NUMERICAL MODELING OF WAVE TRANSFORMATION, BREAKING AND RUNUP ON DIKES AND GENTLE SLOPES

BY

JILL PIETROPAOLO, NOBUHISA KOBAYASHI AND JEFFREY A. MELBY

RESEARCH REPORT NO. CACR-11-05 AUGUST 2011



CENTER FOR APPLIED COASTAL RESEARCH

Ocean Engineering Laboratory University of Delaware Newark, Delaware 19716

ACKNOWLEDGEMENTS

This study was supported by the U.S. Army Corps of Engineers, Coastal and Hydraulics Laboratory under Contract Nos. W912HZ-10-P-0234 and W912HZ-11-P-0173. The writers would like to thank Marcel van Gent for providing his reports.

TABLE OF CONTENTS	
LIST OF FIGURES	iii
LIST OF TABLE	X
ABSTRACT	X1
Chapter	
1 INTRODUCTION	1
2 NUMERICAL MODEL AND CALIBRATION	
2.1 Governing Equations	
2.1.1 Wave overtopping	6
2.1.2 Wave runup	
1	
2.2 Input Parameters	
	10
3 WAVE RUNUP ON DIKE WITH BARKED BEACH	
3.1 Physical Model	12
3 2 Wave Conditions	
3.3 Comparisons	
-	
3.3.1 Significant wave height	
3.3.2 Wave runup	
A WAVE DUNUD ON DIVES WITH SLODING DEACHES	22
4 WAVE RUNUL ON DIKES WITH SLOTING BEACHES	
4.1 Physical Models	
4.2 Wave Conditions	
4.3 Comparisons	
4.3.1 Significant wave height	
4.3.2 wave runup	
5 MINOR WAVE OVERTOPPING	
5.1 Experimental Setup	
5.2 Threshold of Wave Overtopping	
5.3 Wave Overtopping Rate	
6 WAVE RUNUP ON GENTLE SLOPES	27
UWAVE KUNUL ON GENILE SLOFES	
6.1. Experimental Setup and Wave Conditions	
6.2 Wave Runup	

TABLE OF CONTENTS

6.3 Wave Runup on Natural Beaches	. 40
7 CONCLUSIONS REFERENCES	. 42 . 44
Appendix	
A. CROSS-SHORE VARIATIONS OF WAVE SETUP AND HEIGHTS FOR 40 TESTS IN SERIES P	. 47
B. CROSS-SHORE VARIATIONS OF WAVE SETUP AND HEIGHTS FOR 42 TESTS IN SERIES A	. 68
C. CROSS-SHORE VARIATIONS OF WAVE SETUP AND HEIGHTS FOR 31 TESTS IN SERIES B	. 90
D. CROSS-SHORE VARIATIONS OF WAVE SETUP AND HEIGHTS FOR 24 TESTS IN SERIES C	107

LIST OF FIGURES

2.1 Three intersection points along runup wire placed at height δ_r above impermeable bottom.	9
3.1 Experimental setup for series P	. 13
3.2 Wave period ratios as a function of $T_{m-I,0}$ at $x = 0$ for series P	. 16
3.3 Cross-shore variation of wave setup, wet probability P_w and wave height H_{mo} for test P 27	. 18
3.4 Measured and computed wave heights H_{mo} at $x = 160, 335, 505, and 570$ (toe) m	. 19
3.5 Measured and computed 2% runup heights $R_{2\%}$ for series P	. 21
3.6 Measured and computed 1% runup heights $R_{1\%}$ for series P	. 22
4.1 Experimental setup for series A (top), B (middle), and C (bottom)	. 25
4.2 Wave period ratios as a function of $T_{m-1,0}$ at $x = 0$ for Series A, B, and C	. 27
4.3 Measured and computed wave heights at $x = 10, 20, \text{ and } 30$ (toe) m for series A (top), B	
(middle), and C (bottom)	. 28
4.4 Measured and computed 2% runup heights $R_{2\%}$ for series A, B, and C	. 30
4.5 Measured and computed 1% runup heights $R_{1\%}$ for series A, B, and C	. 31
5.1 Measured wave overtopping rate q_o over crest height R_c above SWL as a function of ($R_{1\%}$)	-
<i>R_c</i>)	. 34
5.2 Measured and computed wave overtopping rates q_o for series A, B, and C	. 36
6.1 Measured and computed 2% runup heights $R_{2\%}$ for slopes A, B, C, and D	. 39
6.2 Measured and computed significant runup heights $R_{1/3}$ for slopes A, B, C, and D	. 40
A.1 Test P1	. 48
A.2 Test P2	. 48
A.3 Test P3	. 49

A.4	Гest P4	. 49
A.5	Гest P5	. 50
A.6	Гest P6	. 50
A.7	Гest P7	. 51
A.8	Гest P8	. 51
A.9	Гest P9	. 52
A.10	Test P10	. 52
A.11	Test P11	. 53
A.12	Test P12	. 53
A.13	Test P13	. 54
A.14	Test P14	. 54
A.15	Test P15	. 55
A.16	Test P16	. 55
A.17	Test P17	. 56
A.18	Test P18	. 56
A.19	Test P19	. 57
A.20	Test P20	. 57
A.21	Test P21	. 58
A.22	Test P22	. 58
A.23	Test P23	. 59
A.24	Test P24	. 59
A.25	Test P25	. 60
A.26	Test P26	. 60

A.27	Test P27	61
A.28	Test P28	61
A.29	Test P29	62
A.30	Test P30	62
A.31	Test P31	63
A.32	Test P32	63
A.33	Test P33	64
A.34	Test P34	64
A.35	Test P35	65
A.36	Test P36	65
A.37	Test P37	66
A.38	Test P38	66
A.39	Test P39	67
A.40	Test P40	67
B.1 7	Test A1	69
B.2 7	Test A2	69
B.3 7	Test A3	70
B.4 7	Sest A4	70
B.5 1	Sest A5	71
B.6 7	Sest A6	71
B.7]	Fest A7	72
B.8 7	Fest A8	72
B.9 7	Cest A9	73

B.10	Test A10	73
B.11	Test A11	74
B.12	Test A12	74
B.13	Test A13	75
B.14	Test A14	75
B.15	Test A15	76
B.16	Test A16	76
B.17	Test A17	77
B.18	Test A18	77
B.19	Test A19	78
B.20	Test A20	78
B.21	Test A21	79
B.22	Test A22	79
B.23	Test A23	80
B.24	Test A24	80
B.25	Test A25	81
B.26	Test A26	81
B.27	Test A27	82
B.28	Test A28	82
B.29	Test A29	83
B.30	Test A30	83
B.3 1	Test A31	84
B.32	Test A32	84

B.33 Test A33	85
B.34 Test A34	85
B.35 Test A35	86
B.36 Test A36	86
B.37 Test A37	87
B.38 Test A38	87
B.39 Test A39	88
B.40 Test A40	88
B.41 Test A41	89
B.42 . Test A42	89
C.1 Test B1	91
C.2 Test B2	91
C.3 Test B3	92
C.4 Test B4	92
C.5 Test B5	93
C.6 Test B6	93
C.7 Test B7	94
C.8 Test B8	94
C.9 Test B9	95
C.10 Test B10	95
C.11 Test B11	96
C.12 Test B12	96
C.13 Test B13	97

C.14 Test B14	
C.15 Test B15	
C.16 Test B16	
C.17 Test B17	
C.18 Test B18	
C.19 Test B19	
C.20 Test B20	
C.21 Test B21	
C.22 Test B22	
C.23 Test B23	
C.24 Test B24	
C.25 Test B25	
C.26 Test B26	
C.27 Test B27	
C.28 Test B28	
C.29 Test B29	
C.30 Test B30	
C.31 Test B31	
D.1 Test C1	
D.2 Test C2	
D.3 Test C3	
D.4 Test C4	
D.5 Test C5	

D.6	Test C6	110
D.7	Test C7	111
D.8	Test C8	111
D.9	Test C9	112
D.1 0) Test C10	112
D. 11	1 Test C11	113
D.12	2 Test C12	113
D.13	3 Test C13	114
D.14	F Test C14	114
D.15	5 Test C15	115
D.16	5 Test C16	115
D.17	7 Test C17	116
D.18	3 Test C18	116
D.19	• Test C19	117
D.2 0) Test C20	117
D.2 1	1 Test C21	118
D.22	2 Test C22	118
D.23	3 Test C23	119
D.2 4	F Test C24	119

LIST OF TABLE

3.1 Wave conditions at x= 0 for Series P	
4.1 Wave Conditions at $x = 0$ for Series A, B, and C	
5.1 Number of Tests with Overtopping Rates $q_o > 1$ ml/s/m	
6.1 Wave Conditions at $x = 0$ for Four Uniform Slopes	

ABSTRACT

The numerical cross-shore model CSHORE is extended to predict irregular wave runup on impermeable dikes. CSHORE is tested against 40 wave runup tests on an impermeable dike on a barred beach and 97 wave runup tests on an impermeable dike with a gently sloping beach. CSHORE is also tested against 97 wave overtopping tests. The spectral wave period and peak wave period from a seaward boundary located outside the surf zone are both used as the representative period for input to CSHORE. The difference between these two periods is compared. The significant wave height at the seaward boundary is also used as input. The significant wave height transformation from the seaward boundary to the location of the dike toe is compared for all 137 tests to show the capability and limitation of CSHORE. The measured 2% and 1% exceedence runup heights are predicted within errors of about 20%. CSHORE predicts the threshold of wave overtopping but the minor wave overtopping rates can be predicted only within a factor of 10.

The upper limit elevation of wave action along coastal regions has become increasingly important over the past decade, especially as the sea level to rises. Wave action during storms can cause beach and dune erosion. Areas of high risk for flooding need to be determined in order to create coastal flood risk maps such as those produced by the U.S. Federal Emergency Management Agency (FEMA). CSHORE is thus compared with 120 tests for wave runup on gentle uniform slopes and wave runup data on natural beaches in order to assess the utility of CSHORE for coastal flood risk mapping on sand beaches. CSHORE is a good practical choice because it can also be used to predict beach and dune profile evolution during a storm.

Chapter 1

INTRODUCTION

Wave runup, the upper landward limit of wave uprush above the still water level (Kobayashi 1999), is important to coastal engineers for several reasons. One particular importance is determining areas affected by wave action during extreme events in order to create coastal flood risk maps such as those produced by the U.S. Federal Emergency Management Agency (FEMA) (Crowell et al. 2010). In order to warn the people who live in the 100 year coastal flood zones, these coastal flood risk maps require the prediction of extreme wave runup exceeded by 2% or 1% of incident irregular waves denoted $R_{2\%}$ and $R_{1\%}$, respectively. Wave runup is also necessary to determine the crest height for which a coastal structure should be designed for in order to prevent wave overtopping of the structure. This is a major concern for structures such as levees and dikes whose primary function is sea defense (EurOtop Manuel 2007). The objective of this study is to develop a physically realistic and robust numerical model for better predicting the landward limit of wave action during an extreme storm for engineering applications such as coastal flood risk mapping.

A number of empirical formulas have been proposed for the prediction of extreme wave runup such as wave runup exceeded by 2% of incident irregular waves. The runup formulas for coastal structures require the input of the representative wave height and period at the toe of the structures. If the toe is located inside the surf zone, the representative wave height and period may be difficult to specify because infragravity waves may not be negligible inside the surf zone in comparison to wind (sea and swell) waves (e.g., van Gent 2001). On the other hand, the runup formulas for beaches without any toe employ the representative wave height and period measured offshore (e.g., Holman 1986). Wave transformation from the offshore point to the swash zone on a beach is neglected in these formulas, although wave setup and swash on the beach depends on the bathymetry of the entire surf zone (e.g., Raubenheimer et al. 2001). These formulas are simple and easy to use, however they are limited to specific data fitted to the formulas and may not be applicable to different beach bathymetries and structures (Kobayashi et al. 2008) because of the neglect of wave transformation.

Numerical models based on the depth-averaged one-dimensional nonlinear shallow-water wave equations have been developed to predict the time series of the shoreline elevation on coastal structures and beaches. Raubenheimer and Guza (1996) and Raubenheimer (2002) applied the numerical model by Kobayashi et al. (1989) to predict the free surface elevation and fluid velocities in the surf and swash zones on natural beaches. The numerical model was initialized with time series of the free surface elevation and cross-shore velocity observed in the mean water depth of 80 to 300 cm. The model was shown to predict both wind and infragravity wave motions. The seaward boundary of the shallow-water wave model must be located in shallow water. To initiate the computation farther offshore, use can be made of numerical models based on Boussinesq wave equations (e.g., Nwogu and Demirbilek 2010). These time-dependent models predict the time series of the hydrodynamic variables on the specified bathymetry and can be used to examine the wind and infragravity wave motions in detail. Although these models offer much more detail than the empirical formulas, they also come with their drawbacks. These models require significant computational time and are not easy to use for

2

coastal flood risk mapping along a long coastline. Furthermore, this mapping normally requires only the landward extent of flooding and wave action during specified storm conditions. The excess details produced by the models are often not necessary. The computational time and difficulty of use cause these models to be inefficient for practical use (Kobayashi et al. 2008).

Kobayashi et al. (2008) developed a time-averaged probabilistic model to predict irregular wave runup statistics instead of the time series of the shoreline elevation. The initial model limited to the wet zone only was extended by Kobayashi et al. (2010a) to the wet and dry zone above the still water shoreline. This cross-shore model CSHORE is efficient computationally and convenient for practical applications. In addition, CSHORE allows for arbitrary bottom profile and can also predict beach profile evolution if necessary. This becomes important when determining the damage progression during a storm, as the beach profile is eroded. CSHORE is more empirical than the time-dependent models, and must be shown to be reliable and applicable to a verity of conditions at different beaches.

In the following chapters, CSHORE is tested with several different data sets from different structures and beaches. CSHORE is calibrated to predict irregular wave runup on impermeable dikes with barred and sloping beaches, dikes with minor wave overtopping, and beaches with gentle slopes. Chapter 2 explains the equations and methods behind the numerical model CSHORE as well as the calibration of CSHORE. Chapter 3 discusses the comparison of the calibrated CSHORE to the 40 physical model tests by van Gent (1999a) of wave runup of an impermeable dike with a barred beach. This study makes use of van Gent's 1999 data because it appears to be the best available data set. Chapter 4 compares the calibrated CSHORE to 97 tests of impermeable dikes on a sloping beach by van Gent (1999b). In Chapter 5, the same 97 tests by van Gent (1999b) are used to examine the relation between the extreme wave runup and wave

3

overtopping rate. While wave runup depends on the height of a runup gauge above the impermeable slope, the wave overtopping rate in Chapter 5 is independent of the runup gauge height. In Chapter 6, the calibrated CSHORE is compared with 120 tests by Mase (1989) for irregular wave runup on gentle impermeable slopes. Stockdon et al. (2006) assembled wave runup data on natural beaches but it is found to be difficult to compare CSHORE with the field data in quantitative manners as will be explained in Chapter 6. Finally, in Chapter 7, the findings of this study are summarized. It is noted that the summary of this study is presented by Kobayashi, Pietropaolo, and Melby (2011).

Chapter 2

NUMERICAL MODEL AND CALIBRATION

The cross-shore model CSHORE, which was developed for various applications, has a number of options. This chapter describes the version of CSHORE used in the study. The first section of this chapter is separated into two subsections. The first subsection describes the governing equations of the numerical model used in the computation of wave overtopping and the second describes the equations used to calculate the extreme runup. The second section of this chapter describes the input parameters and calibration of the numerical model which will be used throughout the following chapters.

2.1 Governing Equations

In the following, the cross-shore coordinate *x* is positive onshore with x = 0 at the seaward boundary where the incident waves are specified. Incident irregular waves are assumed to propagate in the *x* direction. These assumptions are made in both the computations for wave runup and overtopping. In order to predict wave runup on a dike, the significant wave height at the toe of the dike is required. For the CSHORE computation, the significant wave height and representative period are specified at the seaward boundary. CSHORE predicts the significant wave height as it transforms from the input wave height at the seaward boundary x = 0 to the wave height at the toe of the dike. This is necessary for an improved runup and overtopping computation in comparison to empirical formulas based on wave conditions at the dike toe.

2.1.1 Wave overtopping

In order to predict irregular wave transformation and overtopping, CSHORE uses the time-averaged continuity, momentum, and energy equations expressed as

$$\frac{g\sigma_{\eta}^{2}}{C} + \bar{h}\overline{U} = q_{o} \tag{1}$$

$$\frac{dS_{xx}}{dx} = -\rho g \bar{h} \frac{d\eta}{dx} - \tau_b \tag{2}$$

$$\frac{dF}{dx} = -D_B - D_f \tag{3}$$

where g = gravitational acceleration; $\sigma_{\eta} = \text{standard deviation of the free surface elevation } \eta$ above the still water level (SWL); C = linear wave phase velocity; $\overline{h} = \text{mean of the water depth}$ h; $\overline{U} = \text{mean of the depth-averaged velocity } U$; $q_o = \text{wave overtopping rate}$; $S_{xx} = \text{cross-shore}$ radiation stress; $\rho = \text{fluid density}$; $\overline{\eta} = \text{mean free surface elevation}$; $\tau_b = \text{time-averaged bottom}$ shear stress; F = wave energy flux per unit width; and D_B and $D_f = \text{time-averaged wave energy}$ dissipation rate per unit horizontal area due to wave breaking and bottom friction, respectively. An equation for roller energy, which is used in the calculation of roller volume flux and its energy dissipation rate, is neglected in this study for simplicity. The computed wave overtopping is found to be insensitive to the roller effect. The equations for S_{xx} , τ_b , F, D_B and D_f are given by Kobayashi et al. (2010b). Eqs. (1) – (3) yield the cross-shore variations of \overline{U} , $\overline{\eta}$ and σ_{η} where the spectral significant wave height H_{mo} is given by $H_{mo} = 4 \sigma_{\eta}$. Eqs. (1) – (3) are limited to the wet zone where water is present always. The wet and dry zone is assumed to occur landward of the still water shoreline located at $x = x_{swl}$. The time-averaged continuity and momentum equations derived from the nonlinear shallow-water wave equations are expressed as

$$hU = q_o \tag{4}$$

$$\frac{d}{dx}\left(\overline{hU^{2}} + \frac{g}{2}\overline{h^{2}}\right) = -g\,\overline{h}\frac{dz_{b}}{dx} - \frac{1}{2}f_{b}\overline{|U|U}$$
(5)

where *h* and *U* = instantaneous water depth and depth-averaged velocity; z_b = elevation of the fixed bottom; and f_b = bottom friction factor. The overbar denotes averaging for the wet duration only because no water exists during the dry period. The probability density function *f*(*h*) for *h* is assumed to be exponential

$$f(h) = \frac{P_w^2}{\bar{h}} \exp\left(-P_w \frac{h}{\bar{h}}\right) \quad \text{for} \quad h > 0 \tag{6}$$

where P_w = wet probability of the water depth h > 0; \overline{h} = mean water depth for the wet duration. The mean depth for the entire duration is equal to $P_w \overline{h}$. The velocity U in Eqs. (4) and (5) is expressed as

$$U = 2\sqrt{gh} + U_s \tag{7}$$

Where U_s = steady velocity varying with *x* to account for offshore return flow on the upward slope above the still water shoreline. If $U_s = 0$, Eq. (7) produces the onshore flow with U > 0only. Eqs. (4) and (5) along with Eqs. (6) and (7) are solved to obtain the cross-shore variations of \overline{h} and P_w as explained by Kobayashi (2010b) for the impermeable wet and dry zone. The standard deviations of *h* and η are the same and given by

$$\frac{\sigma_{\eta}}{\overline{h}} = \left(\frac{2}{P_w} - 2 + P_w\right)^{0.5} \tag{8}$$

which is derived using Eq. (6).

The landward marching computation in the wet and dry zone starts at $x = x_{swl}$ where $P_w = 1$ and the mean depth \overline{h} is matched with that computed using Eqs. (1) – (3). In the wet zone, $P_w = 1$. The computation is continued until \overline{h} becomes less than 10⁻⁶ m or to the landward end of the computation domain. If the computation does not reach the landward end, the wave overtopping rate $q_o = 0$ is assumed. If the computation reaches the landward end located at $x = x_c$, q_o is computed using the computed values of $\overline{h} = \overline{h}_c$ and $P_w = P_c$ at $x = x_c$

$$q_o = \frac{3\sqrt{\pi}}{2} \overline{h}_c \left(\frac{g\overline{h}_c}{P_c}\right)^{0.5} \quad \text{at} \quad x = x_c \tag{9}$$

which is derived using Eqs. (4), (6), (7) and $U_s = 0$ at $x = x_c$.

2.1.2 Wave runup

The statistics of wave runup on the impermeable slope is predicted by modifying the method by Kobayashi et al. (2008) who analyzed wave runup on permeable slopes using CSHORE limited to the wet zone only. Their method is based on the runup measurement using a runup wire placed at the vertical height δ_r above the bottom whose elevation is denoted as z_b . The runup wire measures the instantaneous elevation η_r above SWL of the intersection between the wire and the free surface elevation. The mean $\overline{\eta}_r$ and standard deviation σ_r of the time-varying η_r are estimated using the three intersection points $(x_1, z_1), (x_2, z_2),$ and (x_3, z_3) along the wire as depicted in Fig. 2.1 where x_i and $z_i =$ onshore coordinate and elevation, respectively, at point *i* with i = 1, 2 and 3 and $z_1 > z_2 > z_3$. The mean water level during the entire duration of the

runup measurement is given by $(z_b + P_w \overline{h})$. The water levels corresponding to one standard deviation $(P_w \sigma_\eta)$ above and below the mean water level are given by $[z_b + P_w(\overline{h} + \sigma_\eta)]$ and $[z_b + P_w(\overline{h} - \sigma_\eta)]$, respectively. These three water levels are used to obtain the three

intersections. The mean $\overline{\eta}_r$ above SWL and standard deviation σ_r are estimated as

$$\overline{\eta}_r = \frac{z_1 + z_2 + z_3}{3} ; \ \sigma_r = \frac{z_1 - z_3}{2} ; \ S_r = \frac{z_1 - z_3}{x_1 - x_3}$$
(10)

where S_r = representative slope in the zone of the runup measurement. Eq. (10) is an extension of the earlier method by Kobayashi et al. (2008) limited to the wet zone only.



Figure 2.1 Three intersection points along runup wire placed at height δ_r above impermeable bottom.

The crest elevation of the time-varying elevation η_r is defined as the runup height Rabove SWL. The runup height above the mean water level is given by $\left(R - \overline{\eta}_r\right)$. The exceedence probability P for the runup height $\left(R - \overline{\eta}_r\right)$ is assumed to be given by the Rayleigh distribution (Kobayashi et al. 2008)

$$P = \exp\left[-2\left(\frac{R - \overline{\eta}_r}{R_{1/3} - \overline{\eta}_r}\right)^2\right]$$
(11)

where $R_{1/3}$ = significant runup height defined as the average of 1/3 highest values of R. The significant runup height is estimated as

$$R_{1/3} = (1+4S_r)(\overline{\eta}_r + 2\sigma_r) \tag{12}$$

If the probability distribution of η_r is Gaussian, $R_{1/3} = (\overline{\eta} + 2\sigma_r)$. The correction term (4*S_r*) in

Eq. (12) is obtained on the basis of the subsequent comparisons of the numerical model with the data by van Gent (1999a,b). The runup heights $R_{2\%}$ and $R_{1\%}$ corresponding to P = 0.02 and 0.01, respectively, in Eq. (11) are given by

$$R_{2\%} = \overline{\eta}_r + 1.40 \left(R_{1/3} - \overline{\eta}_r \right) \; ; \; R_{1\%} = \overline{\eta}_r + 1.52 \left(R_{1/3} - \overline{\eta}_r \right)$$
(13)

2.2 Input Parameters

The input to CSHORE includes two empirical parameters. The breaker ratio parameter γ involved in the energy dissipation rate D_B in Eq. (3) is taken as its default value of $\gamma = 0.7$. This parameter affects the cross-shore variation of the spectral significant wave height $H_{mo} = 4\sigma_{\eta}$. The computed H_{mo} is found to increase about 10% when γ is increased to 0.8. The computed runup is also found to increase about 10% when γ is changed from 0.7 to 0.8.

CSHORE does not separate wind and infragravity waves. The representative wave period, which has been taken as the spectral peak period T_p , is assumed to be invariant landward of the seaward boundary located at x = 0. The location of x = 0 is normally taken outside the surf zone so that the mean water level $\overline{\eta}$ above SWL may be assumed to be zero because the measured value of $\overline{\eta}$ is not available for practical applications. The values of H_{mo} and T_p at x =0 and the still water level above the datum need to be specified as input together with the bottom elevation z_b as a function of x. In addition, CSHORE does not account for reflected waves.

The other empirical parameter is the bottom friction factor f_b involved in the timeaveraged bottom shear stress in Eqs. (2) and (5) and the energy dissipation rate D_f in Eq. (3). The field observations of wave runup and swash velocities on natural beaches by Raubenheimer et al. (2004) indicated $f_b = 0.01 - 0.06$. CSHORE is calibrated initially using $f_b = 0.01$ and $f_b =$ 0.05. The computed runups using CSHORE are found to be insensitive to this range of f_b . A 500% change of f_b is found to cause less than 20% variations of $R_{2\%}$ and $R_{1\%}$. Use is made of $f_b =$ 0.02 in the following computations of van Gent's 1999 data in Chapters 3 - 5. The value of $f_b =$ 0.02 is now the default value for f_b for impermeable smooth slopes and sandy beaches. The computed H_{mo} is also found to be insensitive to changes in f_b . The time-averaged bottom shear stress is negative (onshore) due to the return (undertow) current and increases the cross-shore gradient of $\overline{\eta}$ in Eq. (2). As a result, the increase of f_b leads to the slight increase of the wave setup $\overline{\eta}$. The spectral significant wave height H_{mo} is reduced slightly by the increase of f_b and D_f in Eq. (3). As a whole, wave overtopping and runup on smooth impermeable slopes are not sensitive to f_b for the range of $f_b = 0.01$ to 0.06.

Chapter 3

WAVE RUNUP ON DIKE WITH BARRED BEACH

The numerical model CSHORE described in Chapter 2 is applied to data from a physical model based on Froud similitude in a wave flume that simulated field measurements as described by van Gent (2001). In this chapter, the computed significant wave height H_{mo} and extreme runup $R_{2\%}$ and $R_{1\%}$ using the numerical model CSHORE are compared to the measured data from the physical model. The first section of this chapter describes the basis of the physical model and its experimental setup. The second section describes the range of experimental conditions in the physical model. The final section of this chapter is separated into two subsections. The first subsection discusses the computed and measured H_{mo} . The second subsection discusses the computed and measured $R_{2\%}$ and $R_{1\%}$.

3.1 Physical Model

Field measurements were made of wave runup on a dike of the Petten Sea defense in The Netherlands as reported by van Gent (2001). A physical model based on Froude similitude with a length scale of 1/40 and a time scale of $\sqrt{40}$ was constructed in a wave flume to simulate the field measurements. The physical model was shown to reproduce the field data with the difference less than 10%. In the following comparison, use is made of the physical model data tabulated by van Gent (1999a) who presented the data in the prototype length and time scales.

This test series for the Petten Sea defense is called series P. Fig. 3.1 shows the geometry of the beach and dike in series P. The seaward boundary (x=0) of the CSHORE computation is taken at the most seaward location of the wave measurements. This seaward boundary location represents a gauge location immediately outside the surf zone during a storm, allowing of assumption that wave setup = 0 at x = 0 to be made. The dike consists of the slopes of 1/4.5, 1/20, and 1/3. The toe of the dike is located at x = 570 m. The bar crest is located in the vicinity of x = 160 m. The seaward and landward slopes of the bar are 1/30 and 1/25, respectively. The still water level S is varied up to 4.3 m. The vertical coordinate z is shown in Fig. 3.1 with z = 0 at the lowest still water level. The bar crest and dike toe are located at z = -4.8 and -1.9 m, respectively. The wave measurement will be discussed in the following sections.



Figure 3.1 Experimental setup for series P

3.2 Wave Conditions

The ranges of the wave conditions at x = 0 for 40 tests in series P are summarized in Table 3.1. The values of the still water level, spectral significant wave height, spectral period, and peak period at tabulated by van Gent (1999a) for each of the 40 tests in series P and are used as input to the numerical model CSHORE. The spectral significant wave height $H_{mo} = 4 \sigma_{\eta}$ is related to the root mean square wave height $H_{rms} = \sqrt{8} \sigma_{\eta}$ to be used as input to CSHORE. Three wave gauges were used to separate incident and reflected waves at x = 0, 160, 335, and 505 m. Incident waves at x = 570 m (toe location) were measured without the dike. Table 3.1 lists the range of periods and height of the incident waves for the 40 tests. The spectral wave period $T_{m-l,0}$ is defined as

$$T_{m-1,0} = \frac{m_{-1}}{m_0} \; ; \; m_n = \int_0^\infty f^n S(f) df \quad (n = 0 \text{ and } -1)$$
(14)

where S(f) = wave energy spectrum as a function of frequency f. The spectral period is now used in Europe (e.g., EurOtop Manual 2007) as a representative wave period instead of the spectral peak period T_p which is difficult to specify for multipeaked spectra. The spectral significant wave height H_{mo} was large enough for wave breaking over the bar when the still water level was low in Fig. 3.1. The bottom geometry depicted in Fig 3.1 is specified as input. The wave reflection coefficient K_R was about 0.3 at x = 0 and increased landward as observed on beaches with no structure (Baquerizo et al. 1997). The wave board was equipped with active wave absorption. The value of H_{mo} including the reflective waves at x = 0 may be estimated as $H_{mo} =$ $(1 + K_R^2)^{0.5}$ because partial standing waves decay seaward from the dike (e.g., Klopman and van der Meer 1999). H_{mo} may increase by 3 –7% in Table 3.1 if reflected waves with $K_R = 0.26$ – 0.37 are included. This estimate is useful in estimating the error of CSHORE which does not account for reflected waves.

C ani a c	Number of	$T_{m-1,0}$	T_p	H_{mo}	V
Series	Tests	(s)	(s)	(cm)	κ _R
Р	40	6.9 - 15.3	7.2 – 18.5	180 - 600	0.26 - 0.37

Table 3.1 Wave conditions at x = 0 for Series P.

Both $T_{m-1,0}$ and T_p at x = 0 are adopted as the representative period used in CSHORE to assess the period effect in CSHORE which assumes that the period is constant in the computation domain of x > 0. For the JONSWAP spectrum, $T_{m-1,0} = T_p/1.1$ (van Gent 1999a). The measured values of $T_{m-1,0}$ and T_p at the toe location and x = 0 are compared in Fig. 3.2 as a function of $T_{m-1,0}$ at x = 0. Fig. 3.1 shows the ratio between T_p at x=0 and $T_{m-1,0}$ at x=0 being in the range of 0.98 - 1.41. Both periods are affected by the decay of wind waves due to wave breaking and the generation of infragravity waves in the surf zone. The ratio between $T_{m-1,0}$ at the toe and x = 0 was in the range of 0.86 - 1.39. This ratio for T_p was in the range of 0.88 -2.32, indicating that T_p varied more from x = 0 to the toe. Based on these ratios, the assumption of constant wave period in CSHORE is more applicable to the spectral period $T_{m-1,0}$.



Figure 3.2 Wave period ratios as a function of $T_{m-1,0}$ at x = 0 for series P

3.3 Comparisons

CSHORE is used to compute the significant wave height H_{mo} , the extreme runup height of 2% exceedence probability, $R_{2\%}$, and the extreme runup height of 1% exceedence probability, $R_{1\%}$. The grid spacing of the CSHORE computation is 1.0 m to resolve the detailed wave transformation. The comparison between the measured and computed values is shown in the following sections. Both $T_{m-1,0}$ and T_p are used as input as the representative period in CSHORE and the differences between the two periods are also discussed.

3.3.1 Significant wave height

In order to show that CSHORE is capable of predicting the wave transformation from x = 0 to the toe of the dike, the measured wave heights H_{mo} at x = 0, 160, 335, 505, and 570 (toe) are compared to the computed results. The measured and computed cross-shore variations of H_{mo} are compared for each of the 40 tests in Appendix A. The comparison for test P27 (27th test in series P) is shown in Fig. 3.3 where S = 3.4 m in Fig. 3.1, and $T_{m-1,0} = 12.6$ s, $T_p = 14.4$ s, and $H_{mo} = 5.9$ m at x = 0. The computed wave setup above SWL is shown in the top panel to indicate that the berm of the 1/20 slope is submerged below the mean water level. The wet probability P_w is unity in the wet zone and decreases upward above SWL. The agreement of H_{mo} is similar for both periods and CSHORE overpredicts H_{mo} at the toe. Fig. 3.3 and the figures in Appendix A indicate the small differences between $T_{m-1,0}$ and T_p . The wave setup above the still water level and the wet probability are almost the same for $T_{m-1,0}$ and T_p . The significant wave height for T_p is slightly larger than that for $T_{m-1,0}$.



Figure 3.3 Cross-shore variation of wave setup, wet probability P_w and wave height H_{mo} for test P27

Fig. 3.4 displays the comparison of the measured and computed H_{mo} at x= 160, 335, 505 and 570 (toe) m for all 40 tests. The comparison of H_{mo} at the toe is differentiated because of the overprediction by CSHORE using $T_{m-1,0}$ and T_p at x = 0 and assuming the constant period. The perfect agreement and 20% deviations are indicated by a solid line and dashed lines, respectively in Fig. 3.4 and all subsequent figures unless otherwise specified. The root-mean-square relative error E is defined as

$$E = \left[\frac{1}{I}\sum_{i=1}^{I} \left(\frac{C_i}{M_i} - 1\right)^2\right]^{0.5}$$
(15)

where M_i and C_i = measured and computed values of the i-th point plotted in the figure, and I = number of the plotted points. The root-mean-square error is smaller for $T_{m-1,0}$, showing the agreement is slightly better for $T_{m-1,0}$ than T_p . The cause of the overprediction of H_{mo} at the toe might be related to the measurement of H_{mo} at the toe without the dike. This measurement neglects the effect of reflected waves on the incident waves. The values of H_{mo} at the other locations were obtained from the incident waves in the presence of the dike. This measurement based on three wave gauges and linear wave theory may not be very accurate for breaking waves. As a result, both methods for estimating the incident waves are not perfect.



Figure 3.4 Measured and computed wave heights H_{mo} at x = 160, 335, 505, and 570 (toe) m for series P
In addition to the overprediction at the toe, Fig. 3.4 shows that H_{mo} for T_p is predicted slightly larger than that for $T_{m-1,0}$, resulting in the better agreement (lower root mean square error) for the spectral period.

3.3.2 Wave runup

Wave runup on the dike was measured using a step gauge consisting of a beam with a large number of conductive probes. The probes were placed at a distance of $\delta_r = 0.1$ m (prototype scale) above the slope of 1/3 in Fig. 3.1. The exceedence probability P_I for each probe with the known elevation was obtained by dividing the contact number between the probe and water surface by the number N_I of incident waves in front of the dike. The exceedence probability P in Eq. (11) is based on the number N_R of individual runup heights. The relation of the two probabilities may be expressed as $P_I = P (N_R / N_I)$ where the ratio (N_R / N_I) tends to decrease from unity with the decrease of the dike slope. This ratio for the 40 tests in series P may be in the range of 0.7 - 1.0 on the basis of the empirical formula by Mase (1989). The runup heights for P = 0.02 and 0.01 given by Eq. (13) are not sensitive to the uncertainty of P of the order of 20%. As a result, $P_I = P$ is assumed in the following. Figs. 3.5 and 3.6 compare the measured and computed $R_{2\%}$ and $R_{1\%}$ above SWL, respectively, for the 40 tests in series P. The agreement for $R_{2\%}$ and $R_{1\%}$ is very similar because the measured $R_{2\%}$ and $R_{1\%}$ are well correlated and can be approximated by $R_{1\%} = 1.07 R_{2\%}$ within 10% errors. Eq. (13) predicts $R_{1\%}$ slightly larger than $R_{2\%}$.

The agreement for $R_{2\%}$ and $R_{1\%}$ is also similar for either $T_{m-1,0}$ or T_p at x = 0 as input to CSHORE. This difference in the representative wave period outside the surf zone results in small differences in computed runups in Figs. 3.5 and 3.6. This implies that the uncertainty of

the input wave period will be negligible (within 10 % error) if the seaward boundary x = 0 is selected to be outside but close to the surf zone. CSHORE predicts $R_{2\%}$ and $R_{1\%}$ within errors of about 20% partly because of the correction term added to Eq. (12).



Figure 3.5 Measured and computed 2% runup heights $R_{2\%}$ for series P



Figure 3.6 Measured and computed 1% runup heights $R_{1\%}$ for series P

Chapter 4

WAVE RUNUP ON DIKES WITH SLOPING BEACHES

In addition to the physical model testing of the barred beach and dike, van Gent (1999b) conducted experiments on physical models of dikes fronted by sloping beaches. This chapter uses these model tests to further assess the ability of CSHORE to predict the significant wave height and extreme runup. These tests included three different setups of beach and dike slopes. The water level and wave conditions including double-peaked wave energy spectra were varied for 97 tests in all. Like Chapter 3, the first section of this chapter describes these physical models for the three series with different beach and dike slopes. The second section describes the range of conditions for each of the three series. The third section is separated into two subsections and compares the measured and computed significant wave heights and the extreme runup heights of $R_{2\%}$ and $R_{1\%}$.

4.1 Physical Models

The experimental procedure for these models was essentially the same as that for series P. Use is made of the data tabulated by van Gent (1999b). Like in series P, the values of the still water level, significant wave height, spectral period, and peak period are used to make the three input files for CSHORE for the three test series, referred to as series A, B, and C by van Gent (1999b). The three test series were conducted for the beach slopes of 1/100 and 1/250 and the

dike slopes of 1/4 and 1/2.5 as shown in Fig. 4.1. Series A had a foreshore slope of 1/100 and a dike slope of 1/4, series B corresponded to a foreshore slope of 1/100 and a dike slope of 1/2.5, and series C had a foreshore slope of 1/250 and a dike slope of 1/2.5. The still water level S was varied up to 0.306 m. The wave reflection coefficient K_R at x = 0 was larger for series B and C with the dike slope of 1/2.5 as shown in the next section. The water depth at the toe located at x = 30 m was 4.7 cm below the lowest still water level. The degree of wave breaking on the beach increased with decrease of *S*. In Fig. 4.1, the datum z = 0 is chosen at the lowest still water level, S = 0, for all three series. The toe is located at x = 30 m and z = -0.047 m. The significant wave height used as input into CSHORE was measured at x = 0. For all 97 tests, the location x = 0 is mostly outside the surf zone, however when the still water level is very low this might not have been the case.

For each of these three series, the wave flume was divided into two test sections. One section was used to measure the wave runup height. The runup measurements will be discussed later on in this chapter. The other section of the wave flume was used to measure wave overtopping. The measured wave overtopping will be discussed in Chapter 5. For the runup computation, the landward limit of the dike is located 1.1 m above the toe for no wave overtopping as depicted in Fig. 4.1.



Figure 4.1 Experimental setup for series A (top), B (middle), and C (bottom)

4.2 Wave Conditions

The number of tests and the wave conditions at x = 0 are summarized in Table 4.1 where the wave conditions for the models (series A, B, and C) become similar to these for series P in Table 3.1 if use is made of the length and time scales as of 40 and $\sqrt{40}$, respectively, between the prototype and model.

Series	Number of	$T_{m-1,0}$	T_p	H_{mo}	K _R
	Tests	(s)	(s)	(cm)	
А	42	1.37 - 2.42	1.28 - 2.48	13.2 - 15.0	0.21 - 0.36
В	31	1.38 - 2.30	1.28 – 1.56	13.2 – 15.0	0.23 – 0.66
С	24	1.40 - 2.68	1.26 - 2.56	7.9 – 15.4	0.41 - 0.66

Table 4.1 Wave Conditions at x = 0 for Series A, B, and C

Fig. 4.2 shows the ratios of the wave periods at x = 0 and the toe for the 97 tests in series A, B, and C in the same way as in Fig. 3.2. The ratio $T_p/T_{m-1,0}$ at x = 0 is in the range of 0.70 - 1.45. The ratio between the measured periods at the toe and x = 0 is in the range of 1.01 - 4.47 for $T_{m-1,0}$ and 0.99 - 10.0 for T_p . The wave periods $T_{m-1,0}$ and T_p at x = 0 (mostly outside the surf zone) are not very different. The wave periods can increase considerably from x = 0 to the toe if wave breaking occurs on the gentle slope especially for double-peaked wave energy spectra. Like in series P (Fig. 3.2), the cross-shore variability is less for $T_{m-1,0}$ than T_p .



Figure 4.2 Wave period ratios as a function of $T_{m-1,0}$ at x = 0 for Series A, B, and C

4.3 Comparisons

CSHORE is used to compute H_{mo} , $R_{2\%}$, and $R_{1\%}$ for each test of series A, B, and C in the same way as in Chapter 3. These computed values for H_{mo} , $R_{2\%}$, and, $R_{1\%}$ are compared to the measured values. The measured H_{mo} at x = 0 was used as input to CSHORE. The wave transformation from x = 0 to the landward limit of wave uprush on the dike is computed for each test. Both $T_{m-1,0}$ and T_p are used as input for the representative period in CSHORE.

4.3.1 Significant wave height

Fig. 4.3 compares the measured and computed H_{mo} at x = 10, 20 and 30 (toe) m for series A, B, and C. The measured and computed cross-shore variations of H_{mo} for each of the tests (97)

in all) of series A, B, and C are reported in Appendices B, C, and D, respectively, in the same way as in Appendix A for series P. Unlike Fig. 3.4 for series P, the agreement remains similar at the toe for series A, B, and C. Consequently, the comparisons H_{mo} at x = 10, 20, and 30 m are presented together in Fig. 4.3 for series A, B, and C. The measured H_{mo} at the toe is not distinguished from the rest of the measurements as in Fig. 4.3. The agreement is similar for the spectral and peak periods. Fig. 4.3 shows that CSHORE predicts the wave height transformation for all 97 tests within about 10% errors.



Figure 4.3 Measured and computed wave heights at x = 10, 20, and 30 (toe) m for series A (top), B (middle), and C (bottom)

4.3.2 Wave runup

Figs. 4.4 and 4.5 compare the measured and computed $R_{2\%}$ and $R_{1\%}$, respectively. The height δ_r (see Fig. 2.1) of the step gauge was $\delta_r = 2.5$ mm. All the 97 tests are plotted together because the agreement is similar for the three series. CSHORE predicts $R_{2\%}$ and $R_{1\%}$ within errors of about 20% when $T_{m-1,0}$ at x = 0 is used as the representative wave period. van Gent (2001) developed an empirical formula for $R_{2\%}$ using the measured values of H_{mo} and $T_{m-1,0}$ at the toe in series P, A, B, and C where $T_{m-1,0}$ was shown to be a better representative period for the formula than T_p . This is also seen in the computations based on CSHORE. In Figs. 4.4 and 4.5, the agreement for $T_{m-1,0}$ is slightly better than the agreement of T_p . Figs. 4.4 and 4.5 also show a slight systematic error where CSHORE tends to overpredict larger runup heights and underpredict smaller runup heights.

The agreement shown in Figs. 3.5 and 4.4 for CSHORE is no better than the simple empirical formula by van Gent (2001). For actual applications, the empirical formula is difficult to apply if the toe of the dike is located well inside the surf zone because spectral wave models such as SWAN (Booij et al. 1999) limited to wind wave frequencies may not predict the wave periods $T_{m-1.0}$ and T_p at the toe accurately. CSHORE may be applied if its seaward boundary location is chosen to be within the zone where the existing wind wave models can predict H_{mo} , $T_{m-1.0}$ and T_p accurately. This practical approach avoids the prediction of infragravity waves in the surf zone. As a result, CSHORE may be a good choice for practical applications such as coastal flood risk mapping.



Figure 4.4 Measured and computed 2% runup heights $R_{2\%}$ for series A, B, and C



Figure 4.5 Measured and computed 1% runup heights $R_{1\%}$ for series A, B, and C

Chapter 5

MINOR WAVE OVERTOPPING

This chapter presents the computations made by CSHORE to predict minor wave overtopping. The 97 tests by van Gent (1999b) in Chapter 4 included the measurement of wave overtopping rates. The first section of this chapter discusses the experimental setup used for the wave overtopping measurements. The second explains the degree of the measured wave overtopping. The final section compares the measured and computed overtopping rates.

5.1 Experimental Setup

For series A, B, and C, the 1-m wide flume used in the experiment was divided into two sections separated by a thin plate. The wave runup measurement was conducted in the section where the dike was high enough for no wave overtopping. In the other section, the dike crest was lower to allow wave overtopping for some tests (van Gent 1999b).

There were three different crest elevations, R_c , used in the overtopping section of the flume. The first crest was located 0.654 m above the bottom of the flume (datum used for series A, B, and C) to measure the wave overtopping rate for the lowest still water level (SWL) of 0.494 m above the bottom of the flume. The first crest was 0.16 m above the SWL. The second crest located 0.898 m above the bottom of the flume was used to measure the wave overtopping rate for the intermediate still water level of 0.588 m above the bottom of the flume. The second

crest height was 0.31 m above the SWL. The third crest located 1.153 m above the bottom of the flume was used to measure the wave overtopping rate of the highest still water level of 0.753 m above the bottom of the flume. The third crest height was 0.4 m above the SWL. In short, these combinations of the crest height and SWL were selected to produce no or minor wave overtopping.

The measured wave overtopping rate q_o was regarded to be unreliable if q_o was less than about 1 ml/s/m where 1 ml (milliliter) equals 10^{-6} m³. For a length scale of 1/40, this minimum rate in the physical model corresponds to 0.25 l/s/m in the prototype. The overtopping rate of 1 l/s/m is considered to be allowable for the design of a dike (EurOtop Manual 2007). It should be noted that the wave overtopping rate measurement does not depend on the height δ_r (see Fig. 2.1) of the step gauge (or runup wire) where this height δ_r is known to have noticeable effects on the runup measurement (e.g., Raubenheimer and Guza 1996).

5.2 Threshold of Wave Overtopping

Fig. 5.1 shows the measured overtopping rate q_o over the dike crest height R_c above SWL in one section of the flume as a function of $(R_{1\%} - R_c)$ where $R_{1\%}$ is the measured 1% runup height above SWL in the other section of the flume. For the logarithmic plot of q_o , use is made of $q_o = 1$ ml/s/m if $q_o < 1$ ml/s/m. Table 5.1 lists the number of tests with $q_o > 1$ ml/s/m in comparison to the total number of tests in series A, B, and C. The different dike crests heights R_c appears to have been chosen so as to examine the threshold of wave overtopping. Fig. 5.1 indicates the difficulty in predicting the overtopping rate q_o near the threshold even when the measured $R_{1\%}$ is known. Wave overtopping occurred when the crest height of the dike was clearly exceeded by the measured runup height $R_{1\%}$. The transition of no wave overtopping ($R_{1\%}$ sufficiently smaller than R_c) and wave overtopping occurred for the range of $q_o = 1 - 10$ ml/s/m.



Figure 5.1 Measured wave overtopping rate q_o over crest height R_c above SWL as a function of $(R_{1\%} - R_c)$

Table 5.1 Number of Tests with Overtopping Rates $q_o > 1$ ml/s/m

	Number	Number of tests with $q_o > 1$ ml/s/m			
Series	of tests	Measured	Computed $(T_{m-1,0})$	Computed (T_p)	
A	42	20	26	31	
В	31	27	20	23	
С	24	15	6	11	

5.3 Wave Overtopping Rate

The computation of wave overtopping using CSHORE is made for the dike geometry with its crest located at the specified elevation R_c above SWL. The overtopping rate q_o is predicted using Eq. (9) if the CSHORE computation reaches the landward end at $x = x_c$ of the input bottom geometry. If the computation does not reach $x = x_c$, no wave overtopping occurs and $q_o = 0$. Table 5.1 lists the number of tests with the computed $q_o > 1$ ml/s/m. The wave overtopping computations are made using the measured $T_{m-1.0}$ and T_p at x = 0 for each of the 97 tests in Table 5.1. The number of tests with $q_o > 1$ ml/s/m is overpredicted for series A and underpredicted for series B and C. The slope 1/4 of series A is gentler than the slope 1/2.5 of series B and C. The use of T_p produces a greater number of tests with $q_o > 1$ ml/s/m for all the series.

Fig. 5.2 compares the measured and computed q_o for series A, B, and C where use is made of $q_o = 1$ ml/s/m if $q_o < 1$ ml/s/m for the logarithmic plot of q_o . The solid line and dashed lines indicate the perfect agreement and 1,000% (a factor of 10) error. CSHORE can predict only the order of magnitude of q_o for the case of minor wave overtopping where the crest height is close to the 1% runup height. The overtopping rate tends to be overpredicted for series A (circles), and underpredicted for series B and C (squares and triangles, respectively). This trend is consistent with the comparison in Table 5.1. Fig. 5.2 also shows that the use of T_p tends to yield higher overtopping rates than $T_{m-1,0}$.

An empirical formula for q_o was developed by van Gent (1999b) using the measured values of H_{mo} and $T_{m-1,0}$ at the toe. His formula predicts $q_o > 0$ even for the case of no wave overtopping. This is also the case with other available formulas (e.g., EurOtop Manual 2007). His formula predicts q_o somewhat better because the comparison is limited to the tests with the measured $q_o > 1$ ml/s/m. In any case, the threshold of wave overtopping is very difficult to predict accurately because of the very small water depth in the upper limit of the wet and dry zone. The agreement in Fig. 5.2 could be improved by calibrating the bottom friction factor f_b (f_b = 0.02 in Fig. 5.2) for each of series A, B, and C, but the overall agreement will not improve significantly.



Figure 5.2 Measured and computed wave overtopping rates q_o for series A, B, and C

Chapeter 6

WAVE RUNUP ON GENTLE SLOPES

The comparisons in Chapters 3 and 4 are limited to wave runup on dikes with barred and sloping beaches. This chapter assesses the applicability of CSHORE to gentler impermeable slopes. The applicability of CSHORE to gentler slopes is examined by comparing CSHORE with the smooth impermeable slope tests by Mase (1989). The first section of this chapter describes the experimental setup and wave conditions. The second section compares the computed and measured runup heights. The final section of this chapter discusses the applicability of CSHORE to natural beaches.

6.1. Experimental Setup and Wave Conditions

Table 6.1 summarizes the wave conditions at the toe of the 1/5, 1/10, 1/20 and 1/30 slopes in the experiment by Mase (1989) where 30 tests were conducted for each slope. For brevity, the four different slopes are called slopes A, B, C, and D. The significant wave height and wavelength in deep water were tabulated for each test. Linear theory for wave shoaling is used to calculate the significant wave height and period at the toe of the uniform slope in water depth of 43 or 45 cm. The seaward boundary x = 0 for CSHORE is taken at the toe. The significant wave period T_s at x = 0 is the representative wave period in this comparison. The shoaled significant wave height is assumed to be the same as the spectral significant wave height

 H_{mo} at x = 0 required as input for CSHORE. Comparison of Tables 3.1 and 4.1 with Table 6.1 indicates that these uniform slope tests included smaller periods and heights.

Slope	Uniform	Number	Depth	Ts	H_{mo}
name	slope	of tests	(cm)	(s)	(cm)
А	1/5	30	45	0.84 - 2.42	4.0 - 10.2
В	1/10	30	45	0.84 - 2.29	2.9 - 10.2
С	1/20	30	45	0.83 - 2.28	2.7 - 9.3
D	1/30	30	43	0.81 – 2.29	2.6 - 9.2

Table 6.1 Wave Conditions at x= 0 for Four Uniform Slopes

The shoreline oscillation on the uniform slope was measured using a capacitance runup wire that was 2 m long with a diameter of 2.2 mm. The runup wire was installed in a 3 cm wide and 1 cm deep groove along the center of the slope so that the runup wire was at the same elevation of the slope surface. This runup measurement is consistent with the runup model in CSHORE except for the groove. The height δ_r of the runup wire in Fig. 2.1 is assumed to be $\delta_r = 1$ mm which corresponds to the radius of the wire. The groove effect on wave runup is crudely accounted for by calibrating the bottom friction factor f_b in CSHORE where $f_b = 0.02$ for series P, A, B, and C. The value of f_b calibrated for the 120 tests in Table 6.1 is $f_b = 0.001$, implying that the groove might have reduced the bottom shear stress experienced by the shoreline oscillation.

6.2 Wave Runup

The measured and computed $R_{2\%}$ and $R_{1/3}$ are compared in Figs. 6.1 and 6.2, respectively, where the significant runup height $R_{1/3}$ is predicted using Eq. (12). The measured $R_{2\%}$ and $R_{1/3}$ are well correlated and can be approximated by $R_{2\%} = 1.34 R_{1/3}$. CSHORE with $f_b = 0.001$ predicts $R_{2\%}$ and $R_{1/3}$ within errors of about 20% for the four slopes but the root-mean-square relative error E defined by Eq. (15) varies among the four slopes where the value of E for the four slopes are listed in Figs. 6.1 and 6.2. Mase (1989) proposed empirical formulas for $R_{2\%}$ and $R_{1/3}$ using his data. The agreement is slightly better for his formulas which are limited to uniform slopes. CSHORE is versatile enough to predict wave runup on the slope of an arbitrary geometry.



Figure 6.1 Measured and computed 2% runup heights R_{2%} for slopes A, B, C, and D



Figure 6.2 Measured and computed significant runup heights $R_{1/3}$ for slopes A, B, C, and D

6.3 Wave Runup on Natural Beaches

CSHORE is also compared qualitatively with the sets of wave runup data on natural beaches assembled by Stockdon et al. (2006) who developed an empirical formula for the 2% runup height $R_{2\%}$ using the assemble data. This formula expresses $R_{2\%}$ in terms of the deepwater significant wave height, the deep-water wavelength based on the spectral peak period, and the foreshore beach slope. The runup data were collected using video techniques. Holman and Guza (1984) compared wave runup measurements based on resistance wires and films. Their limited intercomparison on a natural beach indicated appreciable differences. The runup model in CSHORE corresponds to the measurement using a runup wire as shown in Fig. 2.1. Stockdon

et al. (2006) presented the time-averaged beach profile near the shoreline for each data set. The wave transformation computation using CSHORE requires the entire beach profile from the seaward boundary to the landward limit of wave action. The bulk of the data (91%) was collected at the U.S. Army Corps of Engineers Field Research Facility (FRF) in Duck, NC. The wave measurements in the vicinity of the FRF pier by Elgar et al. (2001) indicated reduction (as much as 50%) of wave energy downwave of the pier. In short, additional uncertain assumptions are required to compare CSHORE with these data sets. The predictive capability of CSHORE is found to be no better than the simple formula by Stockdon et al. (2006). In other words, it is not worth applying CSHORE if the input to CSHORE is highly uncertain.

Beach and dune profile evolution during a severe storm will need to be predicted for coastal flood risk mapping. The formula by Stockdon et al. (2006) indicates that $R_{2\%}$ is approximately proportional to the foreshore beach slope except for extremely dissipative conditions. The foreshore beach slope can change considerably during a storm. This implies that the runup formula will need to be coupled with a model for beach and dune profile evolution. Alternatively, CSHORE can be used to predict the beach and dune profile evolution and the time series of the wave overtopping rate at the land end of the computation domain during a storm as has been attempted by Figlus et al. (2011) for laboratory data.

Chapter 7

CONCLUSIONS

The cross-shore numerical model CSHORE is extended to predict irregular wave runup on impermeable dikes. CSHORE is compared to wave runup data by van Gent (1999a and b) for a dike on a barred beach and dikes on gently sloping beaches with a total of 137 wave runup tests. The computation of CSHORE is initiated from the seaward location where wave setup may be assumed negligible. The measured spectral period and peak period were not too different at the seaward boundary of the CSHORE computation and are used as the representative period for the computation. CSHORE does not predict the cross-shore variations of these periods but can predict the cross-shore variation of the spectral significant wave height on the barred and sloping beaches in front of the dikes. CSHORE also predicts the 2% and 1% exceedence runup heights within errors of about 20%. The agreement is slightly better for the spectral period perhaps because the cross-shore variation of the spectral period was less than that of the peak period. CSHORE is also compared with 97 wave overtopping tests by van Gent (1999b). CSHORE predicts the threshold of wave overtopping but can predict only the order of magnitude of the relatively small wave overtopping rate.

The capability and limitation of CSHORE for predicting wave runup on gentle slopes and beaches are examined using the laboratory data by Mase (1989) and the field data by Stockdon et al. (2006). The bottom friction factor used in CSHORE is required to be calibrated to account

42

for the laboratory setup for the runup measurement and predict the measured runup heights for the 120 tests within errors of about 20%. The quantitative comparison with the field data is not feasible for lack of the input data required for the CSHORE computation. For coastal flood risk mapping, beach and dune profile evolution during a severe storm will need to be predicted because wave runup and overtopping depend on the foreshore and dune profile. CSHORE has been shown to be capable of predicting the beach and dune profile evolution within a factor of about 2 (Kobayashi et al. 2010b; Figlus et al. 2011). Consequently, CSHORE is a useful tool for coastal flood risk mapping.

REFERENCES

- Baquerizo, A., Losada, M.A., Smith, J.M., and Kobayashi, N. (1996). "Cross-shore variation of wave reflection from beaches." J. Waterway, Port, Coastal, Ocean Eng., 123(5), 274-279.
- 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.
- Crowell, M., Coulton, K., Johnson, C., Westcott, J., Bellomo, D., Edelman, S., and Hirsch, E. (2010). "An estimate of the U.S. population living in 100-year coastal flood hazard area." J. Coastal Res., 26(2), 201-211.
- Elgar, S.T., Guza, R.T., O'Reilly, W.C., Raubenheimer, B., and Herbers, T.H.C. (2001). "Wave energy and direction observed near a pier." J. Waterway, Port, Coastal, Ocean Eng., 127(1), 2-6.
- EurOtop Manual. (2007). Wave overtopping of sea defenses and related structures: Assessment Manual. <u>http://www.overtopping-manual.com</u>.
- Figlus, J., Kobayashi, N., Gralher, C., and Iranzo, V. (2011). "Wave overtopping and overwash of dunes." J. Waterway, Port, Coastal, Ocean Eng., 137(1), 26-33.
- Holman, R.A. (1986). "Extreme value statistics for wave run-up on a natural beach." Coastal Eng., 9, 527-544.
- Holman, R.A., and Guza, R.T. (1984). "Measuring run-up on a natural beach." Coastal Eng., 8, 129-140.
- Klopman, G., and van der Meer, J.W. (1999). "Random wave measurements in front of reflective structures." J. Waterway, Port, Coastal, Ocean Eng., 125(1), 39-45.

Kobayashi, N. (1999). "Wave runup and overtopping on beaches and coastal structures."

Advances in Coastal and Ocean Engineering, Vol. 5, World Scientific, Singapore, 95-154.

- Kobayashi, N., de los Santos, F.J., and Kearney, P.G. (2008). "Time-averaged probabilistic model for irregular wave runup on permeable slopes." J. Waterway, Port, Coastal, Ocean Eng., 134(2), 88-96.
- Kobayashi, N., DeSilva, G.A., and Watson, K.D. (1989). "Wave transformation and swash on gentle and steep slopes." J. Geophys. Res., 94(C1), 951-966.
- Kobayashi, N., Farhadzadeh, A., and Melby, J.A. (2010a). "Wave overtopping and damage progression of stone armor layer." J. Waterway, Port, Coastal, Ocean Eng., 136(5), 257-265.
- Kobayashi, N., Pietropaolo, J.A., and Melby, J.A. (2011). "Wave transformation and runup on dikes and gentle slopes." J. Waterway, Port, Coastal, Ocean Eng., (submitted).
- Kobayashi, N., Farhadzadeh, A., Melby, J.A., Johnson, B., and Gravens, M. (2010b). "Wave overtopping of levees and overwash of dunes." J. Coastal Res., 26(5), 888-900.
- Mase, H. (1989). "Random wave runup height on gentle slope." J. Waterway, Port, Coastal, Ocean Eng., 115(5), 649-661.
- Nwogu, O., and Demirbilek, Z. (2010). "Infragravity wave motions and runup over shallow fringing reefs." J. Waterway, Port, Coastal, Ocean Eng., 136(6), 295-305.
- Raubenheimer, B. (2002). "Observations and predictions of fluid velocities in the surf and swash zones." J. Geophys. Res., 107(C11), 3190.
- Raubenheimer, B., and Guza, R.T. (1996). "Observations and predictions of runup." J. Geophys. Res., 101(C10), 25,575-25,587.

Raubenheimer, B., Guza, R.T., and Elgar, S. (2001). "Field observations of wave-driven

setdown and setup." J. Geophys. Res., 106(C3), 4629-4638.

- Reubenheimer, B., Elgar, S., and Guza, R.T. (2004). "Observations of swash zone velocities: A note on friction coefficients." J. Geophys. Res., 109, C01027.
- Stockdon, H.F., Holman, R.A., Howd, P.A., and Sallenger, A.H. (2006). "Empirical parameterization of setup, swash, and runup." Coastal Eng., 53, 573-588.
- van Gent, M.R.A. (1999a). "Physical model investigations on coastal structures with shallow foreshores: 2D model tests on the Petten Sea – defense." MAST-OPTICREST Rep. and Delft Hydr. Rep. H3129, Delft Hydraulics, Delft, The Netherlands.
- van Gent, M.R.A. (1999b). "Physical model investigations on coastal structures with shallow foreshores: 2D model tests with single and double-peaked wave energy spectra." Rep. H3608, Delft Hydraulics, Delft, The Netherlands.
- van Gent, M.R.A. (2001). "Wave runup on dikes with shallow foreshores." J. Waterway, Port, Coastal, Ocean Eng., 127(5), 254-262.

Appendix A

CROSS-SHORE VARIATIONS OF WAVE SETUP AND HEIGHTS FOR 40 TESTS IN SERIES P

For each of the 40 tests in series P, the computed cross-shore variation of wave setup above the datum for series P and the bottom elevation z_b are plotted in the top panel of each figure where wave setup above the still water level is zero at x = 0. The peak period T_p and spectral period $T_{m-I,0}$ at x = 0 are used as the representative period specified as input to examine the degree of the period effect. The computed spectral significant wave height H_{mo} and the measured values of H_{mo} at x = 0, 160, 335, 505, and 570 (toe) m are plotted in the middle panel of each figure. The computed wet probability P_w is plotted in the bottom panel of each figure where $P_w = 1$ seaward of the still water shoreline.

A.1 Test P1



A.3 Test P3







A.5 Test P5


A.7 Test P7



A.9 Test P9



A.11 Test P11



A.13 Test P13



A.15 Test P15



A.17 Test P17



A.19 Test P19



A.21 Test P21



A.23 Test P23



A.25 Test P25



A.27 Test P27



A.29 Test P29



A.31 Test P31



A.33 Test P33



A.35 Test P35



A.37 Test P37



A.39 Test P39



Appendix B

CROSS-SHORE VARIATIONS OF WAVE SETUP AND HEIGHTS FOR 42 TESTS IN SERIES A

For each of the 42 tests in series A, the computed cross-shore variation of wave setup above the datum for series A and the bottom elevation z_b are plotted in the top panel of each figure where wave setup above the still water level is zero at x = 0. The peak period T_p and spectral period $T_{m-1,0}$ at x = 0 are used as the representative period specified as input to examine the degree of the period effect. The computed spectral significant wave height H_{mo} and the measured values of H_{mo} at x = 0, 10, 20, and 30 m are plotted in the middle panel of each figure. The computed wet probability P_w is plotted in the bottom panel of each figure where $P_w = 1$ seaward of the still water shoreline.

B.1 Test A1



69

B.3 Test A3







B.5 Test A5



B.7 Test A7



72

B.9 Test A9



B.11 Test A11



B.13 Test A13



B.15 Test A15



B.17 Test A17



B.19 Test A19



B.21 Test A21



B.23 Test A23



B.25 Test A25



B.27 Test A27



B.29 Test A29



B.31 Test A31



B.33 Test A33


B.35 Test A35



B.37 Test A37



B.39 Test A39



B.41 Test A41



Appendix C

CROSS-SHORE VARIATIONS OF WAVE SETUP AND HEIGHTS FOR 31 TESTS IN SERIES B

For each of the 31 tests in series B, the computed cross-shore variation of wave setup above the datum for series B and the bottom elevation z_b are plotted in the top panel of each figure where wave setup above the still water level is zero at x = 0. The peak period T_p and spectral period $T_{m-1,0}$ at x = 0 are used as the representative period specified as input to examine the degree of the period effect. The computed spectral significant wave height H_{mo} and the measured values of H_{mo} at x = 0, 10, 20, and 30 m are plotted in the middle panel of each figure. The computed wet probability P_w is plotted in the bottom panel of each figure where $P_w = 1$ seaward of the still water shoreline.

C.1 Test B1







C.3 Test B3







C.5 Test B5



C.7 Test B7



C.9 Test B9



C.11 Test B11



C.13 Test B13



C.15 Test B15



C.17 Test B17



C.19 Test B19



C.21 Test B21



C.23 Test B23



C.25 Test B25



C.27 Test B27



C.29 Test B29



C.31 Test B31



Appendix D

CROSS-SHORE VARIATIONS OF WAVE SETUP AND HEIGHTS FOR 24 TESTS IN SERIES C

For each of the 24 tests in series C, the computed cross-shore variation of wave setup above the datum for series C and the bottom elevation z_b are plotted in the top panel of each figure where wave setup above the still water level is zero at x = 0. The peak period T_p and spectral period $T_{m-1,0}$ at x = 0 are used as the representative period specified as input to examine the degree of the period effect. The computed spectral significant wave height H_{mo} and the measured values of H_{mo} at x = 0, 10, 20, and 30 m are plotted in the middle panel of each figure. The computed wet probability P_w is plotted in the bottom panel of each figure where $P_w = 1$ seaward of the still water shoreline. D.1 Test C1







D.3 Test C3







D.5 Test C5



D.7 Test C7



D.9 Test C9



D.11 Test C11



D.13 Test C13



D.15 Test C15



D.17 Test C17



D.19 Test C19



D.21 Test C21



D.23 Test C23

