

CHAPTER 205

CROSS-SHORE PROFILE MODELLING UNDER RANDOM WAVES

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Abstract

This paper presents part of the test results conducted in the Large Wave Flume (LWF) for 2D beach profile response under random wave input as well the numerical modelling effort to simulate the laboratory data. The tests were conducted with various input wave spectra, initial profiles and for both erosional and accretional cases. The numerical model is a 2DV(two dimensional vertical) beach profile model. It couples a sediment transport model with a random wave model. The sediment model is a modified version of the SBEACH model developed by Larson and Kraus (1989) for regular waves. The input wave condition is a time-series of irregular waves simulated from a given wave height probability density function, here selected as the Rayleigh distribution. The final profile is then computed from the cumulative changes due to each randomly occurring individual wave.

Introduction

In the last few decades, many studies have been carried out for modelling beach profile evolution under storm wave conditions. Most of these efforts, whether physical or numerical, have been for regular waves and for beaches under the state of erosion. It is only natural to extend the effort for cases of random waves. Therefore, since 1987, a systematic series of experiments were carried out in the Large Wave Flume (LWF) located in Hannover, Germany to study the beach response under random wave inputs. The tests were conducted for various types of input wave spectra, with different initial slopes and for both accretional and erosional conditions. In a parallel effort, attempts were made to simulate the laboratory results with numerical model. The numerical model is largely built upon existing modeling techniques.

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Two different approaches have evolved in the past decades for beach evolution modelling. One approach is to establish the mechanics of sediment mobilization and transport first on micro-scale; transport models are then constructed at integrated temporal and spatial scales. This approach is referred to as mechanics approach or fine-scale approach. This type of model could yield better temporal and spatial resolution but may not be practical for long term or large spatial scale simulation. The other approach avoids the details of the mechanics of sediment transport and simply attempts to relate sediment transport to flow properties on a macro scale. The commonly inferred flow properties are temporal and spatially averaged wave energy, wave energy flux or rate of wave energy dissipation. Sediment transport models of this kind ignore the details and bypass the basic sediment mechanics; they are, more or less, heuristic and mainly based upon physical reasoning and/or empirical evidence. This approach is referred to as heuristic approach or macro-scale approach. Since models of the latter type deal with integrated flow properties they are usually more suitable for long term simulation. At present, most of the workable models are of the latter type. And, as mentioned they are mostly restricted to 2D, regular wave input and erosional case. In principle, extending these models for irregular wave application presents no fundamental difficulty, although different approaches can be taken. Indeed, various efforts have been made towards this extension. One of the major handicaps has been the lack of adequate data for calibration and verification.

One of the objectives of the present study is to develop and test a 2DV numerical model that simulates the changes of cross-shore profile under random wave action. The model like most of its kind is composed of a sediment transport model driven by a hydrodynamic model. The beach profile change is then computed by use of conservation of sediment mass. The model is intended for engineering application. The aim is to keep the basic characteristics of the model simple, but still reliable and realistic. Generality and mathematical rigor are, at times, compromised.

Laboratory Measurements and Results

The experiments were carried out in the LWF located in the University of Hannover, which is 320 m long, 5 m wide and 7 m deep and is capable of generating waves up to 2 m high. Beach profile response tests under random waves inputs were carried out in four test series in 1986/87, 90, 91 and 93, respectively. The 1986/87 test was concentrated on dune erosion under regular and irregular waves. The same initial profile were exposed to both regular waves and irregular waves with height of 1.5 m and period of 6 sec. In 1990 and 1991, the main concern of the experiments was to measure the energy dissipation and the sediment transport with an initial equilibrium profile. In 1993, experiments were carried out to study both erosional profile evolution under short waves and accretionary profile evolution under long waves. The effect of water level variations due to tide effect was also examined. The test conditions are given in Table 1.

Table 1 GWK TEST SCHEDULE

Spectrum type	Initial Profile	H_s (m)	T_p (sec)	Water Depth,	Duration (hr)	Response Type
JONSWAP (1987)	$m=1:20$, $D_{50}=0.33$ (mm)	1.5	6	5 m	9.8	Erosion
PM(1990)	$h=Ax^{2/3}$, $A=0.08$ $D_{50}=0.22$	0.8	6 - 8	2 m	9.0	Equilibrium
TMA (1991)	$h=Ax^{2/3}$; $A=0.08$ $D_{50}=0.33$	0.9- 1.04	6 - 8	2.5 m	9.6	Equilibrium
TMA (1993)	Uniform slope $m=1:30$ $D_{50}=0.22$	1.2	5-10	4.5 m	23.5 (no tide) 84.0 (with tide)	Erosion and Accretion

Detailed results can be found in Wu (1994). Only a selected sets of results from 1993 tests were given here. The 1993 test was a long sequential experiment of 3 phases. Phase I was regular waves of erosional(short wave period)-accretional(long wave period)-erosional(short wave period) sequence. It was followed by Phase II test of irregular wave input with short period waves followed by long period wavesl. Phase III has the longest test period with irregular wave input coupled with water level variations, consisting of 45 hrs of short period wave test followed by 39 hrs of long period wave. The test conditions are summarized in Table 2.

Table 2 Summary of Test Conditions, 1993**PHASE I. REGULAR WAVE**

H=1.2 M; T=5 SEC	H=1.2 M; T=10 SEC	H=1.2 M; T=5 SEC
17 HRS	15.75 HRS	7.33 HRS

PHASE II. IRREGULAR WAVE, NO TIDE

$H_s=1.2$ M; $T_p=5$ SEC	$H_s=1.2$ M; $T_p=10$ SEC
13 HRS	10.5 HRS

PHASE III. IRREGUAR WAVE, WITH TIDE

$H_s=1.1$ M; $T_p=5$ SEC	$T_s=1.1$ M; $T_p=10$ SEC
45 HRS	39 HRS

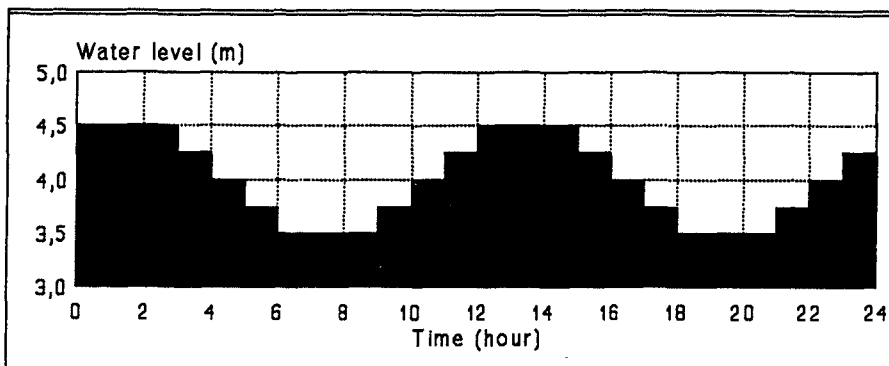


Figure 1 shows the end profiles of Phase I tests. In the first 17 hrs. of phase I-1, the 5 sec waves built a bar 0.75 m high. Then under 15.75 hrs. action of 10 sec waves in phase I-2, the bar moved about 3 m in onshore direction. In the next 17 hrs. of short waves, the bar moved offshore again and a smaller inner bar was developed. The change in the dune region took place mainly in the 2 short-wave phases.

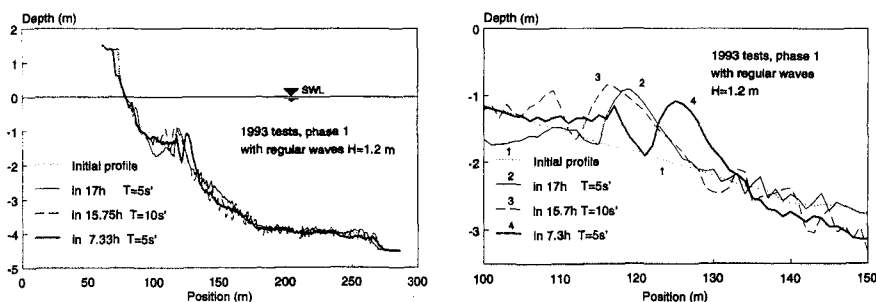


Figure 1. Profile Evolutions in Phase I test, 1993

Figure 2 shows the end profiles of Phase II tests. It is quite apparent that the initial bar-trough profile was smoothened into a foreshore tarrace during the short period wave test. In the subsequent long-period wave test, onshore sediment motion was observed. Different from the regular wave case, the bar was much lower and broader. The bed changes in the inner surf zone were also more rigorous and rapid. The results of Phase III test are given in **Fig. 3**. In this Phase, water surface variations due to tide (12-hr period) were also simulated in addition to the irregular wave input.

During the short-period wave test, the variation of water depth appeared to aid in offshore sediment movement causing the bar to move further offshore. In the subsequent long-period wave test, the development of the profile was very gradual as the bar was flattened and moved inshore.

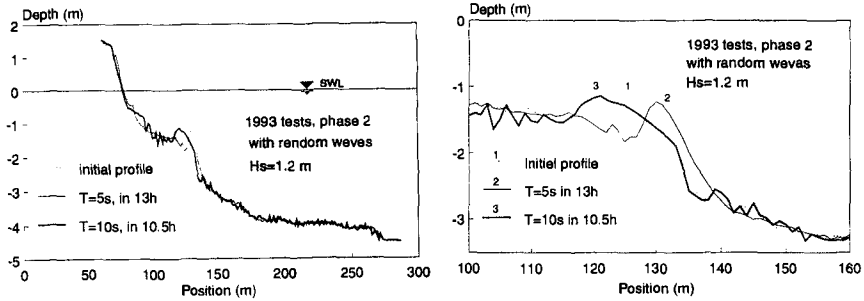


Figure 2. Profile Evolutions in Phase II test

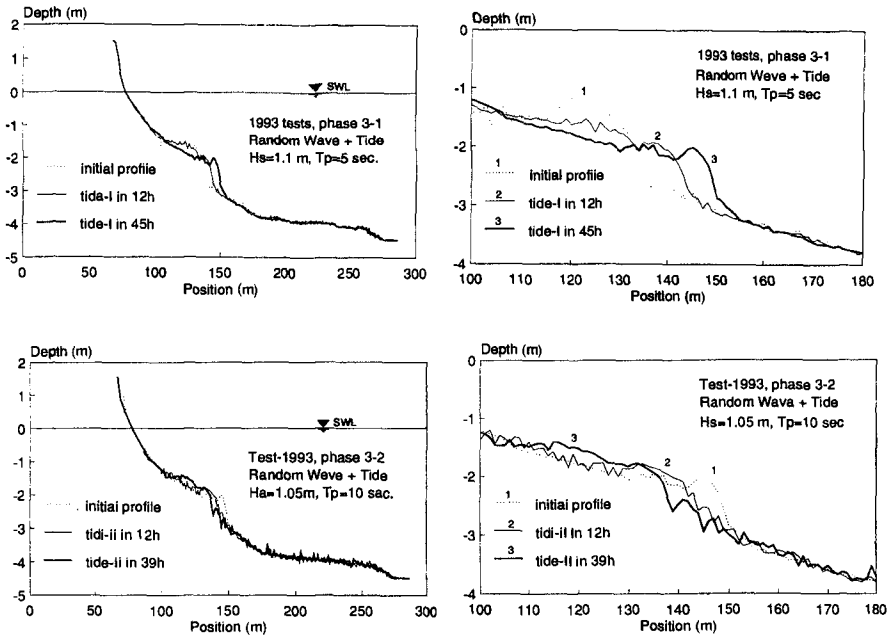


Figure 3. Profile Evolutions in Phase III test

Numerical Modelling With Random Waves

The profile response model consists of two major parts: the wave input model and the sediment transport model. Conceptually, the randomness should be entered in both parts. At present the randomness can be incorporated in the input wave only.

(1) Random Wave Model

There appear to be three basic approaches for treatment of random wave transformation in shoaling water and through the surf zone: They are: (1). Using a deepwater random wave time series as input and transforming each individual wave in the series as if it were a regular wave component with a distinct wave amplitude and period. (2). Carrying out a spectral transformation first from deepwater into shallow water. A time series is then created from the shallow water spectrum for further shoaling and surf zone transformation of each individual wave. The first step is equivalent to transformation of Fourier components under the constraint of energy conservation. Numerous numerical models can be used to accomplish this transformation. (3). Conducting a "parametric" type transformation of deepwater random wave directly into surf zone. The surf zone wave properties are then expressed as significant wave parameters and, sometimes, with associated distribution functions. Each approach presented above can be considered for random wave input information. Approach (1) and (2) are conceptually the same except that another layer of model is required in (2) to carry out shallow water wave transformation. The third approach yields local wave information inside the surf zone but it is difficult to reconstruct a continuous spatial variations of each wave which is required in some of the sediment transport models.

The wave model used here was based on approach (1) for its directness. The essential assumptions are: shoaling and breaking are not affected by wave-wave interaction, reflection is weak and the waves are unidirectional. The method is called as the "time dependent discrete pdf method". As an initial attempt the input wave is assumed to be uni-directional and narrow-banded in wave period with the deepwater wave heights following Rayleigh distribution.

First, a random wave height series is generated by the Monte-Carlo method. If ρ is a uniformly distributed random number with a value between zero and one, the Rayleigh distributed wave height corresponding to this level of probability is given ;

$$H_p = H_{rms} \sqrt{\ln(1/\rho)} \quad (1)$$

in which H_p is the wave height that is exceeded by ρN waves, $\ln$ is natural logarithm and H_{rms} is root-mean-square wave height. The H_{rms} value is the only required input.

Each wave height from this random series is treated as a regular wave in one profile simulation time-step and is transformed through the computational space by the wave

decay models of Dally et al. (1984). The number of waves used in one simulation run is determined by comparing the simulated wave height distribution with the target Rayleigh distribution. Since Rayleigh distribution has no upper bound, the function must be truncated in high wave end (for example, cutoff at $H/H_{rms}=2.5$ or 2.0) to avoid unrealistically high waves. To save computational time, the distribution is also truncated at the low wave end to cutoff small waves that make no contribution to sediment transport. Figure 4 shows an example of the simulated time series and the comparison of the wave height histogram from 500 waves with the target Rayleigh distribution. Numerical experiments showed that approximately 100 waves are needed to produce a reasonable Rayleigh distribution and 500 waves will produce a very good one.

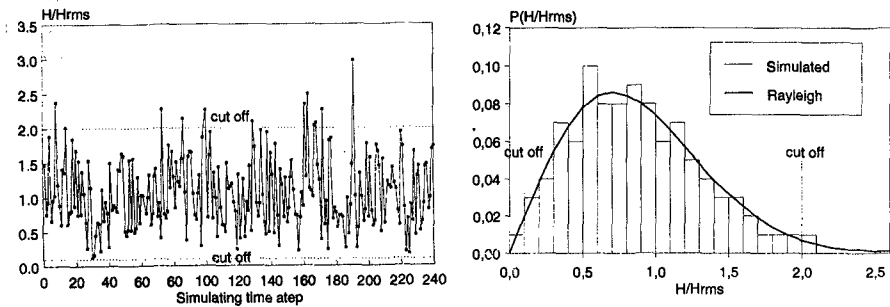


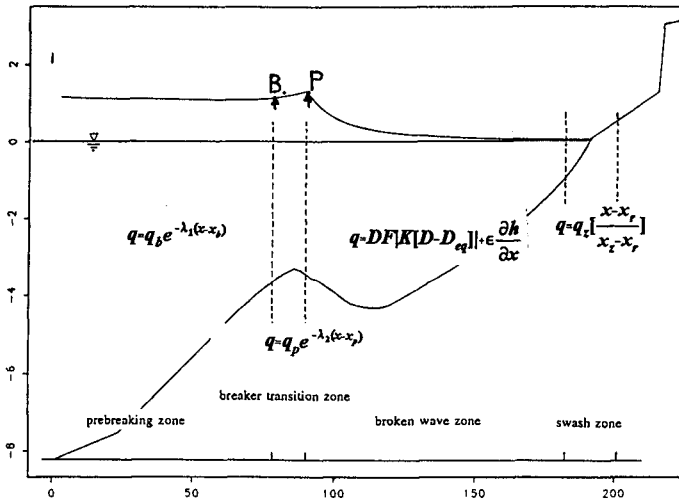
Figure 4 Example of Simulated Random Waves from Rayleigh Distribution

(2) Sediment Model

The selected sediment transport model is one of the macro-scale type which is based on the energy dissipation concept as used by Moore (1982), Kriebel and Dean (1985) and Larson and Kraus (1989). The reason for selecting this type is that the numerical results from the SBEACH model (Larson & Kraus, 1989) appear to yield the best overall agreement with our own laboratory experiments for regular waves. Their model is modified for random wave inputs.

The SBEACH model is expressly developed to incorporate the offshore bar formation and movement produced by storm waves and water levels. Bar formation and movement produced by breaking waves are satisfactorily simulated. In essence, the model is an extension of Kriebel and Dean's approach. The sediment transport model is patterned after Kriebel and Dean but empirically adjusted from large wave tank test results to insure bar formation and movement. The sediment transport is partially restricted to the equilibrium beach profile in the inshore zone of profile evolution. The sediment transport formulae proposed in this model is one of the possibilities of extending Dean's equilibrium beach profile concept to bar-berm profiles. The basic formulation is delineated in the following schematicst:

Schematics of Sediment Transport Model



This scheme is almost the same as the SBEACH with the exceptions that most of the coefficients have been adjusted. Also, the definition of the swash zone in the original model is defined in terms of controlling depth whereas in the present model it is defined in terms of percentage of the total surf width to produce more realistic nearshore profile. For detail see Wu (1994).

Model Sensitivity and Stability

Systematic sensitivity analysis to assess the influence of model parameters and empirical coefficients has been performed, with both regular wave and random wave. This allows us to examine the physical implications of the model parameters and their effects. It also serves to explore the applicability of the model beyond the range for which it was calibrated.

Related to time scale of simulation, with increase of total simulating time in both regular and random wave cases, the change of profile will reach quasi equilibrium form. Regular waves are not sensitive to the size of time step, Δt , which regulate the number of iterations. But for random waves, the time step (used Δt : $5T_p$ - $80T_p$, T_p : peak period) must be small enough to include a sufficient number of sample waves (at least 100 waves) to represent the Rayleigh distribution. Model sensitivity to wave height H_o or H_b , wave period T or T_p , change of water level, and breaker index were analyzed. The profile evolution is significantly affected by the change of wave height, Fig. 5, but it is not sensitive to the wave period. Under given wave condition, beach erosion always increases with increasing water level as the surf zone moves inshore with increasing water level, Fig. 6. The influence of sediment transport model

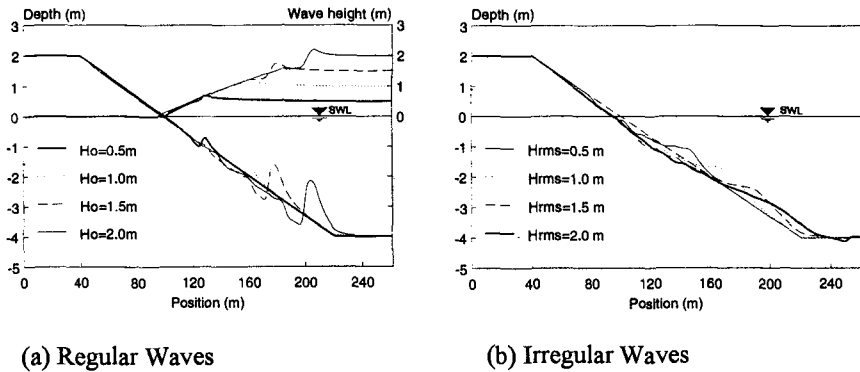


Fig. 5. Effects of Wave Height on Profile response

parameters, i.e. the sediment diameter D_{50} , fall velocity w and coefficient of equilibrium beach profile A were evaluated. As expected, a finer sand will produce flatter beach slope and coarse sand a steep profile. The influence of the coefficient of sediment transport rate K only affects the rate of transport but has very little effect on the final profile.

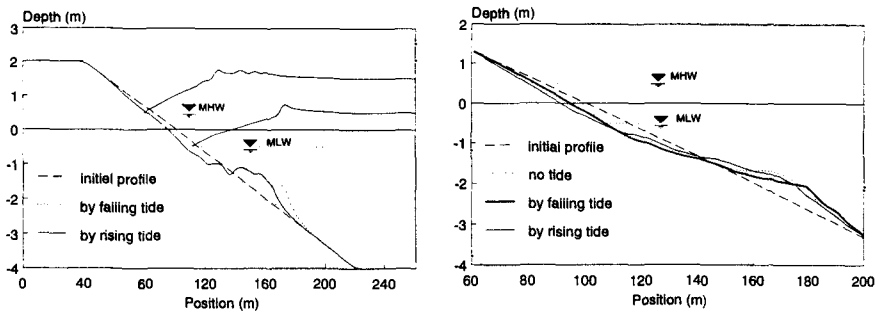
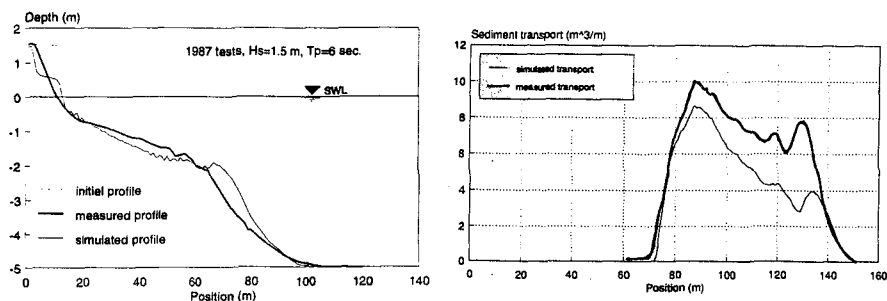


Figure 6. Effect of Water Level Changes on Profile Response

Comparison Between Numerical and Experimental Results

The model simulations are compared with limited cases of experimental results from the Large Wave Flume. Fig. 7 shows the comparison between the numerical simulation and the experiments of 1987. As can be seen, the step-type profile is fairly well represented so is the transport rate. In the 1990 and 91 experiments, the initial profile was shaped in accordance to an equilibrium form (Dean, 1977) with $A=0.07$. Both numerical and experiment results showed only small changes. The results

indicate that the method is capable of simulating equilibrium profile shapes under random wave conditions.



(a) Profile Comparison

(b) Transport Volume Comparison

Figure 7. Model Simulations, 1987 Random Wave Test

Figure 8 shows the comparison of numerical simulation with the experimental result of tests in 1993. The model simulation for phase I test is shown in **Fig. 8a**. The test consisted of a combined erosional cycle with an accretional cycle with regular wave input. It is evident that the numerical model was not successful for the accretional cycle. In the phase II test of irregular waves, the numerical model was reasonably successful for the erosional cycle as shown in **Fig. 8b**. The surf zone became broader, the bar decreased and its position moved in onshore direction. Near the shore line, the dune erosion is somewhat over estimated by the model. Again, the simulation was unsuccessful in the accretional cycle (not shown). In test phase III-1, random waves of 5 sec period were run with change of water level. The model simulation was successful for this erosional cycle. The measured final profile was well represented by the simulated one and it approached to an equilibrium form, **Fig 8c**. In the phase III-2, longer waves were run, which produced onshore sediment movement. The model, again, failed to simulate this case partly because the onshore-offshore criterion used in the model by Larson and Kraus (1989) suggests offshore transport for $H_{1/3} = 1.1$ m and $T_p = 10$ sec whereas in the experiment the sediment was clearly onshore. The criteria from Dean (1973), Hattori and Kawamata (1980) also indicate offshore sediment transport; only the criterion from Sunamura and Horikawa (1975) indicate onshore sediment transport. For the specific case simulated, one actually found that the absolute magnitude of transport volume measured in the laboratory was about the same order as that predicted by the numerical model, **Fig. 8d**.

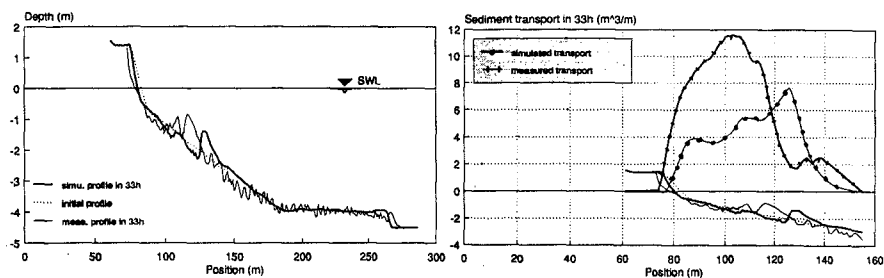


Figure 8a Numerical Simulation for Phase I, 1993

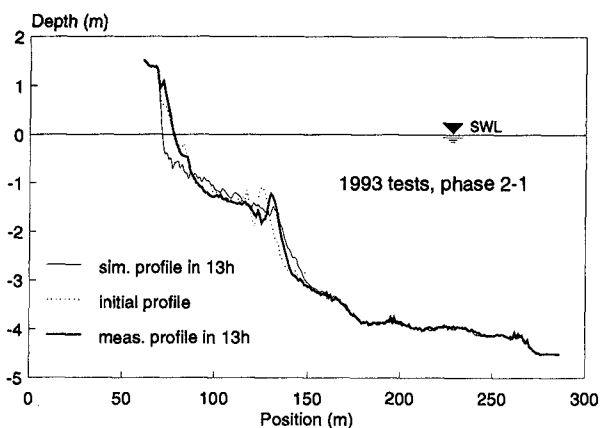


Figure 8b. Numerical Simulation for Phase II-1, 1993

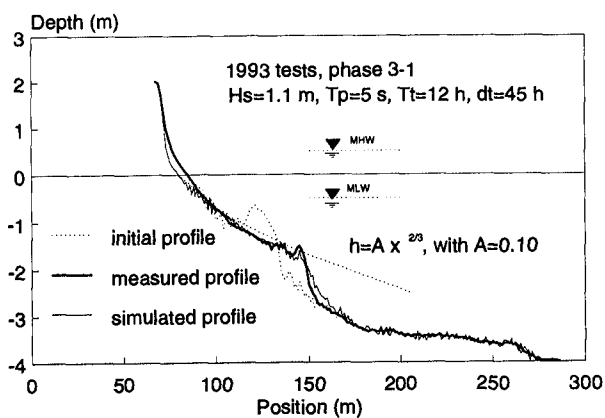


Figure 8c. Numerical Simulations for Phase III-1, 1993

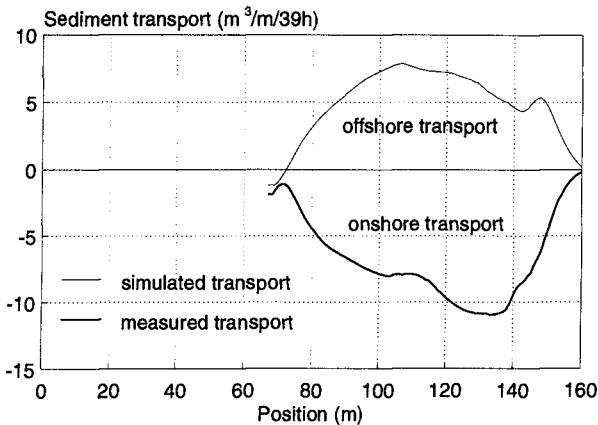


Figure 8d. Rate of Transport Simulated in Phase III-2, 1993

Conclusions

A systematic series of experiments have been carried out in the LWF to study the beach profile response under random wave conditions. A 2DV numerical model was also developed based on the SBEACH model. The study was aimed at documenting the laboratory observation as well as evaluating numerical modelling capabilities. It was found that:

Under erosional condition, berm profiles were generated under random wave instead of the bar profile under regular waves.

Beach recovery was difficult without water level changes and random wave input.

Beach profiles appeared to be able to reach quasi-equilibrium under both erosional and accretional cases and for both regular and irregular waves of constant conditions. At times, the profile appeared to have reached an equilibrium only to become active again due to unknown causes; it will then reach an equilibrium the second time which was usually more stable.

The numerical model adequately predicted the proper shapes of the profile for both regular and irregular waves under erosional condition in both profile shape and rate of erosion. The model as presented in the present study responds well to changes in water level and is numerically stable. The original SBEACH model, at occasions, becomes unstable. The numerical model as presented was not capable of reproducing the profile under accretional condition.

The current erosional and accretional criteria as proposed by various authors should be re-examined.

Further development of the profile model is required, possibly by (1) extending the model for simulation of beach accretion or combined beach erosion-accretion; (2) extending the present "discrete pdf" to a joint wave height and period distribution to accommodate the broad banded random sea; (3) taking the nonlinear wave transformation into account.

Acknowledgement

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