N+1)=ETAP(N+1)+ETAV(N+1)+ETAX(N+1)

C     STORE PRESENT VALUES FOR USE IN NEXT TIME STEP

      TAUAVG(N)=TAUYAV
      V1(N)=V
      ETAV1(N+1)=ETAV(N+1)
      ETAP1(N+1)=ETAP(N+1)
      ETAX1(N+1)=ETAX(N+1)


 1    CONTINUE

C     WATER SURFACE CHANGE AT MOST SEAWARD NODE (N=1) DUE TO ATMOSPHERIC
C     PRESSURE CHANGE ONLY

      ETA(1)=ETAP(1)
      GO TO 4


C     SET INITIAL VALUES FOR KTIME=1 ONLY

2     CONTINUE

      DO 3 N=1,NOFF
        V1(N)=0.0
        ETAP(N)=0.0
        ETAP1(N)=0.0
        ETAV(N)=0.0
        ETAV1(N)=0.0
        ETAX(N)=0.0
        ETAX1(N)=0.0
        ETA(N)=0.0
        TAUAVG(N)=0.0
3     CONTINUE
      KTIME=2

4     CONTINUE

      RETURN
      END
```

```
C--07------------------- END OF SUBROUTINE SURGE -------------------------

C##08########################## SUBROUTINE DIR ###############################

      SUBROUTINE DIR(NSEED,ZETA,XC,YC,IALONG)

C-------------------------------------------------------------------------------
C     THIS SUBROUTINE GENERATES HURRICANE TRANSLATION DIRECTION, ZETA(DEGREES)
C     BASED ON THE HISTORICAL PROBABILITY AT PANAMA CITY, FLORIDA
C-------------------------------------------------------------------------------

      DIMENSION ZETA1(10),PROB(12)
      DIMENSION XC1(10),YC1(10)

C     INPUT CUMULATIVE PROBABILTY DISTRIBUTION FOR ZETA(DEGREES) RELATIVE TO
C     THE CROSS-SHORE LINE (+ OFFSHORE)

      DATA(ZETA1(M),M=1,7)/80.,100.,120.,180.,240.,260.,300./,
     & (PROB(M),M=1,7)/0.0,0.08,0.09,0.60,0.90,0.95,1.0/

C     PICK RANDOM NUMBER (BETWEEN 0 AND 1) AND INTERPOLATE TO FIND ZETA

      RNUM=RAN(NSEED)

      DO 1 M=2,7
      IF(RNUM.GT.PROB(M)) GO TO 1
      DPROB=PROB(M)-PROB(M-1)
      RATIO=(RNUM-PROB(M-1))/DPROB
      DZETA=ZETA1(M)-ZETA1(M-1)
      ZETA=ZETA1(M-1)+DZETA*RATIO
      GO TO 2
1     CONTINUE
2     CONTINUE

C     DETERMINE WHETHER LANDFALLING OR ALONGSHORE STORM

      XC=0.0
      YC=0.0
      IF((ZETA.GT.70.0).AND.(ZETA.LT.110.0)) GO TO 3
      IF((ZETA.GT.250.0).AND.(ZETA.LT.300.0)) GO TO 3
```

```
      GO TO 4

3     CONTINUE

C     STORM IS ALONGSHORE (IALONG=1), USE PROB. DIST. TO FIND OFFSHORE DISTANCE
C     XC(NAUTICAL MILES) OF HURRICANE CROSSING POINT ALONG CROSS-SHORE LINE
C     USING SUBR.09 ALONG


      CALL ALONG(NSEED,XC,IALONG)

      GO TO 5

4     CONTINUE
C     STORM IS LANDFALLING (IALONG=2), USE PROB. DIST. TO FIND ALONGSHORE
C     DISTANCE YC(NAUTICAL MILES) OF LANDFALL POINT FROM THE CROSS-SHORE LINE

      CALL LANDF(NSEED,YC,IALONG)

5     CONTINUE


      RETURN
      END

C--08-------------------- END OF SUBROUTINE DIR ----------------------------

C##09########################## SUBROUTINE ALONG ##############################


      SUBROUTINE ALONG(NSEED,XC,IALONG)

      DIMENSION XC1(10),PROB(12)
      DATA(XC1(M),M=1,5)/50.,75.,105.,125.,150./,
     & (PROB(M),M=1,5)/0.,0.33,0.51,0.84,1.0/
      RNUM=RAN(NSEED)
      DO 6 M=2,5
      IF(RNUM.GT.PROB(M)) GO TO 6
      DPROB=PROB(M)-PROB(M-1)
      RATIO=(RNUM-PROB(M-1))/DPROB
      DXC=XC1(M)-XC1(M-1)
```

```
      XC=XC1(M-1)+DXC*RATIO
      GO TO 7
6     CONTINUE
7     CONTINUE

      IALONG=1
      RETURN
      END

C--09-------------------- END OF SUBROUTINE ALONG ---------------------------

C##10######################### SUBROUTINE LANDF ##############################

      SUBROUTINE LANDF(NSEED,YC,IALONG)

      DIMENSION YC1(10),PROB(12)
      DATA(YC1(M),M=1,5)/-150.,-75.,-0.,75.,150./,
     & (PROB(M),M=1,5)/0.,0.25,0.5,0.75,1.0/
      RNUM=RAN(NSEED)
      DO 8 M=2,5
      IF(RNUM.GT.PROB(M)) GO TO 8
      DPROB=PROB(M)-PROB(M-1)
      RATIO=(RNUM-PROB(M-1))/DPROB
      DYC=YC1(M)-YC1(M-1)
      YC=YC1(M-1)+DYC*RATIO
      GO TO 9
8     CONTINUE
9     CONTINUE

      IALONG=2
      RETURN
      END

C--10-------------------- END OF SUBROUTINE LANDF ---------------------------

C##11######################### SUBROUTINE RAD #################################

      SUBROUTINE RAD(NSEED,RMAX)

C-----------------------------------------------------------------------------
C     THIS SUBROUTINE GENERATES THE RADIUS, R(NAUTICAL MILES) FROM STORM CENTER
```

```
C     TO MAXIMUM WIND, BASED ON THE HISTORICAL PROBABILITY AT PANAMA CITY,
C     FLORIDA
C-----------------------------------------------------------------------

      DIMENSION RMAX1(10),PROB(12)

C     INPUT PROBABILITY DISTRIBUTION FOR RMAX(NMI)

       DATA(RMAX1(M),M=1,6)/5.,10.,15.,23.,34.,45./,
     & (PROB(M),M=1,6)/0.0,0.09,0.17,0.50,0.83,1.0/

C     PICK RANDOM NUMBER (BETWEEN 0 AND 1)AND INTERPOLATE TO FIND RMAX

      RNUM=RAN(NSEED)

      DO 1 M=2,6
      IF(RNUM.GT.PROB(M)) GO TO 1
      DPROB=PROB(M)-PROB(M-1)
      RATIO=(RNUM-PROB(M-1))/DPROB
      DRMAX=RMAX1(M)-RMAX1(M-1)
      RMAX=RMAX1(M-1)+DRMAX*RATIO
      GO TO 2
1     CONTINUE
2     CONTINUE


      RETURN
      END

C--11------------------- END OF SUBROUTINE RAD -----------------------------

C##12######################### SUBROUTINE CENTP ##############################

      SUBROUTINE CENTP(NSEED,PO)

C-----------------------------------------------------------------------
C     THIS SUBROUTINE GENERATES THE HURRICANE CENTRAL PRESSURE, PO(INCHES IN
C     MERCURY) BASED ON THE HISTORICAL PROBABILITY AT PANAMA CITY, FLORIDA
C-----------------------------------------------------------------------

      DIMENSION PO1(10),PROB(12)
```

```
C     INPUT PROBABILITY DISTRIBUTION FOR PO(IN. IN Hg)

      DATA(PO1(M),M=1,7)/26.90,27.25,27.75,28.35,28.80,29.10,29.50/,
     & (PROB(M),M=1,7)/0.0,0.02,0.06,0.17,0.30,0.50,1.0/

C     PICK RANDOM NUMBER (BETWEEN 0 AND 1) AND INTERPOLATE TO FIND PO

      RNUM=RAN(NSEED)

      DO 1 M=2,7
      IF(RNUM.GT.PROB(M)) GO TO 1
      DPROB=PROB(M)-PROB(M-1)
      RATIO=(RNUM-PROB(M-1))/DPROB
      DPO=PO1(M)-PO1(M-1)
      PO=PO1(M-1)+DPO*RATIO
      GO TO 2
1     CONTINUE
2     CONTINUE

      RETURN
      END

C--12-------------------- END OF SUBROUTINE CENTP ---------------------------

C##13########################## SUBROUTINE FORSP ############################

      SUBROUTINE FORSP(NSEED,VF)

C---------------------------------------------------------------------------
C     THIS SUBROUTINE GENERATES THE HURRICANE FORWARD SPEED, VF(KNOTS),
C     BASED ON THE HISTORICAL PROBABILITY AT PANAMA CITY, FLORIDA
C---------------------------------------------------------------------------

      DIMENSION VF1(10),PROB(12)

C     INPUT PROBABILITY DISTRIBUTION FOR VF(KNOTS)

      DATA(VF1(M),M=1,5)/5.0,7.5,12.5,22.5,28.5/,
     & (PROB(M),M=1,5)/0.,0.2,0.6,0.95,1.0/
```

```
C     PICK RANDOM NUMBER (BETWEEN 0 AND 1) AND INTERPOLATE TO FIND VF

      RNUM=RAN(NSEED)

      DO 1 M=2,5
      IF(RNUM.GT.PROB(M)) GO TO 1
      DPROB=PROB(M)-PROB(M-1)
      RATIO=(RNUM-PROB(M-1))/DPROB
      DVF=VF1(M)-VF1(M-1)
      VF=VF1(M-1)+DVF*RATIO
      GO TO 2
1     CONTINUE
2     CONTINUE

      RETURN
      END

C--13-------------------- END OF SUBROUTINE FORSP --------------------------

C##14######################### SUBROUTINE STMCEM ##############################

      SUBROUTINE STMCEN(IALONG,ZETA,XC,YC,DX,DY,IO,JO)

C----------------------------------------------------------------------------
C     THIS SUBROUTINE USES ZETA,XC,YC, AND GRID SPACING DX AND DY TO ASSIGN
C     A REASONABLE STARTING NODAL LOCATION FOR THE CENTER OF THE HURRICANE,
C     IO AND JO
C----------------------------------------------------------------------------

C     ASSUMED THE CONTINENTAL SHELF WIDTH ON THE ORDER OF 150 NAUTICAL MILES

      IF(IALONG.EQ.1) GO TO 3
      IF((ZETA.GE.0.0).AND.(ZETA.LT.90.))  AA=150.
      IF((ZETA.GE.90.).AND.(ZETA.LT.135.)) GO TO 1
      IF((ZETA.GE.135.).AND.(ZETA.LT.225.)) AA=150.
      IF((ZETA.GE.225.).AND.(ZETA.LT.270.)) GO TO 2
      IF((ZETA.GE.270.).AND.(ZETA.LT.360.)) AA=150.
      GO TO 4
1     CONTINUE
      IF(YC.GE.0.0) AA=150.+YC
      IF(YC.LT.0.0) AA=150.
```

```
      GO TO 4
2     CONTINUE
      IF(YC.GE.0.0) AA=150.
      IF(YC.LT.0.0) AA=150.-YC
      GO TO 4
3     CONTINUE
      AA=200.
4     CONTINUE
      ZETA1=ZETA*3.14159/180.
      XCI=XC-AA*COS(ZETA1)
      YCJ=YC-AA*SIN(ZETA1)
      XCI=XCI/DX
      YCJ=YCJ/DY
      IXC=INT(XCI)
      JYC=INT(YCJ)
      IO=1+IXC
      JO=1+JYC

      IF(IO.GT.IMAX) IO=IMAX
      IF(IO.LT.(-IMAX)) IO=-IMAX
      IF(JO.GT.JMAX) JO=JMAX
      IF(JO.LT.(-JMAX)) JO=-JMAX

      RETURN
      END

C--14-------------------- END OF SUBROUTINE STMCEM -------------------------

C##15######################### SUBROUTINE ASTTID ############################

      SUBROUTINE ASTTID(NSEED,AMP,TAST,TPHASE)

C--------------------------------------------------------------------------
C     THIS SUBROUTINE SELECTS THE ASTRONOMICAL TIDAL AMPLITUD, AMP(FT) AND
C     PHASE,TPHASE(RADIAN) AT PANAMA CITY, FLORIDA
C--------------------------------------------------------------------------

      DIMENSION AMP1(10),PROB(12)

C     ASTRONOMICAL TIDE PERIOD, TAST(HOURS)
```

```
      TAST=12.4

C     INPUT PROBABILITY OF TIDAL AMPLITUD, AMP(FT)

      DATA(AMP1(M),M=1,5)/0.1,0.3,0.9,1.5,2.1/,
     & (PROB(M),M=1,5)/0.0,0.09,0.35,0.80,1.00/

C     PICK RANDOM NUMBER (BETWEEN 0 AND 1)AND INTERPOLATE TO FIND RANDOM
C     AMPLITUDE

      RNUM=RAN(NSEED)

      DO 1 M=2,5
      IF(RNUM.GT.PROB(M)) GO TO 1
      DPROB=PROB(M)-PROB(M-1)
      RATIO=(RNUM-PROB(M-1))/DPROB
      DAMP=AMP1(M)-AMP1(M-1)
      AMP=AMP1(M-1)+DAMP*RATIO
      GO TO 2
1     CONTINUE
2     CONTINUE

C     ASSUME RANDOM PHASING OF ASTRONOMICAL TIDE RELATIVE TO STORM

      RNUM=RAN(NSEED)
      TPHASE=RNUM*3.14159*2

      RETURN
      END

C--15------------------- END OF SUBROUTINE ASTTID -------------------------

C##16######################### SUBROUTINE RISE ###############################

      SUBROUTINE RISE(ZETA,RISEIN)

C-------------------------------------------------------------------------
C     THIS SUBROUTINE ESTIMATES THE INITIAL RISE, RISEIN(FT) ACCORDING TO THE
C     DIRECTION OF THE STORM AT FLORIDA PANHANDLE
C-------------------------------------------------------------------------
```

```
      IF((ZETA.GT.240.).OR.(ZETA.LT.120.)) GO TO 1
      IF((ZETA.LT.210.).AND.(ZETA.GE.150.)) GO TO 2
      GO TO 3
1     RISEIN=0.0
      GO TO 4
2     RISEIN=2.0
      GO TO 4
3     RISEIN=1.0
4     CONTINUE

      RETURN
      END

C--16-------------------- END OF SUBROUTINE RISE ---------------------------

C##17#########################  SUBROUTINE WAVE1 ##############################

      SUBROUTINE WAVE1(PO,PN,RMAX,VF,RO,F,UMAX)

C-----------------------------------------------------------------------------
C     THIS SUBROUTINE COMPUTES THE GRADIENT WIND SPEED, UMAX(KNOTS) AT
C     RMAX(NMI)
C-----------------------------------------------------------------------------

      PMAX=PO+(PN-PO)*0.3679
      UCMAX=SQRT((PMAX-PO)/RO)
      UGMAX=UCMAX**2./(F*RMAX)
      GAMMA=0.5*((UCMAX/UGMAX)+(-VF/UCMAX))
      UMAX=UCMAX*(SQRT(GAMMA**2.+1.0)-GAMMA)

      RETURN
      END

C--17-------------------- END OF SUBROUTINE WAVE1 --------------------------

C##18#########################  SUBROUTINE WAVE2 ##############################

      SUBROUTINE WAVE2(RMAX,PN,PO,VF,VR,VMAX,RISEIN,THETAO,WH,WT)

C-----------------------------------------------------------------------------
C     THIS SUBROUTINE COMPUTES THE MAXIMUM DEEP WATER WAVE HEIGHT AT RMAX AND
```

```
C     THE MAXIMUM WAVE HEIGHT AT THE MOST LANDWARD NODE ACCORDING TO 1977 SHORE
C     PROTECTION MANUAL AND THE APPROXIMATION THAT LOCAL WAVE HEIGHT IS
C     PROPORTIONAL TO THE LOCAL WIND SPEED
C-----------------------------------------------------------------------

      PI=3.14159/180.

C     DETERMINE THE LOCAL WIND ANGLE AT MOST LANDWARD NODE

      IF(THETAO.GT.(360.*PI)) THETAO=THETAO-(360.*PI)
      IF((THETAO.LE.(90.*PI)).OR.(THETAO.GE.(270.*PI))) GO TO 10

C     DETERMINE MAXIMUM WAVE HEIGHT AND PERIOD USING 1977 SPM

      A=EXP(RMAX*(PN-PO)/100.)
      B=0.208*VF/(SQRT(VMAX))
      HOMAX=16.5*A*(1.0+B)
      HO=HOMAX*(VR/VMAX)

C     DETERMINE SIMPLEST REFRACTION EFFECTS TO REDUCE HO AND FIND THE
C     SIGNIFICANT WAVE HEIGHT, WH(FT)

      REFRAC=SQRT(ABS(COS(THETAO)))
      WH=HO*REFRAC
      GO TO 15

C     FOR OFFSHORE WIND, SET A SMALL WAVE HEIGHT (3 FT HERE)

10    CONTINUE

      WH=3.0

15    CONTINUE

C     ASSUME THE CORRESPONDING WAVE PERIOD(SEC)

      WT=2.13*SQRT(WH)

      RETURN
      END
```

```
C--18------------------- END OF SUBROUTINE WAVE2 ---------------------------

C##19########################## SUBROUTINE SHELL ###############################

      SUBROUTINE SHELL(N,ARR)

C----------------------------------------------------------------------------
C     THIS SUBROUTINE SORTS AN ARRAY, ARR, OF LENGTH N INTO DESCENDING
C     NUMERICAL ORDER, BY THE SHELL-MEZGAR ALGORITHM. N IS INPUT; ARR IS
C     REPLACED ON OUTPUT BY ITS SORTED REARRANGEMENT
C----------------------------------------------------------------------------

      PARAMETER(ALN2I=1./0.69314718, TINY=1.E-5)
      DIMENSION ARR(N)
      LOGNB2=INT(ALOG(FLOAT(N))*ALN2I+TINY)
      M=N
      DO 12 NN=1,LOGNB2
         M=M/2
         K=N-M
         DO 11 J=1,K
            I=J
 3          CONTINUE
            L=I+M
            IF(ARR(L).GT.ARR(I)) THEN
               T=ARR(I)
               ARR(I)=ARR(L)
               ARR(L)=T
               I=I-M
               IF(I.GE.1) GO TO 3
            ENDIF

 11      CONTINUE
 12   CONTINUE

      RETURN
      END

C************************* END OF SUBROUTINES *****************************
```

Appendix B

COMPUTER PROGRAM CSHORE2

```
C      #######   #######  ##    ##    ######   #######   ########
C     ##        ##        ##    ##   ##    ##  ##    ##  ##
C     ##        ##        ##    ##   ##    ##  ##    ##  ##
C     ##        #######   ########   ##    ##  #######   #######
C     ##             ##   ##    ##   ##    ##  ##  ##    ##
C     ##             ##   ##    ##   ##    ##  ##   ##   ##
C      #######  #######   ##    ##    ######   ##    ##  ########
C
C
C                            #####
C                           ##   ##
C                               ##
C                              ##
C                             ##
C                            ##
C                           #######
C
C
C
C            Nobuhisa Kobayashi and Bradley D. Johnson
C              Center for Applied Coastal Research
C           University of Delaware, Newark, Delaware 19716
C                        August, 1998
C
C
C  Modified by N.Kobayashi and B.Pozueta in 2001 to compute the time series
C  of the mean water depth including wave setup and the significant waves
C  at the toe of a coastal structure for the specified time series of the
C  offshore storm tide and siginificant waves for each of a large number of
C  storms.
C
C############################  GENERAL NOTES  ##################################
C                                                                              #
C  The purpose of each of 10 subroutines arranged in numerical order          #
C  is described in each subroutine and where it is called.                    #
C                                                                              #
C  All COMMON statements appear in the Main Program. Description of           #
C  each COMMON statement is given only in Main Program.                       #
```

```
C                                                                              #
C##00######################### MAIN PROGRAM ##################################
C*                                                                             *
C*   Main program marches from the offshore boundary node to the toe of a      *
C*   coastal structure using subroutines.                                      *
C*                                                                             *
C*******************************************************************************

      PROGRAM CSHORE2
      PARAMETER (NN=1000, NB=30)
      DIMENSION WSETOE(3),HTOE(3),HMOTOE(3)
      DIMENSION STSETM(3,500),HMOMAX(3,500),ARRAY(500)
      DIMENSION SXXH2(NN)
      CHARACTER*10 FBOTTOM

C ... COMMONs

C       Name     Contents
C      ----------------------------------------------------------------
C      /SYEARS/  Integers for 500-year simulation of storms
C      /SEABC/   Storm tide and waves at seaward boundary
C      /TOEDEP/  Toe depths of hypothetical coastal structures
C      /PERIOD/  Quantities at the spectral peak frequency
C      /PREDIC/  Unknowns predicted by CSHORE2
C      /BINPUT/  Input bottom geometry
C      /BPROFL/  Discritized bottom geometry
C      /CONSTA/  Constants
C      /LINEAR/  Linear wave values
C      /NONLIN/  Skewness and kurtosis
C      /WBREAK/  Wave breaking quantities and constants
C      /BRKNEW/  New wave breaking parameters in inner zone
C      /MOMENT/  Terms in momentum equation
C      /ENERGY/  Terms in energy equation
C      /ITERAT/  Iteration loop parameters

      COMMON /SYEARS/ N500, NYEAR, NSTORM
      COMMON /SEABC/ TIME, STORMT, HMOOFF, TP
      COMMON /TOEDEP/ NTOE, TOED(3), JTOE(3)
      COMMON /PERIOD/ FP, WKPO
      COMMON /PREDIC/ HRMS(NN), SIGMA(NN), H(NN), WSETUP(NN), SIGSTA(NN)
      COMMON /BINPUT/ XBINP(NB), ZBINP(NB), NBINP, JSWL
```

```
      COMMON /BPROFL/ DX, XB(NN), ZB(NN), DZBDX(NN), JMAX
      COMMON /CONSTA/ GRAV, SQR8, SQR2, PI, TWOPI
      COMMON /LINEAR/ WKP, CP, WN(NN)
      COMMON /NONLIN/ SKEW(NN), CURTO(NN)
      COMMON /WBREAK/ ALPHA, GAMMA, QBREAK(NN), DBSTA(NN)
      COMMON /BRKNEW/ GAMMAS, BETA, XS, XI
      COMMON /MOMENT/ CS(NN), FS, SXXSTA(NN)
      COMMON /ENERGY/ CF(NN), FE, EFSTA(NN)
      COMMON /ITERAT/ EPS1, MAXITE

      DATA EPS1, MAXITE /0.001, 100/
      DATA ALPHA, GAMMAS, BETA /1.0, 2.0, 2.2/

C ... INPUT N500, NYEAR AND NSTORM ON SCREEN ................................

      WRITE(*,*)
     +'Specify Integer N500 for N500-th simulation of 500 years'
      READ(*,*) N500

C     N500 must be in the range of 1-10 or N500=0
C     For N500=0, only one computation is made for one set of storm tide and
C     waves at seaward boundary.
C     The option of N500=0 should be used to check the accuracy of the input
C     bottom profile before 500-year simulation.

      IF(N500.LT.0.OR.N500.GT.10) WRITE(*,*)
     +'N500 must be in the range 0-10'

      IF(N500.GE.1.AND.N500.LE.10) THEN
         NYEAR=500
         WRITE(*,*)
     +   'Specify Number of Storms for this 500-Year Simulation'
         READ(*,*) NSTORM
         IF(NSTORM.GT.NYEAR) WRITE(*,*)
     +   'Increase 500 in STSETM, HMOMAX and ARRAY'
      ENDIF

      IF(N500.EQ.0) THEN
         NYEAR=1
         NSTORM=1
      ENDIF
```

```
C ... INPUT File Name for Bottom Profile ..................................

      WRITE(*,*) 'Name of Input File for Bottom Profile'
      READ(*,5000) FBOTTOM

 5000 FORMAT(A10)


C     Subr. 1 OPENER opens input and output files.

      CALL OPENER (FBOTTOM,N500)

C     Subr. 2 INPUTB gets input bathymetry information from the input file,
C     FBOTTOM.

      CALL INPUTB(N500)

C ... PREPARATIONS BEFORE STORM COMPUTATIONS ...............................

C     Subr. 3 BOTTOM computes bathymetry at each node.

      CALL BOTTOM

C     Subr. 4 PARAM  calculates constants and parameters.

      CALL PARAM

C     Integer IOUTP=0 or 1 is used to output computed cross-shore variations
C     only once as an example

      IOUTP=1

C --- COMPUTATIONS FOR NSTORM STORMS ---------------------------------------

      DO 9999 LSTORM=1,NSTORM

         WRITE(*,2111) LSTORM

 2111 FORMAT(/'LSTORM=',I3,'-TH STORM')
```

```
         IF(N500.EQ.0) THEN
            NTIMEL=1
         ELSE
            READ(4,2000) LL,NTIMEL
            IF(LL.NE.LSTORM) THEN
               WRITE(*,2800) LL,LSTORM
               WRITE(21,2800) LL,LSTORM
               STOP
            ENDIF
         ENDIF

 2000 FORMAT(2I3)
 2800 FORMAT(/'ERROR:'/
     +       'Storm number from File CYCLONEB**; LL=',I3/
     +       'NOT SAME as LSTORM=',I3,' in DO LOOP')

C ... COMPUTATION FOR NTIMEL TIME LEVELS FOR EACH STORM .......................
C     First, read the seaward boundary conditions

      DO 8888 LTIMEL=1,NTIMEL
         IF(N500.EQ.0) THEN
            WRITE(*,*) 'Input Spectral Peak Period TP in seconds'
            READ(*,*) TP
            WRITE(*,*) 'Input RMS Wave Height HRMS(1) in Meters
     +                  at Node=1'
            READ(*,*) HRMS(1)
            WRITE(*,*) 'Input Wave Setup WSETUP(1) in Meters at Node=1'
            READ(*,*) WSETUP(1)
            STORMT=0.0

            H(1)=WSETUP(1)-ZB(1)

         ELSE

C     Read from File CYCLONEB** with **=N500
C     TIME   = time in hours during each storm
C     STORMT = storm tide in meters
C     HMOOFF = offshore significant wave height in meters
C     TSOFF  = offshore significant wave period in seconds
```

```
C     H(1)   = total water depth at node 1.

            READ(4,2010) TIME,STORMT,HMOOFF,TSOFF

 2010       FORMAT(4F7.2)

C     To reduce the number of shoreward marching computations, no computation
C     is made if storm tide is negative or significant wave height is less
C     than 0.3 m. This corresponds to no waves in the conputation domain.

            IF(STORMT.LT.0.3.OR.HMOOFF.LT.0.3) THEN
              DO 99 J=1,JMAX
                H(J)=STORMT-ZB(J)

                IF(H(J).LT.0.0) H(J)=0.0
                TP=1.05*TSOFF
                HRMS(J)=0.0
                WSETUP(J)=0.0
 99           CONTINUE
              GO TO 7788
            ENDIF

C     Where 7788 is after the end of the shoreward marching computation

C     Assume the following relationships, where HMOOFF limited by water
C     depth H(1)

            H(1) = STORMT + WSETUP(1) - ZB(1)
            HRMS(1)=HMOOFF/SQR2
            IF(HMOOFF.GT.H(1)) HRMS(1)=H(1)/SQR2
            TP=1.05*TSOFF
            WSETUP(1)=0.0
          ENDIF

C     Compute: FP   = spectral peak frequency
C              WKPO = deep water wave number

          FP=1.0/TP
          WKPO = TWOPI**2.0/(GRAV*TP**2.0)

C     Where the parameters have been computed in Subr. 4 PARAM
```

```
C ... LANDWARD MARCHING COMPUTATION
C     For given LSTORM and LTIMEL
C     SIGMA(j) = free surface standard deviation at node j
C     H(j)     = total water depth at node j

         SIGMA(1) = HRMS(1)/SQR8

C     Subr. 5 LWAVE returns the linear wave number and ratio of group
C     velocity to phase velocity for the peak frequency.

         CALL LWAVE(1, H(1))

C     Subr. 6 SKEWKU returns skewness and kurtosis of the free surface
C     using empirical formulas.

         CALL SKEWKU(1,HRMS(1)/H(1))

C     Subr. 7 CSFFSE computes CS, CF, FS, and FE involved in cross-shore
C     radiation stress and energy flux.

         SIGSTA(1) = SIGMA(1)/H(1)
         CALL CSFFSE(1,SIGSTA(1))

         SIGMA2 =  SIGMA(1)**2.0
         SXXSTA(1) = SIGMA2*FS
         EFSTA(1) = SIGMA2*FE

C     Subr. 8 DBREAK computes the fraction of breaking waves and the
C     associated wave energy dissipation and returns DBSTA(1).

         CALL DBREAK(1, HRMS(1), H(1))

C ------------------------ MARCHING COMPUTATION ----------------------------

C     Computation marching landward from seaward boundary, J = 1, to MSL
C     shoreline, JMAX = (JSWL+1)

         DO 7777 J=1,JSWL
            JP1 = J + 1
```

```
         IF(JP1.GT.JMAX) THEN
            WRITE(*,2900) JMAX
            WRITE(21,2900) JMAX
            STOP
         ENDIF

 2900 FORMAT('ERROR: JP1 is greater than JMAX = ',I3)

         IF(XB(JP1).LE.XI) THEN
            DUM = (EFSTA(J) - DX*DBSTA(J))/FE

            IF(DUM.LE.0.0) THEN
               WRITE(*,2901)J
               WRITE(21,2901)J
               STOP
            ENDIF

 2901 FORMAT(/'ERROR: '/'Square of sigma is negative at node ',I3)

            SIGITE  = SQRT(DUM)

 110        SXXSTA(JP1) = FS*SIGITE**2.0

            WSETUP(JP1) = WSETUP(J) - (SXXSTA(JP1) - SXXSTA(J))/H(J)
            HITE = STORMT + WSETUP(JP1) - ZB(JP1)

C     Begin iteration for adopted implicit finite difference method

            DO 200 ITE = 1, MAXITE

               CALL LWAVE(JP1, HITE)
               HRMITE = SIGITE*SQR8
               SIGSTA(JP1) = SIGITE/HITE

               CALL SKEWKU(JP1,HRMITE/HITE)

               CALL CSFFSE(JP1,SIGSTA(JP1))

               CALL DBREAK(JP1, HRMITE, HITE)

               DUM = (EFSTA(J) - DX/2.0*( DBSTA(JP1) + DBSTA(J) ))/FE
```

```
            IF(DUM.LE.0.0) THEN
               WRITE(*,2901)J
               WRITE(21,2901)J
               STOP
            ENDIF

            SIGMA(JP1) = SQRT(DUM)
            SXXSTA(JP1) = FS*SIGMA(JP1)**2.0

            WSETUP(JP1) = WSETUP(J) - 2.0*
     +                    (SXXSTA(JP1)-SXXSTA(J))/(HITE+H(J))

            H(JP1) = STORMT + WSETUP(JP1) - ZB(JP1)

C     Check for convergence

            ESIGMA = ABS(SIGMA(JP1) - SIGITE)
            EH = ABS(H(JP1) - HITE)

            IF(ESIGMA.LT.EPS1.AND.EH.LT.EPS1) GO TO 210

C     Averages of new and previous values are used to accelerate convergence

            SIGITE =  0.50*(SIGMA(JP1) + SIGITE)
            HITE = 0.50*(H(JP1) + HITE)

 200         CONTINUE

          WRITE(*,2903) MAXITE, EPS1, JP1
          WRITE(21,2903) MAXITE, EPS1, JP1
          STOP

 2903 FORMAT(/'ERROR: Convergence was not reached after MAXITE = ',I4/
     +         ' iterations with relative error EPS1 = ',F8.6/
     +         'at node JP1 = ',I4)

 210         CONTINUE

          SIGSTA(JP1) = SIGMA(JP1)/H(JP1)
          HRMS(JP1) = SQR8*SIGMA(JP1)
```

```
          WSETUP(JP1) = H(JP1) + ZB(JP1) - STORMT
          IF(H(JP1).LT.EPS1.OR.JP1.EQ.JMAX) GO TO 400
          CALL LWAVE(JP1, H(JP1))
          CALL SKEWKU(JP1, HRMS(JP1)/H(JP1))
          CALL CSFFSE(JP1,SIGSTA(JP1))
          SIGMA2 = SIGMA(JP1)**2.0
          SXXSTA(JP1) = SIGMA2*FS
          EFSTA(JP1) = SIGMA2*FE

          CALL DBREAK(JP1,HRMS(JP1),H(JP1))

C     Check whether the inner zone is reached

          IF(QBREAK(JP1).EQ.1.00) THEN
             ICHECK = 0
             DO 220 JJ = JP1, JMAX
                IF(DZBDX(JJ).LE.0.0) ICHECK = ICHECK + 1
 220         CONTINUE
             IF(ICHECK.EQ.0) THEN
                XI = XB(JP1)
                JXI = JP1
             ENDIF
          ENDIF

          GO TO 7777

        ENDIF

C************************* End of IF(XB(JP1).LE.XI) ***************************

        IF(XB(JP1).GT.XI) THEN

C     If XB(J) = XI, compute SXXH2(J) and shoreline location XSTIDE
C     corresponding to storm tide STORMT

          IF(XB(J).EQ.XI) THEN
             SXXH2(J) = SXXSTA(J)/H(J)**2.0
             XSTIDE=XS+STORMT*DZBDX(JSWL+1)
          ENDIF

C     Empirical formula for HSTA = HRMS/H for region for XB.GE.XI
```

```
          HSTA = GAMMA + (GAMMAS - GAMMA)*
     +                   ((XB(JP1) - XI)/(XSTIDE - XI))**BETA
          SIGSTA(JP1) = HSTA/SQR8
          CALL SKEWKU(JP1, HSTA)
          DUM = SIGSTA(JP1)**2.0
          CS(JP1) = SIGSTA(JP1)*SKEW(JP1) - DUM
          CF(JP1) = 1.5*SKEW(JP1)*SIGSTA(JP1)*(1.0 - DUM) +
     +              0.5*DUM*(CURTO(JP1)- 5.0) + DUM**2.0

          C1 = SXXH2(J) + 2.0
          C2 = 3.0*SXXH2(J) + 2.0
          C3 = 2.0*(ZB(JP1) - ZB(J))
          HITE = H(J)
          DO 300 ITE = 1, MAXITE
             CALL LWAVE(JP1, HITE)
             FS = 2.0*WN(JP1) - 0.5 + CS(JP1)
             SXXH2(JP1) = DUM*FS
             H(JP1)=  (3.0*SXXH2(JP1)+C1)**(-1.0) *
     +                ((SXXH2(JP1)+C2)*H(J)-C3)
             IF(H(JP1).LE.0.0) THEN
                H(JP1) = 0.0
                GO TO 310
             ENDIF
             IF(ABS((H(JP1) - HITE)).LT.EPS1) GO TO 310
             HITE = H(JP1)

300       CONTINUE

          WRITE(*,2903) MAXITE, EPS1, JP1
          WRITE(21,2903) MAXITE, EPS1, JP1
          STOP

310       CONTINUE

          SIGMA(JP1) = H(JP1)*SIGSTA(JP1)
          HRMS(JP1) = SQR8*SIGMA(JP1)
          WSETUP(JP1) = H(JP1) + ZB(JP1) - STORMT
          IF(H(JP1).LT.EPS1.OR.JP1.EQ.JMAX) GO TO 400

          CALL LWAVE(JP1, H(JP1))
```

```
            CALL CSFFSE(JP1,SIGSTA(JP1))
            SIGMA2 = SIGMA(JP1)**2.0
            SXXSTA(JP1) = SIGMA2*FS
            SXXH2(JP1) = SXXSTA(JP1)/H(JP1)**2.0
            EFSTA(JP1) = SIGMA2*FE

            QBREAK(JP1) = 1.0

            GO TO 7777

         ENDIF

C************************* End of IF(XB(JP1).GT.XI) ***************************

 7777    CONTINUE

C --- END OF MARCHING COMPUTATION ---------------------------------------------



 400     CONTINUE

C     If JP1 is less than JMAX, no water at nodes JP1,...,JMAX

      IF(JP1.LT.JMAX) THEN
         DO 410 J=JP1,JMAX
            H(J)=0.0
            HRMS(J)=0.0
            WSETUP(J)=0.0
 410     CONTINUE
      ENDIF

C     After the first landward marching computation, Subr. 9, OUTPUT,
C     stores input and computed cross-shore variations for printout

         IF(IOUTP.EQ.1) THEN
            CALL OUTPUT
            IOUTP=0
         ENDIF

 7788    CONTINUE
```

```
C     For N500=0, go to the end of Main Program

            IF(N500.EQ.0) GO TO 9990

C     Store: WSETOE(K) = wave setup at toe node JTOE(k)
C             HTOE(K)   = total water depth at toe node JTOE(k)
C             HMOTOE(K) = HMO wave height at toe node JTOE(k)
C               with k=1,...,NTOE

            DO 500 K=1,NTOE
               M=JTOE(K)
               WSETOE(K)=WSETUP(M)
               HTOE(K)=H(M)
               HMOTOE(K)=HRMS(M)*SQR2

 500        CONTINUE

C     For File CSHORE2C** with **=N500:

            WRITE(15,2100) STORMT,HMOOFF, (WSETOE(K),HMOTOE(K),K=1,NTOE)

 2100 FORMAT(8F7.2)

C     For File CSHORE2D** with **=N500:

            IF(LTIMEL.EQ.1) THEN
               WRITE(16,2200) LSTORM,NTIMEL
            ENDIF
            WRITE(16,2100) TIME,TP, (HTOE(K),HMOTOE(K),K=1,NTOE)

 2200 FORMAT(2I3)

C     Find the maximum values of (storm tide + wave setup) and HMO wave height
C     at each toe depth during each storm

            IF(LTIMEL.EQ.1) THEN
               DO 550 K=1,NTOE
                  STSETM(K,LSTORM)=STORMT+WSETOE(K)
                  HMOMAX(K,LSTORM)=HMOTOE(K)
```

```
 550            CONTINUE
             ELSE
                DO 560 K=1,NTOE
                   DUM=STORMT+WSETOE(K)
                   IF(STSETM(K,LSTORM).LT.DUM) STSETM(K,LSTORM)=DUM
                   DUM=HMOTOE(K)
                   IF(HMOMAX(K,LSTORM).LT.DUM) HMOMAX(K,LSTORM)=DUM
 560            CONTINUE
             ENDIF


 8888 CONTINUE

      IF(LSTORM.EQ.1) THEN
         WRITE(7,2251)
      ENDIF

 2251 FORMAT(/'MAXIMUM WATER LEVEL AND HMO WAVE HEIGHT FOR EACH STORM'/)

      WRITE(7,2250) LSTORM,(STSETM(K,LSTORM),HMOMAX(K,LSTORM),K=1,NTOE)

 2250 FORMAT(I5,6F7.2)


C ... END OF NTIMEL TIME LEVELS FOR EACH STORM ................................

 9999 CONTINUE

C ---END OF NSTORM STORMS -----------------------------------------------

C     Rank NSTORM values of STSETM and HMOMAX using Subr. 10, SHELL

      DO 600 K=1,NTOE
         DO 610 L=1,NSTORM
            ARRAY(L)=STSETM(K,L)
 610     CONTINUE
         CALL SHELL(NSTORM,ARRAY)
         DO 620 L=1,NSTORM
            STSETM(K,L)=ARRAY(L)
 620     CONTINUE
         DO 630 L=1,NSTORM
```

```
          ARRAY(L)=HMOMAX(K,L)
 630    CONTINUE
        CALL SHELL(NSTORM,ARRAY)
        DO 640 L=1,NSTORM
          HMOMAX(K,L)=ARRAY(L)
 640    CONTINUE
 600  CONTINUE

C     Output the ranked values with the corresponding recurrence interval
C     REYEAR in File OUTCSHORE2 for subsequent plotting

      WRITE(7,2301)
 2301 FORMAT(/'RANKED VALUES FOR ALL STORMS'/)


      DO 700 L=1,NSTORM
        REYEAR=FLOAT(NYEAR)/FLOAT(L)

C     Store (500/5)=100 year maximum storm setup and significant wave height
C     separately

        IF(L.EQ.5) THEN
          WRITE(8,888) N500,REYEAR,(STSETM(K,L),HMOMAX(K,L),K=1,NTOE)
        ENDIF

        WRITE(7,2300) REYEAR, (STSETM(K,L),HMOMAX(K,L),K=1,NTOE)

 700  CONTINUE

 2300 FORMAT(F8.2,6F7.2)
 888  FORMAT(I3,F8.2,6F7.2)

 9990 CONTINUE

      END

C--00-------------------- END OF MAIN PROGRAM -----------------------------

C##01###################### SUBROUTINE OPENER ###############################
C
C     This subroutine opens all input and output files
```

```
C
C-------------------------------------------------------------------------------

      SUBROUTINE OPENER(FBOTTOM,N500)

      CHARACTER*10 FBOTTOM,FSAVEB(10),FSAVEC(10),FSAVED(10)
      DATA FSAVEB/
     1'CYCLONEB01','CYCLONEB02','CYCLONEB03','CYCLONEB04',
     2'CYCLONEB05','CYCLONEB06','CYCLONEB07','CYCLONEB08',
     3'CYCLONEB09','CYCLONEB10'/
      DATA FSAVEC/
     1'CSHORE2C01','CSHORE2C02','CSHORE2C03','CSHORE2C04',
     2'CSHORE2C05','CSHORE2C06','CSHORE2C07','CSHORE2C08',
     3'CSHORE2C09','CSHORE2C10'/
      DATA FSAVED/
     1'CSHORE2D01','CSHORE2D02','CSHORE2D03','CSHORE2D04',
     2'CSHORE2D05','CSHORE2D06','CSHORE2D07','CSHORE2D08',
     3'CSHORE2D09','CSHORE2D10'/


C     Open Input File FBOTTOM and Output File ODOC for concise documentation

      OPEN (UNIT=2,FILE=FBOTTOM,STATUS='OLD',ACCESS='SEQUENTIAL')
      OPEN (UNIT=21,FILE='ODOC',STATUS='NEW',ACCESS='SEQUENTIAL')

C     For N500=1,2,...,10, open one input file and four output files

      IF(N500.GE.1.AND.N500.LE.10) THEN
         OPEN(UNIT=4,FILE=FSAVEB(N500),STATUS='OLD',ACCESS='SEQUENTIAL')
         OPEN(UNIT=15,FILE=FSAVEC(N500),STATUS='NEW',ACCESS=
     +'SEQUENTIAL')
         OPEN(UNIT=16,FILE=FSAVED(N500),STATUS='NEW',ACCESS=
     +'SEQUENTIAL')
         OPEN(UNIT=7,FILE='OUTCSHORE2',STATUS='NEW',ACCESS=
     +                    'SEQUENTIAL')
         OPEN(UNIT=8,FILE='CSHORE2DATA',STATUS='OLD',ACCESS=
     +                    'APPEND')

      ENDIF

      RETURN
```

```
      END

C--01------------------- END OF SUBROUTINE OPENER -------------------------

C##02####################### SUBROUTINE INPUT ################################
C
C     This subroutine reads data from primary input data file
C
C-----------------------------------------------------------------------------


      SUBROUTINE INPUTB(N500)

      INTEGER NN, NB
      PARAMETER (NN=1000, NB=30)

      CHARACTER*5 COMMEN(14)

      COMMON /TOEDEP/ NTOE, TOED(3), JTOE(3)
      COMMON /BINPUT/ XBINP(NB), ZBINP(NB), NBINP, JSWL

C ... COMMENT LINES ...........................................................
C     NLINES = number of comment lines preceding input data

      READ (2,1110) NLINES

      DO 110 I = 1,NLINES
         READ  (2,1120) (COMMEN(J),J=1,14)
         WRITE (21,1120) (COMMEN(J),J=1,14)
         WRITE (*,*) (COMMEN(J),J=1,14)
 110  CONTINUE


C ... COMPUTATIONAL INPUT DATA ...............................................
C
C     JSWL = number of spatial nodes along the bottom below SWL used to
C            determine nodal spacing DX for given bottom geometry.
C
C     Note : JSWL should be so large that delta x between two adjacent
C            nodes is sufficiently small.
```

```
      READ (2,1110) JSWL

C ... BOTTOM GEOMETRY ...............................................
C
C     The bottom geometry is divided into segments of different inclination
C     starting from seaward boundary.
C
C     NBINP     = number of input bottom points
C     XBINP(J)  = horizontal distance to input bottom point (J) in meters
C                 where XBINP(1) = 0 at the seaward boundary
C     ZBINP(J)  = dimensional vertical coordinate (+ above SWL)of input
C                 bottom point (J) in meters


      READ (2,1110) NBINP
      IF(NBINP.GT.NB) THEN
         WRITE(*,2900) NBINP, NB
         WRITE(21,2900) NBINP, NB
         STOP
      ENDIF

 2900 FORMAT(/'Number of Input Bottom Nodes NBINP = ',I8,' ;NB = ',I8/
     +        'Increase PARAMETER NB.')


      READ (2,1150) (XBINP(J), ZBINP(J), J=1,NBINP)

      XBINP(1)=0.0

C     For N500=1,2,..., or 10, input NTOE(1,2, or 3) toe depths, TOED
C     (positive) of hypothetical coastal structures where the computed
C     time series of wave setup and height for each of NSTORM storms
C     are stored. Specify TOED(K) > TOED(K+1)

      IF(N500.EQ.0) NTOE=0
      IF(N500.GE.1.AND.N500.LE.10) THEN
         READ(2,1110) NTOE
         READ(2,1160) (TOED(K),K=1,NTOE)
      ENDIF
```

```
      CLOSE (2)

 1110 FORMAT (I4)
 1120 FORMAT (14A5)
 1150 FORMAT (F13.6)
 1160 FORMAT (3F10.3)

      RETURN
      END


C--02-------------------- END OF SUBROUTINE INPUT -------------------------

C##03####################### SUBROUTINE BOTTOM #############################
C
C     This subroutine calculates the bottom geometry and DX between two
C     adjacent nodes
C
C-----------------------------------------------------------------------------

      SUBROUTINE BOTTOM

      PARAMETER (NN=1000, NB=30)
      DIMENSION SLOPE(NB)

      COMMON /TOEDEP/ NTOE, TOED(3), JTOE(3)
      COMMON /BINPUT/ XBINP(NB), ZBINP(NB), NBINP, JSWL
      COMMON /BPROFL/ DX, XB(NN), ZB(NN), DZBDX(NN), JMAX
      COMMON /BRKNEW/ GAMMAS, BETA, XS, XI

C     XS =  dimensional horizontal distance between seaward boundary
C           and initial shoreline at SWL
C
C     The structure geometry is divided into segments of different inclination
C
C     NBINP = number of input bottom points
C
C     For segments starting from the seaward boundary:
C
C     SLOPE(K)  = slope of segment K(+ upslope, - downslope)
```

```
C     XBINP(K)  = dimensional horizontal distance from seaward boundary
C                 to the seaward-end of segment K
C     ZBINP(K)  = dimensional vertical coordinate (+ above SWL)
C                 at the seaward-end of segment K

      DO 120 K = 1,NBINP-1
         SLOPE(K) = (ZBINP(K+1)-ZBINP(K))/(XBINP(K+1)-XBINP(K))
 120  CONTINUE

C ... CALCULATE GRID SPACING DX BETWEEN TWO ADJACENT NODES ..................
C
C     The value of JSWL specified as input corresponds to
C     number of nodes along the bottom below SWL.

      K = 0
 900  CONTINUE
      IF (K.EQ.NBINP) THEN
         WRITE(*,2900)
         WRITE(21,2900)
         STOP
      ENDIF
      K = K+1
      CROSS = ZBINP(K)*ZBINP(K+1)
      IF (CROSS.GT.0.0) GOTO 900
      XS  = XBINP(K+1) - ZBINP(K+1)/SLOPE(K)
      DX    = XS/FLOAT(JSWL)

 2900 FORMAT(/'Bottom is always below SWL.'/
     +          'There is no still water shoreline.')


C ... CALCULATE BOTTOM GEOMETRY AT EACH NODE ................................
C
C     JMAX = maximum node number for computation
C     XB(J)= horizontal coordinate of node j where XB(1) = 0
C     ZB(J)= vertical coordinate of bottom at node j (+ above SWL)
C     SLOPE(K) = tangent of local slope of segment K
C
C     Computation in CSHORE2 is limited to JMAX=(JSWL+1), so that depth
C     at node JMAX is equal to zero in absence of storm tide
```

```
      JMAX   = JSWL+1
      IF (JMAX.GT.NN) THEN
         WRITE  (*,2910) JMAX,NN
         WRITE (21,2910) JMAX,NN
         STOP
      ENDIF

 2910 FORMAT (/' End Node =',I8,'; NN =',I8/
     +         ' Bottom length is too long.'/
     +         ' Cut it, or change PARAMETER NN.')

      DIST = -DX
      K    = 1
      XCUM = XBINP(K+1)
      DO 140 J = 1,JMAX
         DIST = DIST + DX
         IF (DIST.GT.XCUM.AND.K.LT.NBINP) THEN
            K    = K+1
            XCUM = XBINP(K+1)
         ENDIF
         ZB(J) = ZBINP(K) + (DIST-XBINP(K))*SLOPE(K)
         XB(J) = DIST
         DZBDX(J) = SLOPE(K)

 140  CONTINUE

C     If NTOE=1-3, find nodal locations JTOE(K) corresponding to toe depths
C     (positive) TOED(K), with K=1-NTOE, where ZB is negative below SWL and
C     increases landward at toe depth TOED(K)

      IF(NTOE.GE.1) THEN
         DO 200 K=1,NTOE
            J=JMAX
 300        J=J-1
            DUM1=TOED(K)+ZB(J)
            IF(DUM1.GT.0.0) GO TO 300
            JTOE(K)=J
            DUM2=TOED(K)+ZB(J+1)
            IF(DUM2.LT.(-0.001)) THEN
               WRITE(*,2920) ZB(J), ZB(J+1), TOED(K)
```

```
            WRITE(21,2920) ZB(J), ZB(J+1), TOED(K)
            STOP
          ENDIF
          IF(ABS(DUM1).GT.DUM2) JTOE(K)=J+1
 200    CONTINUE
      ENDIF

 2920 FORMAT(/'ZB(J)=',F8.3,'ZB(J+1)=',F8.3/
     +       'Toe Depth TOED(K)=',F8.3/
     +       'Corresponding Node JTOE(K) can not be found'/
     +       'Check Input Bathymetry and toe depth.')


C     Set XI = XS until XI is found in Main Program.

      XI = XS

      RETURN
      END


C--03------------------- END OF SUBROUTINE BOTTOM ------------------------

C##04######################## SUBROUTINE PARAM ############################
C
C     This subroutine calculates parameters used in other subroutines
C
C-----------------------------------------------------------------------

      SUBROUTINE PARAM

      COMMON /CONSTA/ GRAV, SQR8, SQR2, PI, TWOPI

C ... CONSTANTS and PARAMETERS ..........................................
C
C     PI      = 3.14159
C     TWOPI   = 2.0 * PI
C     GRAV    = acceleration due to gravity
C     SQR8    = Sqrt(8)
C     SQR2    = Sqrt(2)
```

```
      PI = 3.141590
      TWOPI = 2.0*PI
      GRAV = 9.81
      SQR8 = SQRT(8.0)
      SQR2 = SQRT(2.0)

      RETURN
      END

C--04-------------------- END OF SUBROUTINE PARAM -------------------------

C##05####################### SUBROUTINE LWAVE ##############################
C
C     This subroutine calculates quantities based on linear wave theory
C
C--------------------------------------------------------------------------

      SUBROUTINE LWAVE(J, WD)

      PARAMETER (NN=1000)
      COMMON /PERIOD/ FP, WKPO
      COMMON /CONSTA/ GRAV, SQR8, SQR2, PI, TWOPI
      COMMON /LINEAR/ WKP, CP, WN(NN)


C ... LINEAR WAVE PARAMETERS ..............................................
C
C     WKPO = deep water wave number for FP
C     FP   = peak frequency
C     CP   = phase velocity of peak frequency
C     WN   = ratio of group velocity to phase velocity


      D = WD*WKPO
      IF(J.EQ.1) THEN
         X = D/SQRT(TANH(D))
      ELSE
         X = WKP*WD
      ENDIF
 10   COTH = 1.0/TANH(X)
```

```
      XNEW = X - (X-D*COTH)/(1.0+D*(COTH**2.0-1.0))
      IF (ABS(XNEW - X).GT.1.E-5) THEN
         X = XNEW
         GOTO 10
      ENDIF
      WKP = X/WD
      WN(J) = 0.5*(1.0 + 2.0*X/(SINH(2.0*X)))
      CP = TWOPI*FP/WKP

      RETURN
      END


C--05-------------------- END OF SUBROUTINE  LWAVE -------------------------

C##06######################### SUBROUTINE SKEWKU ##############################
C
C     This subroutine calculates skewness and kurtosis for given HSTA
C
C---------------------------------------------------------------------------

      SUBROUTINE SKEWKU(J,HSTA)

      PARAMETER (NN=1000)

      COMMON /NONLIN/ SKEW(NN), CURTO(NN)
      DATA A, B, C, Y1, Y2 / 2.0, 1.0, 0.7, 0.5, 1.0/


C ... SKEWNESS AND KURTOSIS OF THE FREE SURFACE ..............................
C
C     HSTA = ratio of root-mean-square wave height to mean water depth
C     SKEW(J)  = skewness of the free surface
C     CURTO(J) = kurtosis of the free surface

      IF(HSTA.LE.Y1) THEN
         SKEW(J) = A*HSTA
      ELSEIF(HSTA.LE.Y2) THEN
         SKEW(J) = A*Y1 - B*(HSTA - Y1)
      ELSE
         SKEW(J) = A*Y1 - B*(Y2 - Y1) + C*(HSTA - Y2)
```

```
      ENDIF
      CURTO(J) = 3.0 + SKEW(J)**2.2

      RETURN
      END

C --06-------------------- END OF SUBROUTINE SKEWKU ----------------------

C##07######################### SUBROUTINE CSFFSE ############################
C
C     This subroutine computes CS, CF, FS, and FE
C
C--------------------------------------------------------------------------

      SUBROUTINE CSFFSE(J,SSTA)

      PARAMETER (NN=1000)

      COMMON /LINEAR/ WKP, CP, WN(NN)
      COMMON /NONLIN/ SKEW(NN), CURTO(NN)
      COMMON /MOMENT/ CS(NN), FS, SXXSTA(NN)
      COMMON /ENERGY/ CF(NN), FE, EFSTA(NN)

      CS(J) = SSTA*SKEW(J) - SSTA**2.0
      CF(J) = 1.5*SKEW(J)*SSTA*(1.0 - SSTA**2.0) +
     +        0.5*SSTA**2.0*(CURTO(J) - 5.0) + SSTA**4.0
      FS = (2.0*WN(J) - 0.5) + CS(J)
      FE = WN(J)*CP*(1.0 + CF(J))


      RETURN
      END


C--07-------------------- END OF SUBROUTINE CSFFSE ----------------------

C##08######################### SUBROUTINE DBREAK ############################
C
C     This subroutine calculates QBREAK and DBSTA for wave breaking in
C     region of XB < XI
```

```
C
C-----------------------------------------------------------------------------

      SUBROUTINE DBREAK(J, WHRMS, D)

      PARAMETER (NN=1000, NB=30)

      COMMON /PERIOD/ FP, WKPO
      COMMON /LINEAR/ WKP, CP, WN(NN)
      COMMON /WBREAK/ ALPHA, GAMMA, QBREAK(NN), DBSTA(NN)
      COMMON /CONSTA/ GRAV, SQR8, SQR2, PI, TWOPI

      IF(J.EQ.1) THEN
         S0 = WHRMS*TWOPI*(FP**2.0)/GRAV *
     +        SQRT(TANH(WKP*D)*(1.0 + 2.0*WKP*D/SINH(2.0*WKP*D)))
         GAMMA = 0.5+0.40*TANH(33.0*S0)
      ENDIF

C ... FRACTION OF BREAKING WAVES AND ASSOCIATED DISSIPATION ..................
C
C     QBREAK(J) = Fraction of breaking waves at node J
C     DBSTA(J)  = Time averaged normalized energy dissipation due to wave
C                 breaking at node J


      HM = 0.88/WKP*TANH(GAMMA*WKP*D/0.88)

      NITER=0

      B = (WHRMS/HM)**2.0
      IF(B.LT.0.99999) THEN
         QBOLD = B/2.0
 10      QBREAK(J) = QBOLD - (1.0-QBOLD + B*ALOG(QBOLD))/(B/QBOLD-1.0)
         IF(QBREAK(J).LE.0.0) QBREAK(J) = QBOLD/2.0
         IF(ABS(QBREAK(J)-QBOLD).GT.0.00001) THEN
            NITER=NITER+1
            IF(NITER.GT.100) THEN
               WRITE(21,2900) B, QBOLD, QBREAK(J)
               WRITE(*,2900) B, QBOLD, QBREAK(J)
               STOP
            ENDIF
```

```
            QBOLD = QBREAK(J)
            GOTO 10
         ENDIF
      ELSE
         QBREAK(J) = 1.0
      ENDIF


      DBSTA(J) = 0.25*ALPHA*QBREAK(J)*FP*HM**2.0


 2900 FORMAT(/' B=',F9.6,' QBOLD=',F9.6,' QBREAK(J)=',F9.6/'Iteration
     &       did not converge after 100 iterations in Subr.08 DBREAK')

      RETURN
      END


C--08-------------------- END OF SUBROUTINE DBREAK -------------------------

C##09######################## SUBROUTINE OUTPUT ###############################
C
C     This subroutine stores computed and input quantities
C
C-----------------------------------------------------------------------------

      SUBROUTINE OUTPUT

      PARAMETER (NN=1000, NB=30)

      COMMON /SYEARS/ N500, NYEAR, NSTORM
      COMMON /SEABC/ TIME, STORMT, HMOOFF, TP
      COMMON /TOEDEP/ NTOE, TOED(3), JTOE(3)
      COMMON /PERIOD/ FP, WKPO
      COMMON /PREDIC/ HRMS(NN), SIGMA(NN), H(NN), WSETUP(NN), SIGSTA(NN)
      COMMON /BINPUT/ XBINP(NB), ZBINP(NB), NBINP, JSWL
      COMMON /BPROFL/ DX, XB(NN), ZB(NN), DZBDX(NN), JMAX
      COMMON /WBREAK/ ALPHA, GAMMA, QBREAK(NN), DBSTA(NN)
      COMMON /BRKNEW/ GAMMAS, BETA, XS, XI
      COMMON /ITERAT/ EPS1, MAXITE
```

```
C ... INPUT SIMULATION SPECIFICATION .....................................
C
C     N500   = N500-th simulation of 500 years
C     NYEAR  = Number of simulated years
C     NSTORM = Number of storms

      WRITE(21,900) N500, NYEAR, NSTORM

 900  FORMAT(/'INPUT SIMULATION SPECIFICATION:'/
     +        'N500-th simulation of 500 years; N500   =',I2/
     +        'Number of simulated years;       NYEAR  =',I3/
     +        'Number of storms;                NSTORM =',I3)


C ... INPUT WAVE PROPERTIES FOR FIRST STORM (LSTORM=1) AND FIRST TIME
C     LEVEL (LTIMEL=1)
C
C     TP        = spectral peak period in seconds
C     HRMS(1)   = root mean square wave height at seaward boundary in meters
C     WSETUP(1) = wave setup at seaward boundary in meters

      WRITE (21,1000) TP, FP, HRMS(1), WSETUP(1), STORMT
 1000 FORMAT (/ 'INPUT WAVE PROPERTIES FOR FIRST LANDWARD MARCHING
     +          COMPUTATION WITH LSTORM=1 AND LTIMEL=1:'/
     +         'Peak wave period (sec)              =',E13.6/
     +         'Peak frequency (1/sec)              =',E13.6/
     +         'Root-mean-square wave height '/
     +         '            at seaward boundary (m) =',E13.6/
     +         'Wave setup at seaward boundary (m) =',E13.6/
     +         'Storm tide at seaward boundary (m) =',E13.6)

C ... OUTPUT BOTTOM GEOMETRY ..............................................
C
C     The bottom geometry is divided into segments of different inclination
C     and roughness starting from seaward boundary.
C
C     NBINP     = number of segments
C     XBINP(J)  = horizontal distance from seaward boundary to landward-end
C                 of segment (J-1) in meters
C     ZBINP(J)  = dimensional vertical coordinate (+ above SWL)of the landward
C                 end of segment (J-1) in meters
```

```
      WRITE (21,1100)  0.0-ZBINP(1), NBINP-1, JSWL, DX, JMAX
C
 1100 FORMAT (/'INPUT BOTTOM GEOMETRY'/
     +          'Depth at seaward boundary (m)        =',F13.6/
     +          'Number of linear segments            =',I8/
     +          'Number of spatial nodes  below'/
     +          '    SWL used to find DX              =',I8/
     +          'Node spaceing, DX (m)                =',F13.6/
     +          'Maximum landward node          JMAX =',I8//
     +          '        X (m)              Zb (m)')

      DO 140 J = 1,NBINP
         WRITE (21,1200) XBINP(J), ZBINP(J)
 140  CONTINUE

 1200 FORMAT(2(F13.6,5X))

C ... TOE DEPTHS OF HYPOTHETICAL COASTAL STRUCTURES ........................
C
C     NTOE    = number (1,2 or 3) toe depths with K=1-NTOE
C     TOED(K) = K-th toe depth below MSL
C     JTOE(K) = node location corresponding to TOED(K)

      IF(NTOE.GE.1) THEN
         WRITE(21,1500) NTOE
         DO 150 K=1,NTOE
            WRITE(21,1510) K, JTOE(K), TOED(K)
 150     CONTINUE
      ENDIF

 1500 FORMAT(/'TOE DEPTHS OF COASTAL STRUCTURES:'/
     +          'Number of toe depth;      NTOE=',I1)
 1510 FORMAT(/'***',I1,'-th toe depth at node=',I3,'  depth='
     +       ,F5.2,' m below MSL')


C ... EMPIRICAL PARAMETERS FOR WAVE BREAKING ...............................

      WRITE (21,1300) ALPHA, GAMMAS, BETA, XS
```

```
 1300 FORMAT(/'EMPIRICAL PARAMETERS FOR WAVE BREAKING'/
     +          'Alpha  = ',F13.6/
     +          'Gammas = ',F13.6/
     +          'Beta   = ',F13.6/
     +          'Xs(m)  = ',F13.6)

C ... ITERATION PARAMETERS ..................................................

      WRITE (21,1400) EPS1, MAXITE

 1400 FORMAT(/'ITERATION PARAMETERS'/
     +          'Allowable relative error in iterated depth(m)  =',F13.6/
     +          'Maximum iterations allowed =  ',I8)

C ... CROSS-SHORE VARIATIONS OF WAVE SETUP AND HEIGHT ........................
C     for the first landward marching computation with LSTORM=1 and LTIMEL=1
C     to check the computed cross-shore variations

      WRITE(21,1600)
      DO 160 J=1,JMAX
         WRITE(21,1610) J, XB(J), 0.0-ZB(J), H(J), WSETUP(J), HRMS(J)
 160  CONTINUE
 1600 FORMAT(/'FIRST LANDWARD MARCHING COMPUTATION'/
     +          'Computed Cross-Shore Variations'/
     + 'NODE    XB(m)   -ZB(m)    H(m)  WSETUP(m)  HRMS(m)'/)
 1610 FORMAT(I4,F9.2,F9.3,F8.3,F11.3,F9.3)

      RETURN
      END


C--09------------------- END OF SUBROUTINE OUTPUT ------------------------

C##10####################### SUBROUTINE SHELL ##############################

C------------------------------------------------------------------------
C     THIS SUBROUTINE SORTS AN ARRAY, ARR, OF LENGTH N INTO DESCENDING
C     NUMERICAL ORDER, BY THE SHELL-MEZGAR ALGORITHM. N IS INPUT; ARR IS
C     REPLACED ON OUTPUT BY ITS SORTED REARRANGEMENT
C------------------------------------------------------------------------
```

```
      SUBROUTINE SHELL(N,ARR)

      PARAMETER(ALN2I=1./0.69314718, TINY=1.E-5)
      DIMENSION ARR(N)
      LOGNB2=INT(ALOG(FLOAT(N))*ALN2I+TINY)
      M=N
      DO 12 NN=1,LOGNB2
         M=M/2
         K=N-M
         DO 11 J=1,K
            I=J
 3          CONTINUE
            L=I+M
            IF(ARR(L).GT.ARR(I)) THEN
               T=ARR(I)
               ARR(I)=ARR(L)
               ARR(L)=T
               I=I-M
               IF(I.GE.1) GO TO 3
            ENDIF

 11      CONTINUE
 12   CONTINUE

      RETURN
      END

C**************************  END OF SUBROUTINES  ******************************
```

Appendix C

COMPUTER PROGRAM OVERTOP

```
C
C    ######    ##          ##   #######   ######   ########   ######    ######
C   ##    ##    ##        ##    ##       ##    ##     ##     ##    ##  ##    ##
C   ##    ##     ##      ##     ##       ##    ##     ##     ##    ##  ##    ##
C   ##    ##      ##    ##      ######   #######      ##     ##    ##  #######
C   ##    ##       ##  ##       ##       ##  ##       ##     ##    ##  ##
C   ##    ##        ####        ##       ##   ##      ##     ##    ##  ##
C    ######          ##         #######  ##    ##     ##      ######   ##
C
C
C
C
C                                   # # #
C                                 #       #
C                               #          #
C                             #             #
C                           #           ##   #
C                         #           #    ####
C                       #           #      -------------
C                     #           #      / \           / \
C              ~~~~~ #              # ~~/ \             / \~~~~~~~~~~~~~
C                                      / \               / \
C                                     / \                 / \
C              ----------------------/ \                   / \----------
C              //////////////////////                        ////////////
C
C
C*****************************************************************************
C*                                                                           *
C*                   BEATRIZ POZUETA AND NOBUHISA KOBAYASHI                  *
C*                                                                           *
C*                     CENTER FOR APPLIED COASTAL RESEARCH                   *
C*                                                                           *
C*                                                                           *
C*                University of Delaware, Newark, Delaware 19716             *
C*                                                                           *
C*                                March 2002                                 *
C*                                                                           *
C*****************************************************************************
```

```
C**************************************************************************
C*00*********************  MAIN PROGRAM  *********************************
C
C
C
C**************************************************************************

      PROGRAM OVERTOP
      DIMENSION QMAX(500,3,2), VSTORM(500,3,2)
      DIMENSION ARRAY(500)
      DIMENSION H(3), HMOTOE(3), HTOE(3), WSETOE(3)
      DIMENSION GAMMA(3)

C ... COMMONs

C       Name     Contents
C     --------------------------------------------------------------
C     /DESIGN100/ 100-year design conditions
C     /DESIGN/    Values at each time level during 500-year simulation
C     /TOEDEP/    Toe depths of hypothetical coastal structures
C     /CONSTA/    Constants
C     /PARAM100/  Parameters for 100-year design conditions
C     /PARAM/     Parameters at each time level
C     /FACT/      Structure crest heights above local bottoms
C     /RATES/     Average overtopping rates

      COMMON /DESIGN100/ DS100(3), HS100(3), TP100(3), GAMMA100(3)
      COMMON /DESIGN/    DS(3), HS(3), TP
      COMMON /TOEDEP/    NTOE, TOED(3), D(3)
      COMMON /CONSTA/    GRAV, PI, TPIG, GAMMAB, GAMMABE, TALPHA
      COMMON /PARAM100/  SOP100(3), TSIOP100(3)
      COMMON /PARAM/     SOP(3), TSIOP(3)
      COMMON /FACT/      RCDS(3,2)
      COMMON /RATES/     Q(2), QQ(3,2)

C ... INPUT N500, NYEAR AND NSTORM ON SCREEN ..............................

      WRITE(*,*)
     +'Specify Integer N500 for N500-th simulation of 500 years'
```

```
      READ(*,*) N500

C     N500 must be in the range of 1-10

      IF(N500.LT.1.OR.N500.GT.10) WRITE(*,*)
     +'N500 must be in the range 1-10'

      IF(N500.GE.1.AND.N500.LE.10) THEN
         NYEAR=500
         WRITE(*,*)
     +   'Specify Number of Storms for this 500-Year Simulation'
         READ(*,*) NSTORM
         IF(NSTORM.GT.NYEAR) WRITE(*,*)
     +   'Increase 500 in QMAX, VSTORM and ARRAY'
      ENDIF

C     NTSUBM = number of time steps of 0.5hr during which the crest of the
C     structure is submerged

      NTSUBM=0


C ... NTOE=(1,2 or 3)  toe depths, TOED (positive) of hypothetical coastal
C     structures where the computed time series of wave setup and height
C     for each of NSTORM storms are stored.

      WRITE(*,*)
     +'Specify number of toe depths of hypothetical coastal structures'
      READ(*,*) NTOE

      WRITE(*,*)
     +'Specify toe depths of hypothetical coastal structures'
      READ(*,*) (TOED(K),K=1,NTOE)


C ... INPUT SEAWARD STRUCTURE SLOPE ON SCREEN ..................................

      WRITE(*,*)
     +'Specify Seaward Structure Slope TAN(alpha)'
      READ(*,*) TALPHA
```

```
C ... INPUT LOW AND HIGH ALLOWABLE OVERTOPPING RATE FOR UNIT WIDTH ...........

C     Specify low value of overtopping rate m2/s
      Q(1)=0.001

C     Specify high value of overtopping rate m2/s
      Q(2)=0.01

C     Subr. 1 OPENER opens input and output files.

      CALL OPENER(N500)

C     Subr. 2 INPUTB gets input data and calculates design values for 100-year
C     storm peak

      CALL INPUTB

C     Subr. 4 PARAMETER  calculates constants and parameters.

      CALL PARAMETER

C     Subr. 5 REDFACT1 calculates the reduction factor for influence of shallow
C     foreshore, roughness and combined reduction factor for 100-YR design
C     conditions

      CALL REDFACT1

C     Subr. 6 CREST1 calculates the crest heights above still water level
C     for the specified Q(1) and Q(2) under design conditions

      CALL CREST1

C --- COMPUTATIONS FOR NSTORM STORMS ---------------------------------------
C     Time step of 0.5 hr=1800 s, was used for computation of storm surge
C     and waves

      TSTEP=1800

      DO 9999 LSTORM=1,NSTORM
```

```
C     Read from file CSHORE2D** with **=N500
C     NTIMEL= number of time levels for each storm

        READ(16,2200) LL,NTIMEL
        IF(LL.NE.LSTORM) THEN
           WRITE(*,2800) LL,LSTORM
           WRITE(21,2800) LL,LSTORM
           STOP
        ENDIF

 2200 FORMAT(2I3)
 2800 FORMAT(/'ERROR:'/
     +       'Storm number from File CSHORE2D**; LL=',I3/
     +       'NOT SAME as LSTORM=',I3,' in DO LOOP')

        DO 9991 L=1,2
           DO 9992 K=1,NTOE
             QMAX(LSTORM,K,L)=0.0
             VSTORM(LSTORM,K,L)=0.0
 9992      CONTINUE
 9991   CONTINUE


C ... COMPUTATION FOR NTIMEL TIME LEVELS FOR EACH STORM ......................

        DO 8888 LTIMEL=1,NTIMEL


C     Read from File CSHORE2D** with **=N500
C     TIME      = time in hours during each storm
C     TP        = spectral peak period (in seconds)
C     HTOE(K)   = total water depth at toe node JTOE(k)
C     HMOTOE(K) = HMO wave height at toe node JTOE(k)
C                   with k=1,...,NTOE

           READ(16,2100) TIME,TP, (HTOE(K),HMOTOE(K),K=1,NTOE)

 2100 FORMAT(8F7.2)

C     Read from File CSHORE2C** with **=N500
C     STORMT    = storm tide in meters
```

```
C      HMOOFF    = offshore significant wave height in meters
C      WSETOE(K) = wave setup at toe node JTOE(k)

            READ(15,2100) STORMT,HMOOFF, (WSETOE(K),HMOTOE(K),K=1,NTOE)

            DO 7777 K=1,NTOE

               H(K)=HTOE(K)
               DS(K)=TOED(K)+STORMT
               HS(K)=HMOTOE(K)

               IF(HS(K).LT.10E-3) THEN
                  WRITE(*,*) LSTORM,LTIMEL,K
                  DO 7771 L=1,2
                     QQ(K,L)=0.0
 7771             CONTINUE
                  GO TO 7777
               ENDIF

C ... Wave Steepnes based on deepwater wavelength ............................

               SOP(K)=TPIG*HS(K)/(TP*TP)

C ... Surf similarity parameter ..............................................

               TSIOP(K)=TALPHA/SQRT(SOP(K))

C      Subr. 5 REDFACT2 calculates the combined reduction factor GAMMA(K)

               CALL REDFACT2(K,GAMMA(K))

               DO 6666 L=1,2

                  IF(RCDS(K,L).LT.H(K)) THEN
                     NTSUBM=NTSUBM+1
                     WRITE(*,2400) NTSUBM,RCDS(K,L),H(K),LSTORM,LTIMEL,K,L
                     WRITE(21,2400) NTSUBM,RCDS(K,L),H(K),LSTORM,LTIMEL,K,L
                  ENDIF

 2400 FORMAT(/'STRUCTURE CREST SUBMERGED:'/
     +        'NTSUBM-th time step for submergency, NTSUBM=',I3/
```

```
     +        'Strucutre crest height above local bottom; RCDS=',F7.2/
     +        'LESS THAN mean water depth=',F7.2/
     +        'for LSTORM=',I3,', LTIMEL=',I3,', K=',I1,' and L=',I1)


C     Calculation of the overtopping rate QQ(K,L) with L=1 and 2
C     for the sequences of storms during 500 years

               IF(RCDS(K,L).GE.H(K)) THEN

                  RRC=RCDS(K,L)-DS(K)

                  IF(TSIOP(K).LE.2.0) THEN

                     RRB=RRC*SQRT(SOP(K))/(HS(K)*TALPHA*GAMMA(K))
                     IF(RRB.GT.10.0) THEN
                        QQB=0.0
                     ELSE
                        QQB=0.06*EXP(-5.2*RRB)
                     ENDIF

                     QQ(K,L)=QQB*SQRT(GRAV*HS(K)**3.0)*SQRT(TALPHA/SOP(K))

                  ELSE

                     RRN=RRC/(HS(K)*GAMMA(K))
                     IF(RRN.GT.20.0) THEN
                        QQN=0.0
                     ELSE
                        QQN=0.2*EXP(-2.6*RRN)
                     ENDIF

                        QQ(K,L)=QQN*SQRT(GRAV*HS(K)**3.0)

                  ENDIF

               ELSE

C     CDEPTH = Water depth on structure during submergency
C     CDEPTH = Mean water depth(H(K)) - Structure crest height(RCDS(K,L))
C     ASSUME:
```

```
C        Overtopping rate, QQ = SQRT[g*(h-crest height)]*(h-crest height)


                   CDEPTH=H(K)-RCDS(K,L)
                   QQ(K,L)=SQRT(GRAV)*CDEPTH**1.5

                 ENDIF


 6666          CONTINUE

 7777        CONTINUE

C     Find the maximum overtopping rate, QMAX, and volume of overtopped water,
C     VSTORM, at each toe depth during each storm

             DO 5555 K=1,NTOE

               DO 4444 L=1,2

                 IF(QQ(K,L).GT.QMAX(LSTORM,K,L)) THEN
                   QMAX(LSTORM,K,L)=QQ(K,L)
                 ENDIF

                 VSTORM(LSTORM,K,L)=QQ(K,L)*TSTEP+VSTORM(LSTORM,K,L)

 4444          CONTINUE

 5555        CONTINUE

 8888      CONTINUE

C ... END OF NTIMEL TIME LEVELS FOR EACH STORM ..............................


 9999 CONTINUE

C ---END OF NSTORM STORMS ---------------------------------------------------

C     Rank NSTORM values of QMAX and VSTORM using Subr. 10, SHELL
```

```
      DO 600 L=1,2
         DO 610 K=1,NTOE
            DO 620 LSTORM=1,NSTORM
               ARRAY(LSTORM)=QMAX(LSTORM,K,L)
 620        CONTINUE
            CALL SHELL(NSTORM,ARRAY)
            DO 630 LSTORM=1,NSTORM
               QMAX(LSTORM,K,L)=ARRAY(LSTORM)
 630        CONTINUE
            DO 640 LSTORM=1,NSTORM
               ARRAY(LSTORM)=VSTORM(LSTORM,K,L)
 640        CONTINUE
            CALL SHELL(NSTORM,ARRAY)
            DO 650 LSTORM=1,NSTORM
               VSTORM(LSTORM,K,L)=ARRAY(LSTORM)
 650        CONTINUE
 610     CONTINUE
 600  CONTINUE

C     Output the ranked values with the corresponding recurrence interval
C     REYEAR in File OUTOVERTOP for subsequent plotting

      WRITE(7,2301)
 2301 FORMAT(/'RANKED VALUES FOR ALL STORMS'/)

      WRITE(7,2302) N500
      DO 700 K=1,NTOE
         WRITE(7,2303) D(K)
         WRITE(7,2304) 'RANK','REC.INTERVAL','HIGH CREST (L=1)',
     +                 'LOW CREST (L=2)'
         WRITE(7,2305) 'N','REYEAR','QMAX(m2/s)','VSTORM(m2)',
     +                 'QMAX(m2/s)','VSTORM(m2)'
         DO 800 N=1,NSTORM
            REYEAR=FLOAT(NYEAR)/FLOAT(N)
            WRITE(7,2300) N, REYEAR, (QMAX(N,K,L),VSTORM(N,K,L),L=1,2)

 800     CONTINUE

 700  CONTINUE
```

```
 2302 FORMAT('N500=',I3)
 2303 FORMAT(/'TOE DEPTH=',F3.1,' m'/)
 2304 FORMAT(1X,A4,1X,A14,4X,A16,12X,A15)
 2305 FORMAT(1X,A3,5X,A6,7X,A10,2X,A10,8X,A10,2X,A10)
 2300 FORMAT(1X,I3,3X,F8.2,6X,F10.6,2X,F12.6,2X,4X,F10.6,2X,F12.6)


      END


C--00-------------------- END OF MAIN PROGRAM ------------------------------

C##01###################### SUBROUTINE OPENER ################################
C
C     This subroutine opens all input and output files
C
C---------------------------------------------------------------------------

      SUBROUTINE OPENER(N500)

      CHARACTER*10 FSAVEC(10),FSAVED(10)

      DATA FSAVEC/
     1'CSHORE2C01','CSHORE2C02','CSHORE2C03','CSHORE2C04',
     2'CSHORE2C05','CSHORE2C06','CSHORE2C07','CSHORE2C08',
     3'CSHORE2C09','CSHORE2C10'/
      DATA FSAVED/
     1'CSHORE2D01','CSHORE2D02','CSHORE2D03','CSHORE2D04',
     2'CSHORE2D05','CSHORE2D06','CSHORE2D07','CSHORE2D08',
     3'CSHORE2D09','CSHORE2D10'/


C     Open Output File OVERTOPDOC for concise documentation

      OPEN (UNIT=21,FILE='OVERTOPDOC',STATUS='NEW',ACCESS='SEQUENTIAL')

C     For N500=1,2,...,10, open four input files and three output file

      OPEN(UNIT=1,FILE='CYCLONEDATA',STATUS='OLD',ACCESS='SEQUENTIAL')
      OPEN(UNIT=2,FILE='CSHORE2DATA',STATUS='OLD',ACCESS='SEQUENTIAL')
      OPEN(UNIT=3,FILE='OVERTOPDATA1',STATUS='NEW',ACCESS='SEQUENTIAL')
```

```
      OPEN(UNIT=4,FILE='OVERTOPDATA2',STATUS='NEW',ACCESS='SEQUENTIAL')


      IF(N500.GE.1.AND.N500.LE.10) THEN
         OPEN(UNIT=15,FILE=FSAVEC(N500),STATUS='OLD',ACCESS=
     +'SEQUENTIAL')
         OPEN(UNIT=16,FILE=FSAVED(N500),STATUS='OLD',ACCESS=
     +'SEQUENTIAL')
         OPEN(UNIT=7,FILE='OUTOVERTOP',STATUS='NEW',ACCESS='SEQUENTIAL')

      ENDIF

      RETURN
      END


C--01------------------- END OF SUBROUTINE OPENER -------------------------
C##02####################### SUBROUTINE INPUTB ##############################
C
C     This subroutine reads data from input data files
C
C-------------------------------------------------------------------------

      SUBROUTINE INPUTB

      DIMENSION N500V(10),NSTORMV(10),REYEARV(10)
      DIMENSION STSETMAD(3),HMOMAXAD(3),STSETMAV(3),HMOMAXAV(3)
      DIMENSION SMAXSTV(10),SMAXWHV(10),SMAXWTV(10)
      DIMENSION STSETMV(3,10),HMOMAXV(3,10)

      COMMON /DESIGN100/ DS100(3), HS100(3), TP100(3), GAMMA100(3)
      COMMON /TOEDEP/    NTOE, TOED(3), D(3)


C ... COMPUTATIONAL INPUT DATA FROM CYCLONE AND CSHORE2 ......................

C     Read values from file CYCLONEDATA and write them in file OVERTOPDATA1


      DO 110 I=1,10
```

```
         READ (1,1110) N500V(I), NSTORMV(I), REYEARV(I),
     +                 SMAXSTV(I), SMAXWHV(I), SMAXWTV(I)

         WRITE(3,1110) N500V(I), NSTORMV(I), REYEARV(I),
     +                 SMAXSTV(I), SMAXWHV(I), SMAXWTV(I)

 110  CONTINUE

 1110 FORMAT(I3,I5,F8.2,3F7.2)


C     Read values from file CSHORE2DATA and write them in file OVERTOPDATA2


      DO 220 I=1,10
         READ (2,2220) N500V(I), REYEARV(I),(STSETMV(K,I),
     +                 HMOMAXV(K,I),K=1,NTOE)

         WRITE(4,2220) N500V(I), REYEARV(I),(STSETMV(K,I),
     +                 HMOMAXV(K,I),K=1,NTOE)

 220  CONTINUE

 2220 FORMAT(I3,F8.2,6F7.2)


C ... Water depth below mean sea level (MSL) .................................

      DO 330 K=1,NTOE
         D(K)=TOED(K)
 330  CONTINUE

C ... Average value of Offshore Maximum Storm Tide, Wave Height and Period for
C     10 500-year simulations


      SMAXSTAD=0.0
      SMAXWHAD=0.0
      SMAXWTAD=0.0
```

```
      DO 440 I=1,10

         SMAXSTAD=SMAXSTAD+SMAXSTV(I)
         SMAXWHAD=SMAXWHAD+SMAXWHV(I)
         SMAXWTAD=SMAXWTAD+SMAXWTV(I)

 440  CONTINUE

      SMAXSTAV=SMAXSTAD/10.0
      SMAXWHAV=SMAXWHAD/10.0
      SMAXWTAV=SMAXWTAD/10.0

C     Write averaged values in file OVERTOPDATA1

      WRITE(3,3330) 'AVERAGE', SMAXSTAV, SMAXWHAV, SMAXWTAV

 3330 FORMAT(A7,9X,3F7.2)

C ... Average value of onshore storm setup and significant wave height for
C     the different toe depths of the hypothetical structures


      DO 550 K=1,NTOE
         STSETMAD(K)=0.0
         HMOMAXAD(K)=0.0
         DO 660 J=1,10
            STSETMAD(K)=STSETMAD(K)+STSETMV(K,J)
            HMOMAXAD(K)=HMOMAXAD(K)+HMOMAXV(K,J)
 660     CONTINUE
         STSETMAV(K)=STSETMAD(K)/10.0
         HMOMAXAV(K)=HMOMAXAD(K)/10.0

 550  CONTINUE

C     Write averaged values in file OVERTOPDATA2

      WRITE(4,4440) 'AVERAGE', (STSETMAV(K),HMOMAXAV(K), K=1,NTOE)
 4440 FORMAT(A7,4X,6F7.2)


C ... DESIGN WATER DEPTH, SIGNIFICANT WAVE HEIGHT AND PERIOD FOR 100-YEAR .....
```

```
C     RETURN PERIOD

      DO 770 K=1,NTOE
         DS100(K)=D(K)+SMAXSTAV
         HS100(K)=HMOMAXAV(K)
         TP100(K)=SMAXWTAV
 770  CONTINUE


C ... Write design 100-year storm conditions in output file OVERTOPDOC ........

      K=1
      WRITE(21,*) 'DESIGN 100-YEAR STORM CONDITIONS'
      WRITE(21,5550) 'K=',K, K+1, K+2
      WRITE(21,6660) 'D=',(D(K),K=1,NTOE)
      WRITE(21,7770) 'DS(m)=',(DS100(K),K=1,NTOE)
      WRITE(21,8880) 'HS=HMO(m)=',(HS100(K),K=1,NTOE)
      WRITE(21,9990) 'TP(s)=',(TP100(K),K=1,NTOE)


 5550 FORMAT(1X,A2,10X,3I7)
 6660 FORMAT(1X,A2,10X,3F7.2)
 7770 FORMAT(1X,A6,6X,3F7.2)
 8880 FORMAT(1X,A10,2X,3F7.2)
 9990 FORMAT(1X,A6,6X,3F7.2)


      RETURN
      END


C--02-------------------- END OF SUBROUTINE INPUTB -------------------------

C##03######################## SUBROUTINE PARAMETER #########################
C
C     This subroutine calculates parameters used in other subroutines
C
C---------------------------------------------------------------------------

      SUBROUTINE PARAMETER
```

```
      COMMON /DESIGN100/ DS100(3), HS100(3), TP100(3), GAMMA100(3)
      COMMON /TOEDEP/    NTOE, TOED(3), D(3)
      COMMON /CONSTA/    GRAV, PI, TPIG, GAMMAB, GAMMABE, TALPHA
      COMMON /PARAM100/  SOP100(3), TSIOP100(3)

C ... CONSTANTS and PARAMETERS ...............................................
C
C     PI      = 3.14159
C     GAMMAB  = Reduction Factor for influence of a Berm
C               (No reduction due to a Berm)
C     GAMMABE = Reduction Factor for influence of Angle of Wave Attack
C               (Normally incident waves)
C     GRAV    = acceleration due to gravity = 9.81 m2/sec
C     TPIG    = 2.0*PI/GRAV

      PI = 4.0*ATAN(1.0)
      GAMMAB = 1.0
      GAMMABE = 1.0
      GRAV = 9.81
      TPIG = 2.0*PI/GRAV


C ... CALCULATION OF WAVE STEEPNESS AND SURF SIMILARITY PARAMETER FOR 100-YR
C     DESIGN CONDITIONS

C ... Wave Steepnes based on deepwater wavelength ............................

      DO 331 K=1,NTOE

         SOP100(K)=TPIG*HS100(K)/(TP100(K)*TP100(K))

C ... Surf similarity parameter ..............................................

         TSIOP100(K)=TALPHA/SQRT(SOP100(K))

 331  CONTINUE

      WRITE(21,1000) 'Sop=',(SOP100(K),K=1,NTOE)
      WRITE(21,1001) 'XIop=',(TSIOP100(K),K=1,NTOE)

 1000 FORMAT(1X,A4,8X,3F7.4)
```

```
 1001 FORMAT(1X,A5,7X,3F7.2)


      RETURN
      END


C--03-------------------- END OF SUBROUTINE PARAM --------------------------
C##04########################## SUBROUTINE REDFACT1 ###########################
C
C     This subroutine calculates the combined reduction factor for design
C     conditions
C
C----------------------------------------------------------------------------

      SUBROUTINE REDFACT1

      COMMON /DESIGN100/ DS100(3), HS100(3), TP100(3), GAMMA100(3)
      COMMON /TOEDEP/    NTOE, TOED(3), D(3)
      COMMON /CONSTA/    GRAV, PI, TPIG, GAMMAB, GAMMABE, TALPHA
      COMMON /PARAM100/  SOP100(3), TSIOP100(3)

C ... Computation of reduction factor, GAMMAH, for shallow foreshore

      DO 4441 K=1,NTOE

         DSHS=DS100(K)/HS100(K)
         IF(DSHS.GE.4.0) THEN
            GAMMAH=1.0
         ELSE
            GAMMAH=1.0-0.03*(4.0-DSHS)**2.0
         ENDIF

C ... Computation of reduction factor, GAMMAF, for rough slope
C     Since no equation is given by Van der Meer, tentatively assume:

         IF(TSIOP100(K).LE.3.5) THEN
            GAMMAF=0.55
         ELSE
            GAMMAF=TSIOP100(K)/(TSIOP100(K)+2.9)
         ENDIF
```

```
C ... Computation of the combined reduction factor GAMMA ......................
C     Minimum value suggested by Van der Meer & Janssen is GAMMA=0.5

        GAMMA100(K)=GAMMAB*GAMMAH*GAMMAF*GAMMABE

        IF(GAMMA100(K).LT.0.5) THEN
           GAMMA100(K)=0.5
        ENDIF


 4441 CONTINUE

      WRITE(21,1000) 'GAMMA=',(GAMMA100(K),K=1,NTOE)
 1000 FORMAT(1X,A6,6X,3F7.3)


      RETURN
      END


C--04-------------------- END OF SUBROUTINE REDFACT1 -----------------------
C##05######################### SUBROUTINE CREST1 ##############################
C
C     This subroutine calculates the crest height above still water level
C     for the specified overtopping rates Q under design conditions
C
C----------------------------------------------------------------------------

      SUBROUTINE CREST1

      DIMENSION RC(3,2)

      COMMON /DESIGN100/ DS100(3), HS100(3), TP100(3), GAMMA100(3)
      COMMON /PARAM100/  SOP100(3), TSIOP100(3)
      COMMON /TOEDEP/ NTOE, TOED(3), D(3)
      COMMON /CONSTA/ GRAV, PI, TPIG, GAMMAB, GAMMABE, TALPHA
      COMMON /FACT/   RCDS(3,2)
      COMMON /RATES/  Q(2), QQ(3,2)

      DO 5551 L=1,2
```

```
      DO 5552 K=1,NTOE

         SOPTA=SOP100(K)/TALPHA
         IF(TSIOP100(K).LE.2) THEN
            QB=Q(L)*SQRT(SOPTA)/SQRT(GRAV*HS100(K)**3.0)
            RB=-ALOG(QB/0.06)/5.2
            IF(RB.LT.0.3.OR.RB.GT.2.0) THEN
               WRITE(*,5511) RB
               WRITE(21,5511) RB
               STOP
            ENDIF
            RC(K,L)=RB*HS100(K)*TALPHA*GAMMA100(K)/
     +                SQRT(SOP100(K))

         ELSE
            QN=Q(L)/SQRT(GRAV*HS100(K)**3.0)
            RN=-ALOG(QN/0.2)/2.6
            IF(RN.LT.0.5.OR.RN.GT.4.0) THEN
               WRITE(*,5511) RN
               WRITE(21,5511) RN
               STOP
            ENDIF
            RC(K,L)=RN*HS100(K)*GAMMA100(K)
         ENDIF

         WRITE(*,*) RC(K,L)

         RCDS(K,L)=RC(K,L)+DS100(K)

5552    CONTINUE

5551 CONTINUE


    WRITE(21,5522) 'high RC(m)=',(RC(K,1),K=1,NTOE)
    WRITE(21,5522) 'low RC(m)=',(RC(K,2),K=1,NTOE)
    WRITE(21,5522) 'high RC+DS=',(RCDS(K,1),K=1,NTOE)
    WRITE(21,5522) 'low RC+DS=',(RCDS(K,2),K=1,NTOE)
    WRITE(21,5523) Q(1),Q(2)
```

```
 5511 FORMAT(/'ERROR:'/
     +        'RB or RN=',F7.3/
     +        'OUTSIDE RANGE 0.3-2.0 or 0.5-4.0')

 5522 FORMAT(1X,A11,1X,3F7.2)

 5523 FORMAT(/'where'/
     +'high crest elevation corresponds to overtopping rate'/
     +'Q=',F7.4,' m2/s'/
     +'low crest elevation corresponds to overtopping rate'/
     +'Q=',F7.4,' m2/s')

      RETURN
      END


C--05-------------------- END OF SUBROUTINE CREST1 -------------------------

C##06######################### SUBROUTINE REDFACT2 ##########################
C
C     This subroutine calculates the combined reduction factor
C
C---------------------------------------------------------------------------

      SUBROUTINE REDFACT2(K,CGAMMA)

      COMMON /DESIGN/    DS(3), HS(3), TP
      COMMON /CONSTA/    GRAV, PI, TPIG, GAMMAB, GAMMABE, TALPHA
      COMMON /PARAM/     SOP(3), TSIOP(3)

C ... Computation of reduction factor, GAMMAH, for shallow foreshore

      DSHS=DS(K)/HS(K)
      IF(DSHS.GE.4.0) THEN
         GAMMAH=1.0
      ELSE
         GAMMAH=1.0-0.03*(4.0-DSHS)**2.0
      ENDIF

C ... Computation of reduction factor, GAMMAF, for rough slope
```

```
C     Since no equation is given by Van der Meer, tentatively assume:

      IF(TSIOP(K).LE.3.5) THEN
         GAMMAF=0.55
      ELSE
         GAMMAF=TSIOP(K)/(TSIOP(K)+2.9)
      ENDIF

C ... Computation of the combined reduction factor GAMMA ......................
C     Minimum value suggested by Van der Meer & Janssen is GAMMA=0.5

      GAMMA=GAMMAB*GAMMAH*GAMMAF*GAMMABE

      IF(GAMMA.LT.0.5) THEN
         GAMMA=0.5
      ENDIF

      CGAMMA=GAMMA

      RETURN
      END

C--06-------------------- END OF SUBROUTINE REDFACT2 -----------------------

C##07####################### SUBROUTINE SHELL ###############################

C---------------------------------------------------------------------------
C     THIS SUBROUTINE SORTS AN ARRAY, ARR, OF LENGTH N INTO DESCENDING
C     NUMERICAL ORDER, BY THE SHELL-MEZGAR ALGORITHM. N IS INPUT; ARR IS
C     REPLACED ON OUTPUT BY ITS SORTED REARRANGEMENT
C---------------------------------------------------------------------------

      SUBROUTINE SHELL(N,ARR)

      PARAMETER(ALN2I=1./0.69314718, TINY=1.E-5)
      DIMENSION ARR(N)
      LOGNB2=INT(ALOG(FLOAT(N))*ALN2I+TINY)
      M=N
      DO 12 NN=1,LOGNB2
         M=M/2
```

```
         K=N-M
         DO 11 J=1,K
            I=J
 3          CONTINUE
            L=I+M
            IF(ARR(L).GT.ARR(I)) THEN
               T=ARR(I)
               ARR(I)=ARR(L)
               ARR(L)=T
               I=I-M
               IF(I.GE.1) GO TO 3
            ENDIF

 11      CONTINUE
 12   CONTINUE

      RETURN
      END

C--07-------------------- END OF SUBROUTINE SHELL ---------------------------

C************************** END OF SUBROUTINES *****************************
```

Appendix D

COMPUTER PROGRAM DAMAGE

```
C                                                                              *
C    #######      ######     ###     ###     ######      ######     #######    *
C    ##    ##    ##    ##    ####   ####    ##    ##    ##    ##    ##         *
C    ##    ##    ##    ##    ## ## ## ##    ##    ##    ##          ##         *
C    ##    ##    ########    ##  ###  ##    ########    ##  ####    ######     *
C    ##    ##    ##    ##    ##       ##    ##    ##    ##    ##    ##         *
C    ##    ##    ##    ##    ##       ##    ##    ##    ##    ##    ##         *
C    #######     ##    ##    ##       ##    ##    ##     ######     #######    *
C                                                                              *
C                                                                              *
C                                                                              *
C                                                                              *
C                                                                              *
C                                                                              *
C                                                                              *
C*******************************************************************************
C*                                                                             *
C*                   BEATRIZ POZUETA AND NOBUHISA KOBAYASHI                    *
C*                                                                             *
C*                    CENTER FOR APPLIED COASTAL RESEARCH                      *
C*                                                                             *
C*                                                                             *
C*               University of Delaware, Newark, Delaware 19716                *
C*                                                                             *
C*                             April 2002                                      *
C*                                                                             *
C*******************************************************************************
C*******************************************************************************
C*00********************  MAIN PROGRAM  *****************************************
C
C
C
C*******************************************************************************

      PROGRAM DAMAGE

      DIMENSION TOED(3), HTOE(3), HMOTOE(3)
      DIMENSION DELDN(3,2)
      DIMENSION S(3,2)
      DIMENSION SAS(500,3,2), SNMAX(500,3,2), DELNEQ(500,3,2)
```

```
C ... INPUT N500, NYEAR AND NSTORM ........................................

      WRITE(*,*)
     +'Specify Integer N500 for N500-th simulation of 500 years'
      READ(*,*) N500

C     N500 must be in the range of 1-10

      IF(N500.LT.1.OR.N500.GT.10) WRITE(*,*)
     +'N500 must be in the range 1-10'

      IF(N500.GE.1.AND.N500.LE.10) THEN
         NYEAR=500
         WRITE(*,*)
     +   'Specify Number of Storms for this 500-Year Simulation'
         READ(*,*) NSTORM
      ENDIF

C ... NTOE=(1,2 or 3)  toe depths, TOED (positive) of hypothetical coastal
C     structures where the computed time series of wave setup and height
C     for each of NSTORM storms are stored.

      WRITE(*,*)
     +'Specify number of toe depths of hypothetical coastal structures'
      READ(*,*) NTOE

      WRITE(*,*)
     +'Specify toe depths of hypothetical coastal structures'
      READ(*,*) (TOED(K),K=1,NTOE)


C ... Input on screen the (low) estimated product of nominal stone diameter
C     and (specific gravity -1) for different depths of hypothetical coastal
C     structures (in meters)...[Van der Meer formula].

      WRITE(*,*)
     +'Specify the (low) estimated value of the product: Delta*Dn50'
      READ(*,*) (DELDN(K,1), K=1,NTOE)
```

```
C ... Input on screen the (high) estimated product of nominal stone diameter
C     and (specific gravity -1) for different depths of hypothetical coastal
C     structures (in meters)...[Hudson formula]

      WRITE(*,*)
     +'Specify the (high) estimated value of the product: Delta*Dn50'
      READ(*,*) (DELDN(K,2), K=1,NTOE)


C     Time step of 0.5 hr = 1800 s was used for computation of storm surge and
C     waves

      TSTEP=1800.0

C     For an empirical permeability coefficient P=0.4 (conventional multi-
C     layered rubble mound breakwaters)and cot(alpha)=2, the surf similarity
C     parameter is 3.77 at the minimum armor stability.
C     The critical stability number for this value of the surf similarity
C     parameter is:

      SNC=1.08

C     Empirical parameters considered in the prediction of the damage
C     progression (calibrated by Melby and Kobayashi, 1999-2000)

      A=0.011
      B=0.25

      B1=1.0/B


C     Subr. 1 OPENER opens input and output files.

      CALL OPENER(N500)


C ... COMPUTATION OF DAMAGE PROGRESSION FOR EACH 500-YEAR SIMULATION, THREE
C     DIFFERENT TOE DEPTHS AND HIGH AND LOW ESTIMATES OF THE PRODUCT DELTA*Dn50


C     Initialization of damage S, which accumulates the damage at each time
```

```
C     step of each storm.


      DO 1 K=1,NTOE
         DO 2 L=1,2
            S(K,L)=0.0
 2       CONTINUE
 1    CONTINUE



C --- COMPUTATIONS FOR NSTORM STORMS ----------------------------------------

      DO 9999 LSTORM=1,NSTORM

         WRITE(*,2111) LSTORM

 2111 FORMAT(/'LSTORM=',I3,'-TH STORM')


C     Read from File CSHORE2D** with **=N500=1-10
C     LL = Storm Number
C     NTIMEL = Number of time levels for each storm

         READ(2,2200) LL,NTIMEL

C     If LL is not equal to LSTORM, stop computation because a wrong file has
C     been read.

         IF(LL.NE.LSTORM) THEN
            WRITE(*,2800) LL,LSTORM
            STOP
         ENDIF

 2200 FORMAT(2I3)
 2800 FORMAT(/'ERROR:'/
     +        'Storm number from File CSHORE2D**; LL=',I3/
     +        'NOT SAME as LSTORM=',I3,' in DO LOOP')


C     The following variables are zero before they are computed for each storm
```

```
C      SAS    = Damage S(n) after each storm
C      SNMAX  = Maximum Nmo for each storm, where
C               Nmo = Stability number based on the spectral significant wave
C                     height Hmo
C      DELNEQ = Equivalent number of waves for each storm

       DO 9991 L=1,2
          DO 9992 K=1,NTOE
             SAS(LSTORM,K,L)=0.0
             SNMAX(LSTORM,K,L)=0.0
             DELNEQ(LSTORM,K,L)=0.0
 9992     CONTINUE
 9991 CONTINUE


C ... COMPUTATION FOR NTIMEL TIME LEVELS FOR EACH STORM ......................

          DO 8888 LTIMEL=1,NTIMEL

C      Read from File CSHORE2D** with **=N500
C      TIME      = time in hours during each storm
C      TP        = spectral peak period (in seconds)
C      HTOE(K)   = total water depth at toe node JTOE(k)
C      HMOTOE(K) = HMO wave height at toe node JTOE(k)
C                  with k=1,...,NTOE

             READ(2,2100) TIME,TP, (HTOE(K),HMOTOE(K),K=1,NTOE)

 2100 FORMAT(8F7.2)


C      Assume the following relations between the spectral peak period (Tp) and
C      the significant wave period (Ts), and between the significant wave period
C      (Ts) and the mean wave period (Tm):
C
C      Tp = 1.05*Ts;   Ts = 1.2*Tm
C      TM = Tm = mean wave period during the interval t(n) < t < t(n+1)

             TM=TP/1.26

C      DELN = Number of waves during the interval t(n) < t < t(n+1)
```

```
      DELN=TSTEP/TM

C ... COMPUTATION FOR THREE DIFFERENT TOE DEPTHS FOR EACH TIME LEVEL ..........

      DO 7777 K=1,NTOE

C ... COMPUTATION FOR HIGH AND LOW ESTIMATES OF THE PRODUCT DELTA*Dn50 ........

        DO 6666 L=1,2

C     SNMO = Nmo = Stability number based on the spectral significant wave
C                  height Hmo during the interval t(n) < t < t(n+1)

          SNMO=HMOTOE(K)/DELDN(K,L)

C     Find damage S(n+1) at the time step t(n+1), based on the known damage
C     S(n) at time step t(n).
C     Empirical Formula by Melby and Kobayashi (1999,2000)

          IF(SNMO.LT.SNC) THEN
C           No additional damage
            S(K,L)=S(K,L)
          ELSE

C     EN = Equivalent number of waves based on Nmo during t(n) < t < t(n+1)
C          to cause the same damage S(n)

            ANMO5=A*SNMO**5
            EN=(S(K,L)/ANMO5)**B1
            S(K,L)=ANMO5*(EN+DELN)**B
          ENDIF

C     Find maximum value of SNMO during each storm

          DUM=SNMAX(LSTORM,K,L)
```

```
               IF(DUM.LT.SNMO) SNMAX(LSTORM,K,L)=SNMO

C     Find the damage SAS(LSTORM,K,L) at the end of LSTORM-th storm and the
C     equivalent number of waves, DELNEQ(LSTORM,K,L) based on the maximum
C     value of Nmo, SNMAX(LSTORM,K,L) for the same  damage at the end of
C     this storm.

               IF(LTIMEL.EQ.NTIMEL) THEN
                  SAS(LSTORM,K,L)=S(K,L)
                  DUM=SNMAX(LSTORM,K,L)
                  IF(DUM.LE.SNC) THEN
                     DELNEQ(LSTORM,K,L)=0.0
                  ELSE
                     ANM5=A*DUM**5
                     IF(LSTORM.EQ.1) THEN
                        DELNEQ(LSTORM,K,L)=(SAS(LSTORM,K,L)/ANM5)**B1
                     ELSE
                        SINC=SAS(LSTORM,K,L)-SAS(LSTORM-1,K,L)
                        IF(SINC.GT.0.0001) THEN
                           DELNEQ(LSTORM,K,L)=(SAS(LSTORM,K,L)/ANM5)**B1-
     +                                   (SAS(LSTORM-1,K,L)/ANM5)**B1
                        ELSE
                           DELNEQ(LSTORM,K,L)=0.0
                        ENDIF
                     ENDIF
                  ENDIF
               ENDIF

 6666          CONTINUE

 7777       CONTINUE


 8888    CONTINUE


C ... END OF NTIMEL TIME LEVELS FOR EACH STORM ..............................


 9999 CONTINUE
```

```
C ---END OF NSTORM STORMS -----------------------------------------------

C     Output the results in File OUTDAMAGE

      WRITE(3,2300) N500

 2300 FORMAT('N500=',I3)


      DO 4444 K=1,NTOE

         WRITE(3,2301) TOED(K)
         WRITE(3,2306) (DELDN(K,L),L=1,2)
         WRITE(3,2302) 'LOW Dn50','HIGH Dn50'
         WRITE(3,2304) 'STORM','Year','max.Nmo','DAMAGE','No.WAVES',
     +'max.Nmo','DAMAGE','No.WAVES'

         DO 5555 N=1,NSTORM

C     To plot the computed results over NYEAR=500 years for all 500-year
C     simulations, the storm number N is converted to YEAR as follows:

            YEAR=N*500.0/NSTORM

            WRITE(3,2305) N,YEAR,(SNMAX(N,K,L),SAS(N,K,L),DELNEQ(N,K,L),
     +                    L=1,2)

 5555    CONTINUE

 4444 CONTINUE

 2301 FORMAT(/'TOE DEPTH=',F3.1,' m'/)
 2302 FORMAT(27X,A8,25X,A9)
 2304 FORMAT(1X,A5,2X,A4,2(3X,A7,3X,A6,6X,A8))
 2305 FORMAT(2X,I3,1X,F6.2,2(1X,F9.3,1X,F8.5,1X,F13.3))
 2306 FORMAT(/'Low estimate of the product Delta*Dn50  =',F5.3,' m'/
     +       'High estimate of the product Delta*Dn50 =',F5.3,' m'/)

      END
```

```
C--00-------------------- END OF MAIN PROGRAM ------------------------------

C##01####################### SUBROUTINE OPENER ################################
C
C     This subroutine opens all input and output files
C
C---------------------------------------------------------------------------

      SUBROUTINE OPENER(N500)

      CHARACTER*10 FSAVED(10)

      DATA FSAVED/
     1'CSHORE2D01','CSHORE2D02','CSHORE2D03','CSHORE2D04',
     2'CSHORE2D05','CSHORE2D06','CSHORE2D07','CSHORE2D08',
     3'CSHORE2D09','CSHORE2D10'/


C     For N500=1,2,...,10, open one input file and one output file

      IF(N500.GE.1.AND.N500.LE.10) THEN
         OPEN(UNIT=2,FILE=FSAVED(N500),STATUS='OLD',ACCESS=
     +'SEQUENTIAL')
         OPEN(UNIT=3,FILE='OUTDAMAGE',STATUS='NEW',ACCESS='SEQUENTIAL')
      ENDIF

      RETURN
      END

C--01------------------- END OF SUBROUTINE OPENER -------------------------

C************************** END OF SUBROUTINES ******************************
```